Executive Summary

This report describes the basis for the design of modifications planned for Mill Pond Dam. This final design report forms part of the submittal to the Department of Conservation and Recreation (DCR) Office of Dam Safety (ODS) as required by the Dam Safety Regulations (302 CMR 10.09) for a Chapter 253 Permit to repair the Mill Pond Dam. This final design report is submitted with (1) a series of photos from 2001 and 2006 flood events (Appendix A), (2) selected site photographs (Appendix B), (3) a completed Chapter 253 Permit Application (Appendix C), (4) a series of technical memorandums (Appendices D through G), (5) a low-level outlet waiver request letter (Appendix H), (6) a preliminary construction schedule (Appendix I), (7) the design drawings (Volume II), and (8) technical specifications for the proposed modifications (Volume III).

Mill Pond Dam is owned and operated by the Town of Rockport, Massachusetts for historical, aesthetic, and recreational purposes. The dam is approximately 110 feet long and about 13.7 feet high at the maximum section. Under the normal pond conditions, the elevation of the water surface is approximately 27.3 feet[1] with an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Under the design flood conditions (the 100-year flood, discussed below) the surface area and volume are estimated to be approximately 3.4 acres and 10 acre-feet, respectively.

The repairs and modifications to the dam are intended (1) to increase the stability of the dam, (2) to prevent the dam from being breached due to internal erosion or piping, (3) to provide overtopping protection and decrease the risk of failure due to erosion or scour during overtopping, (4) to contain flood flows and direct them to the overflow section, and (5) to bring the dam into compliance with ODS Standards. Spillway capacity will not be increased, and therefore, the dam will be designed to safely overtop during the design storm. Partial drawdown of the pond to about El. 22.0 (5.3 feet below normal pool) will be required to complete the majority of this work.

The proposed modifications include:

- Demolishing the existing stone masonry parapet walls.
- Removing and replacing the upstream and downstream granite masonry walls and earthen core.
- Constructing a new reinforced concrete wall near the existing downstream face and a new stone masonry wall along the alignment of the existing upstream stone masonry wall.

[1] Elevations are referenced to NGVD, 1929.

- Reinstalling the existing granite masonry on the downstream face of the new retaining wall to preserve the aesthetic and historic appearance of the dam.
- Installing a filter sand drainage layer under the wall footing, and a granular filter at the toe of the dam.
- Removing and replacing the existing granite slab culvert spillways.
- Constructing a masonry and steel handrail system along the downstream side of the crest and a granite bench system on the upstream side of the crest.
- Constructing a parapet wall along the left abutment to direct flows to the overflow section of the dam and reduce damage to private property and historic structures.
- Regrading and armoring portions of the right abutment to reduce erosion during overtopping.
- Installing bituminous concrete pavement or concrete pavers along the dam crest to provide erosion protection during overtopping.

Currently, no low-level outlet exists at Mill Pond Dam. A former mill sluiceway does exist near the left abutment; however, its current intake is about 1.5 feet above the normal pool elevation. We also understand that the sluiceway is inoperable. In January 2010, we submitted a letter to ODS requesting a waiver from the low-level outlet requirements in 302 CMR 10.14 (7), and have included a copy of this letter in Appendix H.

The existing spillway currently operates as an inlet-controlled culvert spillway. The existing spillway consists of two rectangular granite culverts, which are each approximately 2.5 feet wide and 2.5 feet high, located on the right side of the dam. The invert elevation will remain at El. 25.6 and the culverts will be reconnected with a 2% slope towards the downstream face to allow drainage. The capacity of the culverts will not be changed.

Due to historical constraints, the existing vertical granite slab on the right or north end of the downstream side of the former mill sluiceway cannot be removed during construction. However, it is our intent to support the vertical granite slab during construction and extend the toe of the wall footing to the vertical granite slab of the former sluiceway. A portion of the granite block sluiceway walls will be demolished to allow the construction of the new cantilevered retaining wall, and the void between the wall stem and the existing granite facing will be backfilled with flowable fill.

A number of large trees currently exist on or near the dam, and these trees will be removed prior to the construction of the dam safety modifications. In accordance with safe dam practices, an approximately 20-foot-wide buffer will be maintained between the downstream toe of the dam and any woody vegetation.

An eel ladder will be provided on the downstream side of the dam that will allow eels to pass upstream through the spillway culverts.

1. Introduction

1.1 Purpose

This report describes the basis for the design of modifications that are planned for Mill Pond Dam. This final design report forms part of the submittal to the Department of Conservation and Recreation (DCR) Office of Dam Safety (ODS) as required by the Dam Safety Regulations (302 CMR 10.09) for a Chapter 253 Permit to repair Mill Pond Dam. This final design report is submitted with: (1) Description of 2001 and 2006 Flood Events (Appendix A); (2) Selected Site Photographs (Appendix B); (3) Completed Chapter 253 Permit Application (Appendix C); (4) Series of Technical Memorandums (Appendices D through G); (5) Copy of our Low-Level Outlet (LLO) Waiver Request Letter (Appendix H); (6) Preliminary Construction Schedule (Appendix I); (7) 100% Design Drawings (Volume II); (8) 100% Technical Specifications for the Proposed Modifications (Volume III).

1.2 Existing Conditions and Dam Safety Deficiencies

1.2.1 Location

Mill Pond Dam is located in the Town of Rockport, Essex County, Massachusetts at latitude 42° 39’ 30” North, longitude 70° 37’ 24” West. The dam is located near the center of Rockport, about 30 miles northeast of Boston, Massachusetts. The pond and dam are located on Mill Brook, which flows in a northeasterly direction to the Atlantic Ocean (Sandy Bay) about 550 feet downstream of the dam. The dam and reservoir location is shown on the Site Location Map, provided as Fig. 1. An aerial view of Mill Pond Dam is presented in Fig. 2.

The dam can be accessed from Route 128, easterly to Gloucester, then to Rockport via Route 127 (Upper Main Street) to Route 127A (Beach Street) and in a northerly direction towards King Street. Prior to reaching King Street, the dam can be accessed from Beach Street by foot through the town park.

1.2.2 History

According to the Federal Emergency Management Agency (FEMA) Preliminary Engineering Report (FEMA, 2006, “Preliminary Engineering Report, Dam Site Visit Report, Mill Pond Dam, Rockport, MA,” July 12), the Mill Pond Dam was built in 1701 as a mill pond and was used until 1932 when the mill was destroyed by a fire. A resident near the dam, Mr. Martin Ross, stated to GEI that the pond has steadily become filled with sediment and was dredged in 1886 after a flood. Since 1980, Mr. Ross has observed the depth of the pond decrease due to sedimentation at a rate of 1.1 to 1.7 inches per year. He also stated that the dam was

overtopped in March 2001. Photographs taken by Mr. Ross during the 2001 flood are provided in Appendix A (Photographs A-1 to A-3). Based on precipitation data recorded in Beverly, Massachusetts, (approximately 15 miles southwest of Rockport), 4.3 inches of rain fell on March 22, 2001.

The dam was also overtopped during two days of heavy precipitation on May 13 and 14, 2006. The precipitation recorded on these two days in Beverly was 4.32 and 4.95 inches, respectively. As a result of the May 2006 flooding, large stone blocks on the downstream face became dislodged and a 45-foot-long portion of the downstream wall collapsed and toppled downstream of the dam. Since that event, the Town of Rockport has made temporary repairs to the dam by placing stone in the breached area and blocking vehicular traffic from crossing the dam crest. Photographs of the flooding and dam breach are provided in Appendix A as part of a memorandum prepared by Weston & Sampson Engineers, Inc. (Weston & Sampson) of Peabody, Massachusetts. The memorandum summarizes their Emergency Response Inspection on May 15, 2006.

1.2.3 Survey and Site Visits

GEI engineers visited the site on December 27, 2006, May 10, 2007, and August 4, 2009 to observe the existing site conditions, watershed, outlets, and area downstream of the dam. Selected photographs taken during GEI's site visits are provided in Appendix B.

BSC Group (BSC) of Boston, Massachusetts performed a topographic survey of the dam and downstream area in April and May of 2007 and revised the survey to include property line information in June and July of 2009. The survey is based on the Town of Rockport, Benchmark T26 (El. 62.89 feet) located in a ledge outcrop on the corner of Main and Hale Streets. The datum of the benchmark and elevations used in this report and our analyses is the National Geodedic Vertical Datum of 1929 (NGVD 29) datum. The existing conditions survey plan prepared by BSC is provided on Drawing C-03 in Volume II.

BSC's wetland scientists inspected the area around the dam to delineate wetland areas and to collect data as part of the permitting assessment. The wetland delineations marked in the field were surveyed by BSC surveyors and are shown on the site plan on Drawing C-03 (Volume II).

1.2.4 Dam Description

Mill Pond Dam is owned and operated by the Town of Rockport, Massachusetts for historical, aesthetic, and recreational purposes. The dam is approximately 110 feet long and about 13.7 feet high at the maximum section.

According to the FEMA Preliminary Engineering Report (FEMA, 2006), the dam was constructed with a clay core and hand-laid unmortared stone walls on the upstream and

downstream face. Based on our observations of the subsurface conditions encountered in recent borings, the core of the dam appears to be a silty sand. The crest of the dam consists of stone aggregate surface with field stone parapet walls that are each about 95 feet long and extend along most of the upstream and downstream face of the dam. Because the parapets do not extend across the entire crest of the dam they act more as guard rails than parapet walls. It is speculated that the parapets were added to the dam in the 1930's and may have contributed to the dam failure by causing the pond level to reach higher levels during flood events. Under the normal pond conditions, the elevation of the water surface is approximately 27.3 feet with an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Under the design flood conditions (the 100-year flood, discussed below) the surface area and volume are estimated to be approximately 3.4 acres and 10 acre-feet.

The crest of the dam varies from El. 28.5 to 29.5 feet and is about 15 feet wide at the narrowest section. The upstream and downstream faces of the dam are nearly vertical. Only about 4 feet of the upstream face is exposed above the sediment in the pond. The tops of the parapet walls are about 2.0 feet above the dam crest at El. 31.0 feet but do not extend along the full length of the dam. The highest section of the dam is located near the former mill sluiceway on the left side of the dam (looking downstream). The ground surface at the base of the downstream masonry wall near the former sluiceway is at El. 15.8 feet. Based on a maximum crest of El. 29.5 feet, the maximum height of the dam is about El. 13.7 feet.

The dam appears to have been constructed without a filter zone to control seepage and internal erosion or piping through the downstream masonry wall. We observed significant seepage flowing through the downstream face of the dam, just below the spillway outlet during our December 2006 site visit (see flow on left of spillway in Photograph B-9, Appendix B).

<u>Former Mill Sluiceway</u> – The intake to the sluiceway consists of a rectangular granite culvert on the upstream face of the dam (Photograph B-7). Vegetation and sedimentation along the upstream face of the dam have partially blocked the intake to this structure, and we understand that the sluiceway is inoperable. The intake opening is about 3 feet wide and the height varies from 0.7 feet on the left side to 1.5 feet on the right side. The current invert of the sluiceway intake is approximately El. 28.9 feet; however, the historic invert elevation is unknown. The inlet was constructed with several granite slabs that served as the top of the sluiceway intake structure and the crest of the dam. The condition and dimensions of the sluiceway inside the dam are unknown.

The blocked inlet apparently leads to a chamber within the dam with an 8-inch-diameter cast iron pipe and valve that exits on the downstream face of the dam (Photograph B-6). There is a remnant of an old control device on the crest of the dam above the outlet pipe that likely operated a vertical slide gate. The Town of Rockport reports that the controls and the valve

are not operational. Based on photos taken during the 2006 breach, we believe that the sluiceway walls are constructed from granite blocks that extend through the dam. The bottom elevation of the sluiceway is unknown, and the Town of Rockport has informed us that a majority of the sluiceway was backfilled with crushed stone or gravel as part of the temporary repairs following the 2006 breach.

Spillway – The Mill Pond Dam spillway consists of two rectangular granite culverts which are each approximately 2.5 feet wide and 2.5 feet high located on the right side of the dam (Photographs B-8 and B-9). The upstream invert of the culverts is at about El. 25.6 feet with a slightly higher outlet elevation on the downstream side at about El. 26.0 feet. Based on an assumed normal pond elevation of 27.3 feet and a normal base flow of 1.3 cubic feet per second (cfs), we estimated that stoplogs about 1.6 feet high were historically installed in the spillway culverts to maintain the normal pond elevation.

1.2.5 Potential Downstream Hazards

Flow from the dam is contained within Mill Brook. The town maintains a park (Mill Brook Meadow) immediately downstream of the dam. The park includes a gravel walkway and a grassy area that contains a small playground. There are several residential structures located on the left side (looking downstream) of the brook. The sill elevations (first or ground floor) of the residential properties range from about El. 19 to El. 15 feet. The house that almost abuts the dam on the left side of the brook has a walkout basement with an estimated elevation of about 17 feet. Commercial properties (bed and breakfast/restaurant establishments) are located on the left and right side of Mill Brook near Beach Street, about 550 feet downstream of the dam. The first floor elevations of these structures range from approximately El. 15 to 19 feet. Mill Brook passes through a granite block culvert under Beach Street, which has a paved surface elevation of about 15 feet.

1.3 Design Criteria

Mill Pond Dam is classified by 302 CMR 10.06 as a Small, Significant Hazard (Class II) dam. Pursuant to 302 CMR 10.14, a dam of this classification is required to pass or store the potential spillway design flood (SDF), equal to or greater than the 100-year storm per 302 CMR 10.14 (6) without failure of the dam occurring. The dam is currently not designed to safely pass this flow. Due to historical constraints, the historical granite culvert spillways will remain. Therefore, the spillway capacity will not be significantly increased and the dam will be overtopped by the SDF. The proposed modifications will allow Mill Pond Dam to safely pass the SDF by overtopping and provide protection against the risk of failure due to internal erosion or piping. The modifications will also provide a stabilized crest, a splash apron, and riprap armoring at the toe of the dam to reduce erosion and scour during overtopping. A summary of the parameters used during design are provided in Appendix D, and detailed Hydrologic and Hydraulic (H&H) Calculations are provided in Appendix E.

The proposed modifications do not change the pond or the watershed from their current condition. Removal of trees will be required downstream and along the abutments for the proposed modifications.

1.4 Proposed Modifications

1.4.1 Dam Repair and Reconstruction Activities

The repairs and modifications to the dam are intended (1) to increase the stability of the dam, (2) to protect the dam from internal erosion or piping, (3) to provide overtopping protection and decrease the risk of failure due to erosion or scour during overtopping, (4) to contain flood flows and direct them to the overflow section, and (5) to bring the dam into compliance with ODS Standards. Design Drawings for the repairs and modifications are included in Volume II for reference. Technical Specifications are included in Volume III.

The existing granite masonry walls and earthen core will be removed and replaced with a reinforced concrete cantilevered retaining wall. The existing stone masonry parapet walls will also be removed. The new retaining wall has been designed to resist active soil pressures and hydrostatic pressures. A filter sand drainage layer will be installed directly below the wall footing, and a granular filter will be constructed at the downstream toe of the dam. The filters will help to safely collect and discharge under-seepage flows and relieve uplift pressures. The existing stone masonry on the downstream face will be reinstalled over the concrete retaining wall to preserve the aesthetic and historic appearance of the dam, and the upstream stone masonry wall will be reconstructed. The parapet walls along the crest will be replaced with a granite bench on the upstream face of the dam and a handrail system on the downstream face of the dam. Replacing the parapet walls with a handrail will allow water and debris to safely pass over the reconstructed part of the dam during significant storm events at lower peak water surface elevation and allow a more uniform flow direction and velocity over the dam to reduce erosion and scour.

The top of the proposed crest will be at El. 29.0, about the same elevation as the existing crest. The crest surface will be stabilized with bituminous concrete or cobblestone pavers to protect against erosion during overtopping. Because the dam is being designed to be overtopped, the crest was not raised, and therefore does not provide any freeboard during the SDF. Due to historic and site constraints, the proposed repairs do not include significant modifications to the existing granite culvert spillways, and therefore the hydraulic efficiency of the spillways is unchanged. Our calculations indicate that during the SDF, the dam will be overtopped by up to about 2 feet of water for approximately 13 hours.

Improvements to the upstream side of the dam will only include the removal and replacement of the existing stone masonry walls with a new stone masonry wall and granite bench system. The new upstream stone masonry wall will absorb wave energy and prevent erosion of the backfill behind the new cantilevered retaining wall.

A parapet wall will be constructed on the left (west) abutment to prevent the design flood from overtopping the abutment and impacting a historic foundation and an abutting residence immediately downstream of the left abutment. The right abutment and the stone walkway will function as an overflow spillway. The gradual slope of the stone walkway and the right abutment might sustain surficial damage from the design storm but should not erode enough to jeopardize the dam.

Partial drawdown of the pond to about El. 22.0 (5.3 feet below normal pool) will be required to complete the majority of this work. The contractor will be required to design and implement a temporary cofferdam and bypass system for construction of the walls in the dry and to protect the work against flood flows that may occur during construction. Temporary cofferdam and bypass design by a Professional Engineer registered in the Commonwealth of Massachusetts will be required and will be subject to review by GEI and the Town of Rockport.

1.4.3 Low-Level Outlet

Currently, no low-level outlet exists at Mill Pond Dam. A former mill sluiceway does exist near the left abutment; however, its current intake is about 1.5 feet above the normal pool elevation. Refer to Section 1.2.4 above for more information about the former mill sluiceway. In January 2010, we submitted a letter to ODS requesting a waiver from the requirements in 302 CMR 10.14 (7), and have included a copy of this letter in Appendix H. We understand that ODS will evaluate this request as part of the Ch. 253 permit application review.

Because drawdown will not be required to reduce the loading on the dam and because of the small size of the Mill Pond impoundment (3 acre-feet), it is our opinion that the pond can be easily be drawn down with pumps owned or rented by the Rockport Department of Public Works (DPW) and that a low-level outlet would not be required for drawdown. Once construction is complete, drawdown will only be required for routine maintenance.

Given the relatively small size of the watershed, pond, and the normal inflow, our calculations indicate that the pond can be lowered using a mobile end suction centrifugal pump. The normal inflow is approximately 580 gallons per minute (gpm) (1.3 cfs) and the total volume of the pond at normal pool is approximately 3 acre-feet (1 million gallons). Therefore, assuming normal inflows, a pump operating at 800 gpm (e.g. Rain for Rent DV-100) could drain the pond in about 72 hours (3 days). A detailed Drawdown Plan will be included in the Operation and Maintenance Manual. A larger pump could lower the pond faster. However, a large increase in outflow may adversely affect the stream and other resource areas immediately downstream of the dam.

It is our opinion that a mechanical drawdown allows Mill Pond to be lowered in a safe and cost effective manner. Therefore, we request a waiver from the requirement in 302 CMR

10.14 (7) for a low-level outlet at Mill Pond Dam in Rockport, Massachusetts, and request that mechanical means be permitted to draw down Mill Pond for routine maintenance.

1.4.4 Spillway Reconstruction

The existing spillway currently operates as an inlet-controlled culvert spillway. The existing spillway consists of two rectangular granite culverts which are each approximately 2.5 feet wide and 2.5 feet high located on the right side of the dam.

In accordance with 302 CMR 10.14(6), the modifications to the Mill Pond Dam and spillway have been designed to allow the SDF to be stored and pass over the dam without failure occurring. The culvert spillway will flow full during the SDF, and the dam crest and right embankment will act as overflow sections to pass the remainder of the discharge from the SDF. We calculated that the existing spillway will not safely pass the SDF without overtopping the dam, and that the SDF would overtop the overflow section of the dam by up to about 2 feet for about 13 hours. We have included a stabilized crest, a reinforced concrete retaining wall, a splash apron, and riprap armoring in the design to minimize the risk of failure during overtopping.

Significant modifications to the spillway will not be performed as part of the remedial design. The spillway will be removed and reset in a reinforced concrete cradle with a seepage cutoff key near the upstream face of the dam. The invert elevation will remain at El. 26.5 and the culverts will be installed with a 2% slope towards the downstream face to allow drainage. The capacity of the culverts will not be changed.

1.4.5 Former Mill Sluiceway

Due to historical constraints, the existing vertical granite slab and abutting granite block walls on the north end of the downstream side of the former mill sluiceway will not be removed during construction. However, the Contractor will be required to support this section of wall during construction and extend the toe of the wall footing to the upstream side of the vertical granite slab and abutting granite block walls. A portion of the sluiceway walls internal to the dam will be demolished to allow the construction of the new cantilevered retaining wall, and the void between the wall stem and the existing granite walls will be backfilled with flowable fill. At the crest a parapet wall will extend to El. 33.0 feet to direct flood flows to the overflow section, minimize damage to the historic features (grist mill foundation and former mill sluiceway façade) and to provide fall protection.

1.4.6 Maintenance Activities

A number of large trees currently exist on or near the dam, and these trees will be removed prior to the construction of the dam safety modifications. In accordance with safe dam practices, an approximately 20-foot-wide buffer will be maintained between the downstream

toe of the dam and any woody vegetation. This area will act as a non-woody vegetated buffer and will be routinely mowed and trimmed to control the growth of woody vegetation that could damage the integrity of the dam.

2. Design Data and Analyses

2.1 Subsurface Investigations

In May 2009, we advanced two borings at Mill Pond Dam. B-1 was advanced to the west (left) of the former Mill Sluiceway and B-2 was advanced to the west (left) of the existing granite block culverts. Based on our review of photographs and discussions with Town of Rockport personnel, it is our opinion that the borings were advanced outside of the portions of the dam that were repaired following the 2006 breach. The boring logs and a boring location plan are provided in Appendix D.

2.2 Soil Conditions at the Dam

Two generalized soil layers were encountered in the borings:

- Embankment Fill
- Foundation Soil

The soil layers encountered in the borings are described below, in order of increasing depth. Bedrock was not encountered in either of the borings.

Embankment Fill – The embankment fill sampled is generally described as sand and gravel with a wide range of non-plastic fines content. Embankment fill was observed in both borings and had an average thickness of about 13 feet. Fines content in the samples ranged from less than 5% to approximately 40%. Based on Standard Penetration Test (SPT) N-values, the embankment fill can be described as loose to medium dense.

Foundation Soil – The foundation soils sampled generally consisted of non plastic to slightly plastic fines and sands with gravel and occasional cobbles and granite boulders. Based on regional geology, the foundation soils likely consist of glacial till, and by correlating SPT N-values, the foundation soils can be described as medium dense to very dense. Borings B-1 and B-2 were advanced from the dam crest (El. 29.0 ± feet) and were terminated in the foundation soil layer at a depth of 29 feet and 31 feet, respectively.

2.3 Groundwater Conditions

Water levels were not measured during our subsurface investigations, and as a result, water levels measurements are not provided in this report. During normal operating conditions, seepage has been observed in the downstream face. Therefore, we have assumed that the phreatic surface is approximately at the water surface elevation on the upstream side and slightly above tailwater elevation on the downstream face. Appendix F provides a more

detailed description of the phreatic surface and seepage forces that we modeled through the dam.

2.4 Seismic Design Information

From information provided in the Geologic Hazard Maps, we estimated that Mill Pond Dam is in Geologic Hazard Map Zone 3. Per 302 CMR 10.14 (i), we used a Peak Ground Acceleration (PGA) of 0.15g (gravity) in our analyses. Additional information is provided in Appendix G.

2.5 Dam Stability

As part of our design, we performed stability analyses on the cantilevered retaining wall and associated soil mass for a number of different loading conditions under normal and maximum pool. The factors of safety for the cantilever retaining wall calculated during our analyses were greater than or equal to the recommended minimums required by 302 CMR 10.14. Additional information is provided in Appendix G.

2.6 Safety During Overtopping

Hydraulic analyses were performed to determine the maximum reservoir level and overtopping flows associated with the SDF (the 100-year flood). In the proposed configuration, the pond would rise during the SDF to an elevation of 31.0, and overtop the dam by 2.0 feet for approximately 13 hours. The dam will be designed to overtop and should not experience a failure during the overtopping. The H&H analyses for the dam under the existing conditions and with the proposed modifications under the SDF are presented in Appendix E. The removal of the existing parapet walls and installation of a handrail system on the downstream side will provide open area that will allow flow and help reduce the duration and depth of overtopping. The construction of the proposed parapet walls on the left side of the dam and along the left abutment will help prevent flood flows from damaging historic structures and private property.

2.7 Seepage Controls

The proposed modifications to the dam address the current seepage issues that we identified during design. The reinforced cantilever retaining wall will be relatively water tight and will not be prone to failure caused by internal erosion or piping. The installation of the filter sand layer below the wall footing and the granular filter along the downstream toe will collect seepage flows and reduce the movement of fines. The filters will intercept under-seepage and allow seepage flows to be safely channeled below the dam to the downstream channel.

Because of the relatively short seepage path between the parapet wall and the existing historical wall near the left abutment, a layer of low permeability fill will be installed to

decrease gradients under the parapet wall footing and help the historic wall remain stable during the SDF.

Exit gradients are not of concern as the piezometric surface does not extend through the cantilever retaining wall and gradients are reduced by the more permeable filter sand layer under the wall footing. These gradients appear to have reasonable values as water moves from the sand filter under the dam and into granular filter. A detailed description of the seepage analyses is provided in Appendix F.

2.8 Spillway Design

The existing twin granite block culvert spillways will be removed and reconstructed in a concrete cradle at their existing locations and elevations. The re-installed spillway will discharge flow to a riprapped stilling pool in the downstream channel. Riprap armoring and a concrete splash apron will be installed below the water level in the pool. The flow will then travel down the existing channel, under Beach Street, and enter the Atlantic Ocean (Sandy Bay) about 550 feet downstream of the dam.

The calculated spillway capacity with stoplogs removed is about 130 cfs, and the estimated peak discharge during the SDF is 670 cfs. Therefore, the spillway does not have adequate capacity to pass the SDF and the dam will act as an overflow structure to safely pass the SDF. Detailed H&H analyses for the spillway are provided in Appendix E and the structural analyses are provided in Appendix G.

Stoplog guides will be installed on the upstream side of the culvert spillway and an approach channel will extend upstream of the spillway to channel flow to the spillway. The stoplogs will be used to vary the pond level, allow eel migration, and also allow the Town of Rockport DPW to lower the normal pool by about 2 feet for periodic maintenance. The stoplogs will be removed during any large storm event.

2.9 Crest Protection

The proposed modifications include installing concrete or cobblestone pavers or bituminous paving along the crest of the dam, the left and right abutments, and the path down to the meadow on the right abutment. The features will help prevent soil erosion during overtopping and should require minimal maintenance after an overtopping event. The new crest and abutment surfaces will be designed to permit vehicular access across the dam crest for maintenance.

2.10 Eel Ladder

The reconstructed dam will provide passage for American Eels during their migration to and from Mill Pond. American Eels are the only catadromous fish in North America. The term

catadromous refers to fish that are born in the ocean, travel to fresh water to mature, and then return to the ocean to spawn. Currently, the eels are thought to crawl through the stone masonry walls and burrow through the soil to reach Mill Pond. However, since the eels will not be able to penetrate the new reinforced concrete retaining wall, the reconstructed dam must provide alternate passage via an eel ladder or ramp.

The eel ladder will consist of a steel trough and an eel substrate attached to the bottom of the trough to allow the eels to move up the ladder. The ladder will be permanently attached to the downstream face of the dam. The ladder will be located on the right side of the dam, near the spillway culverts and will extend through the spillway culvert to provide passage from the downstream channel to Mill Pond. Normal flow through the culvert should provide adequate flows to attract eels to the ladder and allow them to ascend the ladder. When normal fluctuations in flow rate occur, we expect that the eels will wait in the downstream channel until favorable flows return.

3. Proposed Construction Activities

3.1 Construction Schedule

Construction is expected to begin in the fall or winter of 2010, and the overall project is anticipated to last approximately 6 months. We anticipate that the stream drawdown and bypass plan will control the construction sequencing and duration. Also, based on preliminary discussions with the various permitting agencies, we understand that significant bypass flows or construction in the downstream channel, and possibly all work will be prohibited during the American Eel migration between late February and early June. A construction schedule with an assumed start date of September 1, 2010 is included in Appendix I.

The project has been designed to allow the Contractor to determine the construction sequence that will provide the most efficient means and methods to construct the modifications while managing the pond level and bypassing storm flows to prevent damage to the work area. Temporary cofferdams, dewatering systems, and bypass systems will be designed and constructed by the contractor to provide a continuously dry working environment for retaining wall construction.

In order to complete construction activities, the upstream pond will be drawn down to El. 22.0, or about 5.3 feet below normal pool elevation. This drawdown will allow sufficient space for the contractor to perform the upstream excavation, and the lower water level also will decrease the seepage forces during excavation for the retaining wall and filter layers.

3.2 Proposed Filling Schedule

We have estimated that the volume required to fill Mill Pond from El. 22.0 to El. 27.3 is approximately 2 acre-feet. Assuming a normal inflow of 1.3 cfs, we estimate that it will take approximately 1 day to fill the pond up to the normal pool elevation of 27.3 feet. Filling to this level will require the installation of stoplogs to raise the invert elevation of the granite culvert spillways from El. 25.6 to El. 27.3.

3.3 Construction Impacts and Mitigation

The project will implement a number of measures during the construction period to provide general mitigation of impacts on surrounding uses. These specific mitigation measures will include:

3.3.1 Erosion and Sedimentation Control

Siltation barriers composed of double-staked hay bales and trenched silt fence, or similar, will be installed along the limit of work boundary. The siltation barriers will demarcate the limit of work, form a work envelope, and provide additional assurance that construction equipment not inadvertently leave the work area. All barriers will remain in place until disturbed areas are stabilized.

3.3.2 Construction Stockpiling Location

The contractor will not be permitted to store soil, gravel, or construction debris outside of the equipment and material staging areas shown on the drawings without specific approval from the Rockport Conservation Commission (RCC).

3.3.3 Dewatering Measures

Dewatering will be required for project excavations. Specifically, the contractor must lower groundwater levels to at least 2 feet below excavation bottoms. This dewatering will be critical, especially for installation of the retaining wall footing and sand filter drain.

Groundwater from dewatering will be pumped into temporary settling basins, which will help collect sediment during dewatering. The temporary settling basins will be located within the 100-foot buffer zone and be situated within close proximity of daily work activities. The contract documents include minimum requirements to ensure that proper dewatering controls are used to prevent erosion or sedimentation into wetland resource areas.

4. Project Permitting

The project requires the following Federal and State Permits. A description of the project's applicability is also provided below:

Federal Permits

- United States Army Corps of Engineers (USACE) Category II Screening under the Massachusetts Programmatic General Permit under Section 404 of the Federal Clean Water Act.
- Dewatering General Permit (DGP) under the National Pollutant Discharge Elimination System (NPDES) Program.

State/Local Permits

- Order of Conditions under the Massachusetts Wetlands Protection Act (M.G.L. c. 131 s.40), to be issued by the RCC, or Massachusetts Department of Environmental Protection (MassDEP) upon appeal (a request for Department action).

4.1 Category II Screening

Section 404 of the Federal Clean Water Act regulates the discharge of dredged or fill material into waters of the United States and adjacent wetlands. In Massachusetts, Section 404 is administered by the USACE. Remedial repairs to Mill Pond Dam will result in less than 5,000 square feet in jurisdictional waters/wetlands, normally qualifying the project as a Category I (Non-Reporting) project. However, projects involving water diversions are excluded from Category I and thus require screening under Category II. A Section 404 application was submitted to USACE on December 11, 2009.

As part of the Category II Screening process, consistency review by the Massachusetts Historical Commission (MHC) and the Massachusetts Coastal Zone Management (CZM) is also required. Copies of the Section 404 application were provided to the MHC and CZM on December 11, 2009.

4.2 Order of Conditions

A Notice of Intent (MassDEP File No. 062-0587) was filed with the Town of Rockport Conservation Commission on September 23, 2009 requesting authorization to make remedial dam repairs adjacent to and within jurisdictional wetland resource areas. The RCC issued an Order of Conditions in March 2010.

4.3 Dewatering General Permit

The total land disturbance area associated with the project measures approximately 0.5 acres. An NPDES General Permit for Storm Water Discharges from Construction Activities is not required for disturbance less than one acre. The project will require dewatering and will, therefore, require a DGP. The Contractor will be required to apply for and obtain the DGP through the U.S. Environmental Protection Agency (EPA).

4.4 Previous Permit Approvals

According to the MHC, the project area meets the Criteria Eligibility for listing on the National Register of Historic Places. The project has been designed to be in compliance with Section 106 of the National Historic Preservation Act of 1966 as amended (36 CFR 800) and Massachusetts General laws Chapter 9, Section 26-27C (950 CMR 71) and the terms of the FEMA Programmatic Agreement.

5. Maintenance and Operation Plan

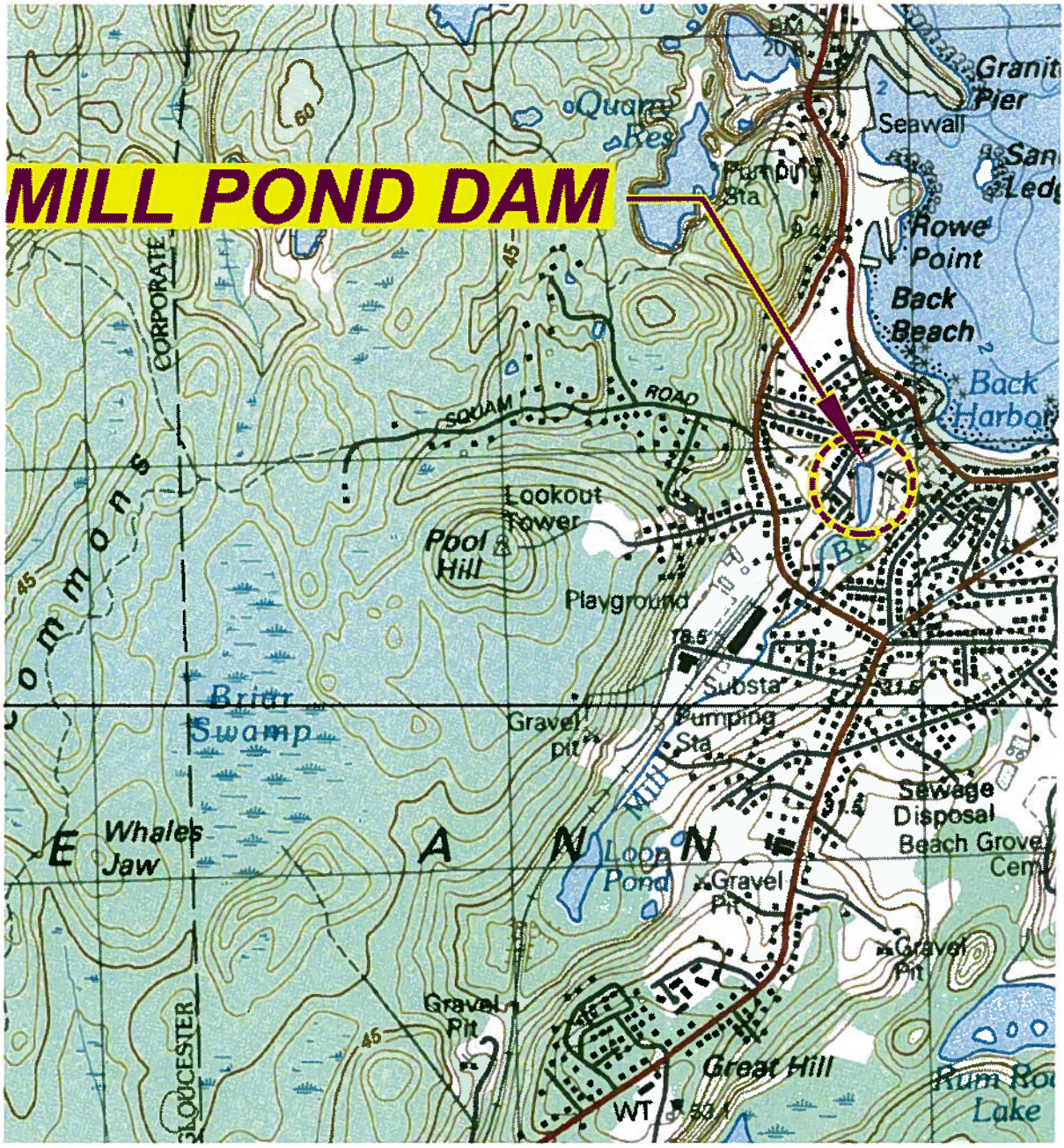
Maintenance at the Mill Pond Dam includes:

- Removing and replacing of stop logs to allow eel migration and adjust pond elevations as needed.
- Mowing the abutments and the downstream buffer zone to discourage growth of trees and shrubs.

A formal maintenance and operation plan will be submitted at a later date.

6. Emergency Action Plan

Per 302 CMR 10.11, an Emergency Action Plan (EAP) is not required for Mill Pond dam because the dam is not classified as a high hazard (Class I) dam.



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SCALE, FEET

This Image provided by MassGIS is from U.S.G.S.
Topographic 7.5 X 15 Minute Series
Rockport, MA Quadrangle, 1984.
Datum is National Geodetic Vertical Datum (NGVD).
Contour Interval is 3 Meters.

MASSACHUSETTS
QUADRANGLE LOCATION

Mill Pond Dam Safety Modifications Rockport, Massachusetts	GEI Consultants	SITE LOCATION MAP
Town of Rockport Rockport, Massachusetts	Project 07104-1	April 2010 Fig. 1

M:\DATA_DESIGN\071041 Rockport Mill Pond Dam\Ch253\Figures\Mill Pond Dam fig 1

Mill Pond Dam Safety Modifications Rockport, Massachusetts	GEI Consultants	AERIAL PHOTOGRAPH
Town of Rockport Rockport, Massachusetts	Project 07104-0	April 2010 Fig. 2

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Appendix A

Description of 2001 and 2006 Flood Events

Photograph A-1: 2001 Flood, view of pond and dam crest looking southeast.
Photograph courtesy of Marty Ross

Photograph A-2: downstream face of dam, post 2001 flood.
Note flow through the downstream face and the eroded crest
Photograph courtesy of Marty Ross

Photograph A-3: Damage to crest of dam, post 2001 flood.

Note flow around upstream rock wall, parallel to crest and out the downstream face

Photograph courtesy of Marty Ross

Weston & Sampson Engineers, Inc.
Five Centennial Drive
Peabody, Massachusetts 01960-7985
www.westonandsampson.com
Tel: (978)532-1900 Fax: (978)977-0100

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MEMORANDUM

TO: Bill Salomaa, DCR Office of Dam Safety

FROM: Mark Mitsch

DATE: May 15 & 16, 2006

SUBJECT: Report on Emergency Response Inspections
Mill Pond Dam
Rockport, MA

At approximately 1930 hours on 5/15/06, I arrived at Mill Pond Dam in Rockport, MA as requested by your office. The site visit was in response to a public safety emergency reported by the Town to MEMA and the Office of Dam Safety. I met with John Tomasz (Rockport DPW Director) to observe the condition of the dam. Approximate location of the dam is indicated on Figure 1.

The emergency was caused by a partial breach of Mill Pond Dam during the heavy rain events of the past week. The pond level rose above the crest of the dam prior to the breach (Photo 1 – provided to me by Eric Hutchins of NOAA). The overtopping flow overwhelmed the downstream channel, flooded the park area adjacent to the channel, and eventually led to a partial breach of the dam (Photos 2 through 6).

A complete breach (collapse of the upstream wall) during this rain event or during a future event could release the full impoundment downstream, which would threaten the homes and businesses (a restaurant and B&B on Beach Street) downstream of the dam, as well as the Beach Street public road. To mitigate the risk I recommended that the Town stabilize the breach area by backfilling the washout area with large riprap and then break down the upstream wall of the dam where water is flowing through (Photo 2) down to the level of the flow to limit the potential volume of impoundment.

I observed that the Town began the recommended emergency measures on 5/16/06.

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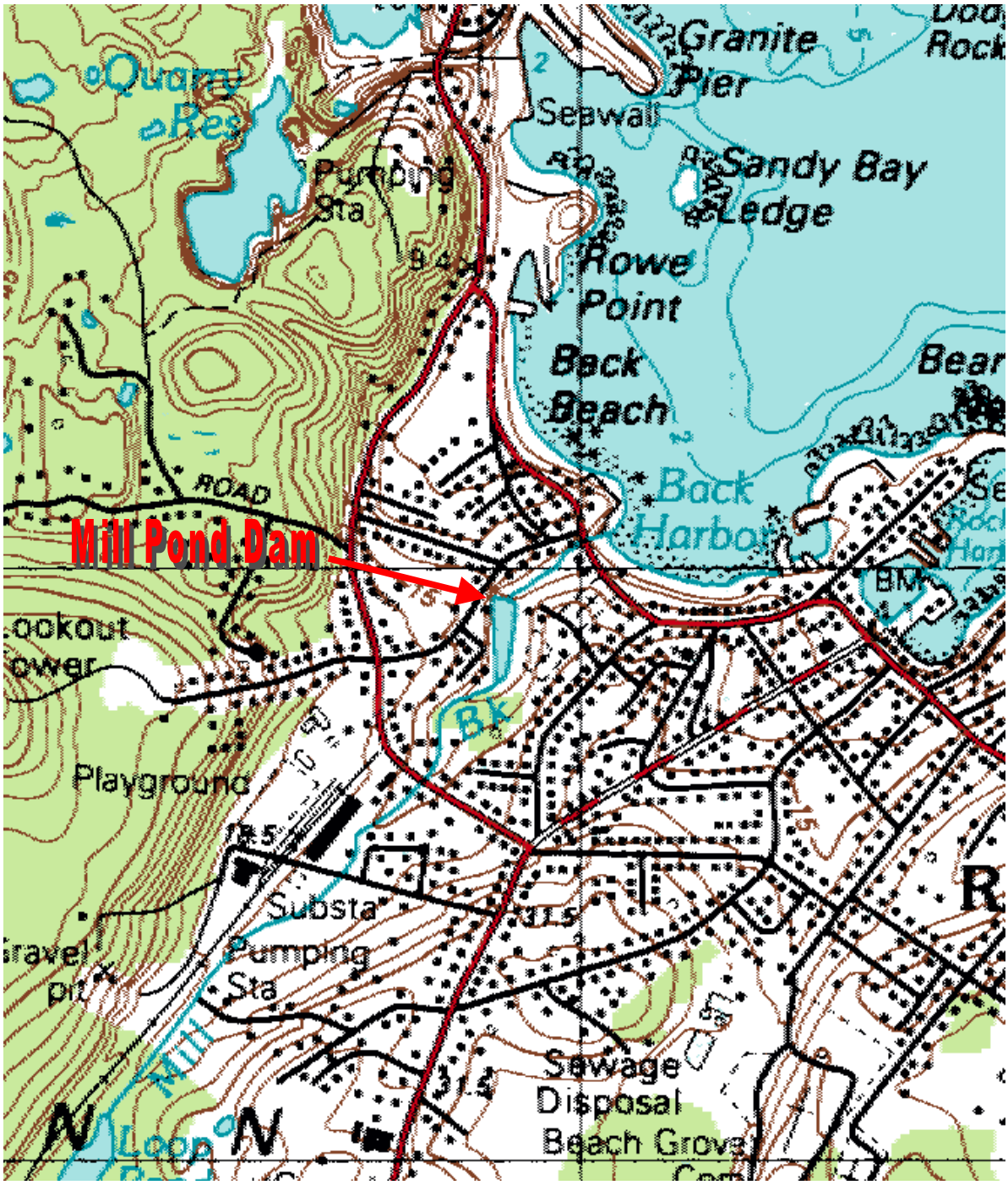

Figure 1 – Locus Map

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May 15, 2006 Rockport, MA Photographs

(1) Mill Pond Dam prior to breach from upstream left shore area.
Water level is above the dam crest (5/15/06 AM)

(2) Mill Pond Dam partial breach from left abutment 5/15/06 PM.

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(3) Mill Pond Dam partial breach looking downstream from left abutment 5/15/06 PM.

(4) Mill Pond Dam partial breach from downstream 5/15/06 PM.

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(5) Mill Pond Dam partial breach from right abutment (over primary spillway culvert).

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(6) Mill Pond Dam after riprap placement began and pond level reduced 5/18/06 AM

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Appendix B

Selected Site Photographs

Photo B1 – Panoramic composite* - Downstream wall of dam

Photo B2 – Panoramic composite* - Downstream wall of dam from a different location

Photo B3 – Panoramic composite* - Upstream side of dam

Photo B4 – Panoramic composite* - Left abutment area

Photo B5 – Former Sluiceway Outlet Control (not functional)

Photo B6 – Former Sluiceway Outlet (not functional)

Photo B7 – Former Sluiceway Intake (not functional)

Photo B8 – Spillway intake (functional)

Photo B9 – Spillway outfall

Photo B10 – Dam crest

*Note: Distortions and color or light irregularities in photos are due to discontinuities between individual photographic images used to create composite panoramas.

Appendix C

Completed Chapter 253 Permit Application

dcr
Massachusetts

JURISDICTIONAL DETERMINATION
CHAPTER 253 PERMIT APPLICATION

Dam Name: **Mill Pond Dam** **Date:** **April 2010**

Dam Location (City or Town)**:** **Rockport**

Owner(s) Name: **Town of Rockport – Department of Public Works**

Any person(s), who proposes to construct, repair, materially alter, breach or remove a dam, pursuant to M.G.L. Chapter 253, as amended by Chapter 330 of the Acts of 2002, must file with the Commissioner a permit application to determine whether or not a Chapter 253 Dam Safety Permit is required. Routine maintenance related work does not require a Chapter 253 Dam Safety Permit; however the owner(s) must file for a determination. No work is to commence before a determination is made by the Commissioner **(PART A)**.

If the Commissioner determines that the proposed work falls within the jurisdiction of M.G.L. Chapter 253 the Owner(s) must apply for a permit **(PART B)**.

The application and notices shall be sent by certified mail to DCR, Office of Dam Safety, Permits. All permit applications must comply with design and construction criteria as specified in 302 CMR 10.00: Dam Safety Rules and Regulations effective November 4, 2005.

Certain dams and reservoirs as defined in 302 CMR 10.00 are excluded from filing. Also, the approval of the Commissioner shall not apply to small dams or embankments constructed for irrigation, detention, storage tanks, or other purposes that impounds less than 15 acre-feet, regardless of height and is not in excess of 6 feet in height, regardless of storage capacity provide that any discharge(s) shall not materially affect property.

Any action taken by the Commissioner in regard to this application does not release the owner(s) from the requirements of any other law or regulatory authority.

Department of Conservation and Recreation
Office of Dam Safety
John Augustus Hall
180 Beaman Street
West Boylston, MA 01583
508-792-7716 508-792-7718 FAX
www.mass.gov/dcr

Deval L. Patrick
Governor

Timothy P. Murray
Lt. Governor

Ian A. Bowles, Secretary
Executive Office of Environmental Affairs

Richard K. Sullivan, Jr., Commissioner
Department of Conservation & Recreation

(PART A) **Notice of Jurisdictional Determination**

PER 302 CMR 10.00

(CHECK THE APPROPRIATE COLUMN)

1. Is there any risk to life and/or property resulting from flooding and/or a dam failure?
Yes(**X**) No( )

2. Is the structure classified as a roll/river dam?
Yes( ) No(**X**)

3. Does the maximum impoundment exceed 15 acre-feet (1 surface acre x 1 foot deep)?
Yes() No(**X**)

4. Does the height of the dam exceed 6 feet above the natural bed of the stream at any point?
Yes(**X)** No( )

If a question is answered **Yes**, then the Commissioner has the authority to determine the need for a permit. If the answer to any question is unknown**,** the applicant needs to request a determination, **(PART A)**. If all questions are answered **No** a written determination may still be requested from the Commissioner **(PART A).**

Any request for jurisdictional determination relative to work to be performed on, or affecting a dam, must contain the following basic information:

The present dam owner(s) name, address and telephone number based on Registry of Deeds records (Book and Page Numbers) and also Town/City records (Assessor maps and tax records).

A topographic or town map clearly indicating the dam location, impoundment, dam name and local access.

A plan view of the dam showing pertinent details: i.e. elevations, length, width, spillway, controls, buildings etc. Also the maximum cross section through the dam as measured from the lowest point.

Elevation(s) and surface area (acres) of the pond. Normal (spillway) and maximum (crest) volumes of water impounded (acre-feet) by the dam (measured or estimated).

A brief statement regarding possible damage and risk to life based on flood discharges and/or a dam breach.

Full nature of work to be performed and signature of owner(s) and/or applicant(s) submitting this request.

Photos of dam showing structures; i.e. gates, outlets, spillways. Also, any past history of repairs, flooding events, owners, etc.

(PART A cont.) **Regular Maintenance**

Further, any request to the Commissioner for a jurisdictional determination relative to maintenance work on a dam must complete the following section:

If considered regular maintenance or permitted water adjustments for pond maintenance, flood operation and/or other dam/pond activity.

Check Here_____

Typical maintenance activities include, but are not limited to the following items.

1. Normal water level pond drawdown? Yes( )No( )
2. Minor Earthwork/masonry maintenance and repair? Yes( )No( )
3. Riprap maintenance and repair? Yes( )No( )
4. Vegetation and tree maintenance (less 6"Diameter)? Yes( )No( )
5. Rodent damage control? Yes( )No( )
6. Traffic damage controls and erosion? Yes( )No( )
7. Mechanical maintenance to outlets? Yes( )No( )
8. Electrical maintenance? Yes( )No( )
9. Cleaning? Yes( )No( )
10. Concrete maintenance? Yes( )No( )
11. Metal component maintenance? Yes( )No( )
12. Other as specified: Yes( )No( )

Please describe the **nature of proposed regular maintenance** checked above by item number. Use additional paper if necessary.

See the Design Report for required information.

STOP HERE if requesting a project determination, If applying for a Permit proceed to PART B.

(PART B) **Application for Dam Safety Permit**

General Information:

1. Dam location (City/Town) attach locus map and local access to the dam.

   Rockport; See Figure 1 of the Design Report for locus map

2. Dam Name(s): **Mill Pond Dam**

3. Impoundment Name(s): **Mill Pond**

4. Assessor's Information (city or town tax assessors office):

   Map Number: **18**
   Section Number: **N/A**
   Lot(s) Number: **319, 325**
   Record Owner(s) and Address: **Town of Rockport, 34 Broadway, Rockport, Massachusetts 01966**

5. Registry Location: **Essex Registry of Deeds, Southern District**

   Book and Page: **N/A**

   Present and/or Prospective owner(s):

   Name: **Town of Rockport**
   Address: **34 Broadway, Rockport, MA 01966**

   Telephone: **978-546-3525**

6. Present or prospective abutter(s), for DCR funded or granted projects:

   Name:

   Address:_____

   Telephone:_____

7. Name of US Geological Survey map quadrangle: **Rockport**

8. Name of reservoir or waterway: **Mill Pond/Mill Brook**

9. Is there specific legislative authority to construct the dam? Yes( )No(**X**)

   If yes, identify:_____

10. Purpose of the dam structure? **Recreational Impoundment**

11. Nature of work to be performed: New dam Yes( )No(**X**), Alteration or major repair of existing dam Yes (**X**)No( ), Other repair or pond work effecting dam Yes( )No(**X**).

12. Is a MEPA Certification required to perform the proposed work? Yes()No(**X**)

(PART B cont.) **HAZARD POTENTIAL EVALUATION**

(Based on downstream field investigation)

1. The number of people that could be effected by failure or overtopping of the dam structure, and to what degree they could be effected:

The number of people that could be affected from a failure of the Mill Pond Dam is estimated to be less than 10 people based on visual observations at Mill Pond and the surrounding area. The loss of life is not expected from a failure or overtopping of the dam.

2. The estimated number of properties (homes, building, etc.) that could be effected by failure or overtopping of the dam structure, and to what degree they could be effected:

The town maintains a public park immediately downstream of the dam which includes a gravel walkway and a grassy area containing a small playground. There are three residential structures located on the left side (looking downstream) of the dam with sill elevations (first or ground floor) of El. 19 to El. 15 feet. The house that almost abuts the dam on the left side of the brook has a daylight basement with an estimated elevation of about 17 feet. Three Commercial properties (bed and breakfast/restaurant establishments) are located on the left and right side of Mill Brook near Beach Street. These structures are located about 400 to 500 feet downstream of the dam.

It is estimated that the park and playground area could be flooded by several inches to several feet as a result of a dam failure or dam overtopping. It is expected that depth of flooding on the residential and commercial properties downstream of the dam would be minimal.

3. The estimated roads or other structures that could be effected by failure or overtopping of the dam structure, and to what degree they could be effected:

The brook and discharge from the dam passes under Beach Street to the ocean through a 50-inch diameter culvert with an invert about seven feet below the road. The paved surface of the road is at approximately El. 15 feet. Overtopping of the road is to an unknown depth is possible depending on the rate of a breach failure.

(PART B cont.)

HYDROLOGIC, HYDRAULIC AND STRUCTURAL CONSIDERATIONS

Hydrologic, hydraulic and structural design procedures should be used, as established by one of the following: The U.S Army Corps of Engineers, the U.S. Bureau of Reclamation, the U.S. Soil Conservation Service and other procedures universally accepted as sound engineering practice.

1. Contributory drainage area (sq. mi.): **0.75 (Appendix E of the Design Report, Fig. 1)**
   (Attach topographic map with outline of drainage area)

2. Design storm duration: **100-year storm**

   Rainfall Intensity (inches/hour): Maximum Intensity = **0.275 inches/hour**

3. Runoff (%) **74** Inches: **6.6**

4. Peak Outflow (cfs): **670**

5. Previous known flood of record (month/year): **Flood of record unknown although the dam has overtopped in March 2001 and May 2006. As a result of the May 2006 flooding, large stone blocks on the downstream face became dislodged and a 45-foot-long portion of the downstream parapet wall collapsed and toppled downstream of the dam. Since that event, the Town of Rockport has made temporary repairs to the dam by placing stone in the breached area and blocking vehicular traffic from crossing the dam crest.**

6. Design maximum flood level elevation: **31.5 ft**

7. Additional information:

DESIGN CRITERIA

1. Datum used: MSL of 1929 (**X**) NAVD 88 _____ Assumed_____Other_____

2. Type of structure (earth, concrete, etc.): **Concrete Cantilever Retaining Wall with Soil Backfill**

3. Maximum structural height of the dam (feet): **14 +/-**

4. Crest length (ft): **107 +/- ; Crest width (ft): 14 +/-**

5. Top elevation of dam: **29.0**

6. Present river or channel elevation at dam (ft): **15.8 to 20.0 +/-**

7. Normal pool elevation (ft): **27.3**

8. Normal pool surface area (acre): **1.3**

9. Normal impoundment (acre-ft): **3**

10. Maximum pool elevation (ft): **100 year storm – 31.0**

11. Maximum pool surface area (acre): **3.4**

12. Maximum impoundment (acre-ft): **10**

13. Freeboard, as measured from the maximum design pool elevation to the crest of dam (ft): **- 2.0**

14. Nature of slope protection: **N/A**

15. Primary Spillway information:

Spillway type: **Twin Rectangular Granite Block Culverts**

Top elevation: **25.6 ft. (Invert)**

Dimensions (ft): **Two culverts each: 2.5 ft High by 2.5 Wide.**

Capacity (cfs): **120 cfs with WSEL at El. 31.0**

Percentage of design flood: **18%**

16. Emergency Spillway information:

Spillway type: **Dam overtopping**

Top elevation: **29.0**

Dimensions (ft): **85 ft long**

Capacity (cfs): **550 CFS (when water level is El. 31.0)**

Percentage of design flood: **82%**

17. Low Level Outlet (s): **None. See Waiver Request Letter provided in Appendix H of the Design Report.**

Type(s):

Invert elevation(s):

Dimension(s) (ft):

Capacity (cfs):

Percentage of design flood:

SUBSURFACE INVESTIGATION

Boring logs, analysis and recommendations to accompany this application.

Boring logs and design parameters are provided in Appendix D of the Design Report.

CONSTRUCTION DRAWINGS

Names and addresses of abutters to the proposed project must be clearly indicated on the construction drawings.
The construction drawings show present abutters and are provided in Volume II of the Design Report.

(PART B cont.) **SIGNATURE SECTION**

Applicant(s)

Name(s) **Town of Rockport, Department of Public Works**

Street: **34 Broadway**

City/Town: **Rockport** State: **MA** Zip: **01966**

Telephone: **978-546-3525** Fax:_____ Email Address: **jparisi@town.rockport.ma.us**

Signature and Title_____ Date:_____

Licensed professional civil engineer registered in Massachusetts.

Name: **R. Lee Wooten**

Company: **GEI Consultants, Inc.**

Street: **400 Unicorn Park Drive, 3rd Floor**

City/Town: **Woburn** State: **MA** Zip: **01801**

Telephone: **781-721-4000** Fax: **781-721-4073** Email Address: **lwooten@geiconsultants.com**

Signature:_____ Date:_____

Stamp and License Number: **31830**

Chapter 253 Application Fee and Permit Fees:

(a) The fee to apply for a Chapter 253 Dam Safety Permit to construct, materially alter, perform major repairs, breach or remove a dam is $50.00

(b) The fee for review and issuance of a Chapter 253 Dam Safety Permit is based on the size and cost of the proposed project (construction and engineering) as follows:

1. For a dam construction project costing up to $100,000.00 the fee will be $250.00
2. For a dam construction project costing from $100,000.00 to $500,000.00 the fee will be $500.00
3. For a dam construction project costing between $500,000.00 and $1,000,000.00 the fee will be $750.00
4. For any dam project over $1,000,000.00, the fee will be $1,000.00

The cost of the proposed project:

(a) The cost of engineering: ~$85,000
(b) The cost of construction: ~$800,000

The total cost: ~$885,000

Please enclosed check or money order payable to: The Commonwealth of Massachusetts, Office of Dam Safety – Permits Section

Exclusions: The Commonwealth, its agencies, authorities and political sub-divisions, including municipalities, are exempt from the payment of fees.

Minimum Submission Requirements:

With this application submit one set of bounded (utilizing plastic comb bindings) final design report with 11”x 17” design drawings. An electronic copy of the design report and drawings in PDF (compatible with Adobe Reader Version 6.0 or later) format presented on compact disc (CD-R media, closed for future recording). CD’s shall be protected by jewel case and contain a label indicating the dam project name, NID ID #, the Town in which the dam is located, and the date of the design report.

All required submittals shall be sent by certified mail to:

Department of Conservation and Recreation
Office of Dam Safety, Permits
John Augustus Hall
180 Beaman Street
West Boylston, MA01583

Approval or denial of a permit will be issued within 60 days from the time the final design report and permit application is received.

Appendix D

Technical Memo 1 – Design Parameters

Memo Title: Technical Memo 1 – Design Parameters

Project Title: Mill Pond Dam – Final Design Report

Project Location: Rockport, Massachusetts

Project #: 07104-1

By: Max Hurd **Date:** 4/27/10

Checked by: PJ Snyder **Date:** 4/28/10

TECHNICAL MEMORANDUM 1

Summary of Design Parameters
Mill Pond Dam
Rockport, MA
GEI Project #07104-1

The purpose of this technical memorandum is to summarize the design parameters that were used in civil, structural, and geotechnical analyses for the design of dam safety modifications to Mill Pond Dam located in Rockport, Massachusetts. All elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Design Storm	100 yr.
Crest Width	16-20 ft.
Normal Pool Elevation	El. 27.3 ft.
Dam Crest Elevation	El. 29.0 ft.
Spillway Culvert Invert Elevation	El. 25.6 ft.
Peak Water Surface (WS) Elev.	El. 31.0 ft.
Depth of Overtopping at Design Storm	2.0 ft.
Allowable Bearing Capacity	4 tsf
Culvert Dimensions (each)	2.5 ft x 2.5 ft.
Number of Culverts	2
Stop Log Invert	El. 27.3 ft.
Downstream Channel Elevation	Varies

Stoplogs will be used to maintain the pond at normal pool (NP) and allow the pond to be self regulating. The stoplog system will be used to lower pond levels for maintenance or to increase discharge capacity during periods of increased inflows.

1.0 Stratigraphy

We performed a subsurface investigation that consisted of two test borings drilled to approximately El. 0 ft. Based on the results of the test borings, we have assumed a stratigraphy of two discreet soil layers:

1.1 Layer 1 – Embankment Fill

The embankment fill soils exist in the embankment of the dam from the existing ground surface to depths of about 15 feet. The fill encountered in the test borings consisted mostly of widely graded sand with varying amounts of silt. Based on standard penetration test (SPT) n-values, the embankment fill can be described as loose to medium dense.

1.2 Layer 2 – Foundation Soil

The foundation soils were encountered beneath the embankment fill soils. Based on our review of the information collected during the investigation, we have classified the foundation soils as Glacial Till. Both borings were terminated in the till, and therefore its thickness is not known. The till encountered in the borings varied from widely graded sand with silt to silt with sand and based on SPT n-values can be described as medium dense to very dense. The borings also encountered frequent cobbles and boulders in this layer.

400 Unicorn Park Drive
Woburn, MA 01801
781.721.4000 fax 781. 721.4073

2.0 Design Parameters

We estimated design parameters for each subsurface layer based on empirical correlations to standard penetration test N-values, soil type, and fines content.

Based on our review of stratigraphy and our knowledge of empirical correlations, we estimate the following soil parameters to use in design:

Soil Layer	Unit Weight, γ (pcf)	Friction Angle, ϕ (degrees)	Permeability, k (cm/sec)
Embankment Fill	130	32	$1x10^{-2}$ to $1x10^{-4}$
Foundation Soil	135	35	$5x10^{-3}$ to $1x10^{-6}$

Logs for Borings B-1 and B-2 are attached.

3.0 Bearing Capacity

Bearing capacity was selected using the Allowable Bearing Capacity for Foundation Soils provided in the Massachusetts Building Code (Table 1804.3) for loose, Class 8 soils (silty sands). We selected a presumptive bearing capacity of 4 tons per square feet (tsf).

4.0 Soil and Aggregate Fill Materials

Refer to the project specifications for additional information, including gradation and other geotechnical criteria for the soil and aggregate fill materials listed below.

4.1 Embankment Fill

M.1.01.0 – Ordinary Borrow – Massachusetts Highway Department Standard Specifications for Highways and Bridges, 1988 (MHD88). Material conforming to AASHTO M-145 group A-1, A-2-4 or A-3. On-site material will be permitted to be re-used if it meets the MHD and AASHTO standards.

4.2 Impervious Fill

Impervious Fill shall be provided from one source and meet the ASTM D2487 soil classification for CL, CH, MH, or SC soils.

4.3 Sand for Aggregate Filter

MHD88 – M4.02.02A – Fine Aggregate, or similar material conforming to ASTM C- 33 Fine Aggregate.

4.4 Aggregate for Granular and Riprap Filter

ASTM C-33 No. 1 Stone.

5.0 References

1. Massachusetts Highway Department Standard Specifications for Highways and Bridges, 1988.

2. McGregor, J.A. and Duncan, J.M. (1998), "Performance and use of the Standard Penetration Test in Geotechnical Engineering Practice", Center for Geotechnical Practice and Research, Virginia Polytechnic Institute and State University.

3. Lambe, T.W., Whitman, R.V. (1969), Soil Mechanics, John Wiley & Sons.

4. United States Army Corps of Engineers, EM 1110-2-1901, Seepage Analysis and Control of Dams.

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Attachment 1

Boring Logs

BORING INFORMATION

LOCATION:
REFERENCE ELEV. (ft): NM
VERTICAL DATUM:
TOTAL DEPTH (ft): 29.0
LOGGED BY: H. Shields
DATE START/END: 5/7/2009 - 5/7/2009
DRILLING COMPANY: Geologic, Inc.
DRILLER NAME: Ray Eastwood
RIG TYPE:

BORING

B1

DRILLING INFORMATION

HAMMER TYPE: Automatic Hammer
AUGER I.D./O.D.: NA / NA
DRILLING METHOD: Rotary Wash
WATER LEVEL DEPTHS (ft): Not measured
CASING I.D./O.D.: 4 inch/ 4.5 inch
DRILL ROD O.D.:
CORE BARREL TYPE:
CORE BARREL I.D./O.D. NA / NA

ABBREVIATIONS:
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation
= Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
SC = Sonic Core
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D. = Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	PID Jar Headspace/ Drilling Remarks	Layer Name	Soil and Rock Description
		S1	0 to 0.2	2/10	1-6-7-10		FILL	S1: SILTY SAND (SM), ~70% widely graded sand, ~20% non-plastic fines, ~10% mostly fine gravel. Brown, sand is coarser near bottom of sample, more gravel in bottom 1 inch.
		S2	2 to 2.2	2/6	7-14-16-10			S2: WIDELY GRADED GRAVEL WITH SAND (GP), ~70% widely graded gravel up to 1 in, ~25% widely graded sand, <5% non-plastic fines. Brown.
	5	S3	4 to 4.2	2/3	7-5-9-14			S3: SILTY SAND (SM), ~60% widely graded sand, ~30% non-plastic fines, ~10% mostly fine gravel plue one 1-inch piece. Brown.
	10	S4	9 to 9.2	2/11	13-36-10-14			S4: SILTY SAND WITH GRAVEL (SM), ~40% widely graded sand, ~40% non-plastic fines, ~20% mostly fine gravel. Brown, slight organic odor. Higher gravel content near top of sample.
	15	S5	14 to 14.1	1/3	4-3/12"-100/1"	Piece of wood flushed out while cleaning out casing at 13 feet.	TILL	S5: SILT WITH SAND (ML), ~80% slightly plastic fines, ~10% widely graded sand, ~10% mostly fine gravel up to 1/2 inch. Gray, dense. Possibly till.
		C1	16 to 16.2	3/27				C1: GRANITE BOULDER
	20	S6	20 to 20	0/2	100/2"			S6: NARROWLY GRADED SAND WITH SILT (SP-SM), ~90% narrowly graded mostly fine sand, ~10% non-plastic fines. Light brown.
		C2	20.5 to 20.7	3/35				C2: GRANITE BOULDER

NOTES:

PROJECT NAME: Rockport Mill Pond Dam
CITY/STATE: Rockport, Massachusetts
GEI PROJECT NUMBER: 07104-1

GEI Consultants

GEI WOBURN STD 1-LOCATION-LAYER NAME 071041 ROCKPORT DAM.GPJ GEI DATA TEMPLATE.GDT 8/3/09

LOCATION:
REFERENCE ELEV. (ft): NM
VERTICAL DATUM:
DATE START/END: 5/7/2009 - 5/7/2009
DRILLING COMPANY: Geologic, Inc.

BORING B1

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	PID Jar Headspace/ Drilling Remarks	Layer Name	Soil and Rock Description
	25					Boulder at 25-27 feet.	TILL	
		S7	27 to 27.2	2/10	12-25-14-16			S7: SILTY SAND (SM), ~50% widely graded sand, ~40% slightly plastic fines, ~10% mostly fine gravel. Light brown, dense. Possibly till.
	30							Bottom of boring at depth 29 ft. Borehole backfilled with bentonite chips.
	35							
	40							
	45							
	50							
	55							

NOTES:

PROJECT NAME: Rockport Mill Pond Dam
CITY/STATE: Rockport, Massachusetts
GEI PROJECT NUMBER: 07104-1

GEI Consultants

GEI WOBURN STD 1-LOCATION-LAYER NAME 071041 ROCKPORT DAM.GPJ GEI DATA TEMPLATE.GDT 8/3/09

BORING INFORMATION

LOCATION:
REFERENCE ELEV. (ft): NM
VERTICAL DATUM:
TOTAL DEPTH (ft): 31.0
LOGGED BY: H. Shields
DATE START/END: 5/8/2009 - 5/8/2009
DRILLING COMPANY: Geologic, Inc.
DRILLER NAME: Ray Eastwood
RIG TYPE:

BORING

B2

DRILLING INFORMATION

HAMMER TYPE: Automatic Hammer
AUGER I.D./O.D.: NA / NA
DRILLING METHOD: Rotary Wash
WATER LEVEL DEPTHS (ft): Not measured
CASING I.D./O.D.: 4 inch/ 4.5 inch
DRILL ROD O.D.:
CORE BARREL TYPE:
CORE BARREL I.D./O.D. NA / NA

ABBREVIATIONS:
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation
= Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
SC = Sonic Core
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D. = Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	PID Jar Headspace/ Drilling Remarks	Layer Name	Soil and Rock Description
		S1	0 to 0.2	2/2	2-3-5-4			S1: SILTY SAND (SM), ~60% widely graded sand, ~30% non-plastic fines, ~10% widely graded gravel up to 1 in. Brown.
	5	S2	4 to 4.2	2/9	4-4-4-5			S2: NARROWLY GRADED SAND WITH SILT (SP-SM), ~85% narrowly graded mostly medium sand, ~10% non-plastic fines, <5% fine gravel. Brown.
	10	S3	9 to 9.2	2/0	3-7-14-14	S3 - No recovery. Pushed 2-in-OD split spoon 9-11 feet (S3A).		S3: No Recovery S3A: WIDELY GRADED SAND WITH SILT AND GRAVEL (SW-SM), ~70% widely graded sand, ~20% widely graded gravel up to 1 in, ~10% non-plastic fines. Brown, lense of light gray and orange sand at 3 in.
	15	S4	14 to 14.2	2/9	15-17-16-11	Boulder at 16.5 to 18.5 feet.		S4: SILTY SAND WITH GRAVEL (SM), ~40% widely graded sand, ~40% slightly plastic fines, ~20% mostly fine gravel. Light gray-brown, sand is coarser near top of sample. Dense, possibly till.
	20	S5	19 to 19.2	2/9	19-21-21-31			S5: WIDELY GRADED SAND WITH GRAVEL (SW), ~75% widely graded sand, ~20% widely graded gravel up to 1in, <5% fines. Gray, possibly till.

NOTES:

PROJECT NAME: Rockport Mill Pond Dam
CITY/STATE: Rockport, Massachusetts
GEI PROJECT NUMBER: 07104-1

GEI Consultants

GEI WOBURN STD 1-LOCATION-LAYER NAME 071041 ROCKPORT DAM.GPJ GEI DATA TEMPLATE.GDT 8/3/09

LOCATION:
REFERENCE ELEV. (ft): NM
VERTICAL DATUM:
DATE START/END: 5/8/2009 - 5/8/2009
DRILLING COMPANY: Geologic, Inc.

BORING B2

Elev. (ft)	Depth (ft)	Sample Information: Sample No.	Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	PID Jar Headspace/ Drilling Remarks	Layer Name	Soil and Rock Description
	25	S6	24 to 24.2	2/14	29-40-43-30			S6: SILTY SAND WITH GRAVEL (SM), ~50% widely graded sand, ~30% slightly plastic fines, ~20% widely graded gravel up to 3/4 in. Brown, dense, possibly till.
	30	S7	29 to 29.2	2/17	41-43-39-56			S7: Similar to S6.
								Bottom of boring at depth 31 ft. Borehole backfilled with bentonite chips.
	35							
	40							
	45							
	50							
	55							

NOTES:

PROJECT NAME: Rockport Mill Pond Dam
CITY/STATE: Rockport, Massachusetts
GEI PROJECT NUMBER: 07104-1

GEI Consultants

GEI WOBURN STD 1-LOCATION-LAYER NAME 071041 ROCKPORT DAM.GPJ GEI DATA TEMPLATE.GDT 8/3/09

Appendix E

Technical Memo 2 – Hydrology and Hydraulic Analyses

Memo Title: Technical Memo 2 – Hydrology and Hydraulic Analyses

Project Title: Mill Pond Dam – Final Design Report

Project Location: Rockport, Massachusetts

Project #: 07104-1

By: _____ **Date:** _____

Checked by: _____ **Date:** _____

TECHNICAL MEMORANDUM 2

Design Storm Hydrologic and Hydraulic Analysis
Mill Pond Dam
Rockport, MA
GEI Project #07104-1

1.0 Design Storm Hydrologic and Hydraulic Analysis

GEI Consultants performed the hydrologic and hydraulic (H&H) analysis to estimate the design storm for the existing and proposed Mill Pond Dam. The analysis was performed by a registered professional engineer experienced with the inspection, construction, and design of dams.

2.0 Design Criteria

We performed this study to determine the design storm for the rehabilitation of Mill Pond Dam in accordance with the Department of Conservation and Recreation (DCR) dam safety requirements and Massachusetts General Law (MGL) 253, §§ 44-50 and 302 CMR 10.00-10.16.

The Town of Rockport requested that the Mill Pond Dam normal pool elevation be maintained near El. 27.3 ft. This H&H analysis is based on the National Geodetic Vertical Datum 1929 (NGVD 29) elevation datum.

2.1 Size and Hazard Potential Classification

The existing dam is approximately 13.7 ft high at the maximum section. At maximum pool, we estimate the pond impounded by the Mill Pond Dam has a storage volume of 10 acre-ft with a surface area of 3.4 acres. Based on the height and storage the dam is classified as a "small" dam in accordance with the 302 CMR 10.06. At normal pool, we estimate that Mill Pond has a storage volume of 3 acre-ft and a surface area of 1.3 acres.

Based on our visual assessment of the potential downstream impacts, the dam height and the storage capacity, we estimated the hazard potential classification to be "significant" (Class II). According to the DCR definitions, the failure of a significant hazard classification dam "may cause the loss of life and damage home(s), industrial or commercial facilities, secondary highway(s) or railroad(s) or cause interception of use of service of relatively important facilities."

In accordance with the 302 CMR, the spillway design flood (SDF) for an existing, small, significant hazard potential dam is the 100 year storm.

3.0 Hydrology and Hydraulics Analyses

A narrative of the methodology and assumptions used for the H&H modeling and the analysis results are provided below. Supporting information used in the H&H analysis for the design storm is provided in Attachments 1 through 8.

400 Unicorn Park Drive
Woburn, MA 01801
781.721.4000 fax 781. 721.4073

3.1 Previous Studies

There are no known hydrologic and hydraulic studies or design reports available for the Mill Pond Dam. Flooding history of the dam is unknown with the exception of the storms occurring in March 2001 and May 2006. Based on precipitation data recorded in Beverly, Massachusetts (about 15 miles to the South), 4.3 inches of rain fell on March 22, 2001; 4.32 inches of rain fell on May 13, 2006; and 4.95 inches of rain fell on May 14, 2006.

3.2 Watershed Description

GEI delineated the drainage area upstream of the dam using a digital 7.5-minute USGS topographic map in AutoCAD. The measured area was estimated to be approximately 481 acres (0.75 square miles). The two highest points in the watershed include Pool Hill, west of the dam, and Great Hill near Route 127, located southwest of the dam. The watershed consists of moderately steep hills and knolls with small wetland areas. A figure depicting the delineated watershed is provided in Attachment 1.

The dam is located on Mill Brook and generally flows in an easterly direction to the Atlantic Ocean, which is located about 550 feet downstream of the dam. The watershed and brook are ungaged. There are no known instruments in the watershed that measure precipitation, flow, or stage.

There is one small pond (Loop Pond) located about 0.7 miles upstream of the Mill Pond Dam. The physical characteristics and the hydrologic and hydraulic properties of Loop Pond are unknown. It is our opinion that Loop Pond would have a negligible effect on the 100-year flood at Mill Pond Dam.

3.3 Cover Types and Soils Data

The drainage area generally consists of wooded areas on the northwestern side of the watershed and includes Briar Swamp. The drainage area near the dam and south of Mill Brook is much more populated with residential properties and some commercial areas. Surficial soils in the watershed area generally consist of about 57 percent fine sandy loam; 13 percent muck; 13 percent impervious area and urban land; 10 percent bedrock outcrops; and 7 percent loam and loamy sand. The hydrologic soil groups (HSG) for the watershed are summarized in Table 1. Attachment 2 contains the hydrologic soil groups, cover type areas, and curve number calculations.

Table 1. Mill Pond Dam Watershed Hydrologic Soil Groups

Hydrologic Soil Group	Infiltration Rate	Area, (acres)	Percent of Total Area
A	High	26.3	5%
B	Moderate	170.1	35%
C	Slow	135.3	28%
D	Very slow	91.9	19%
Impervious areas (water, urban areas, paved, etc)	Impervious	58.1	12%
	Total*	**481.7**	**100%**

*Watershed delineated on soils map is approximate and differs slightly from measured watershed area; (481.1 versus 481.7 acres).

3.4 Unit Hydrographs

The unit hydrograph for the drainage area was estimated using the SCS unit hydrograph procedure (also known as the TR-20 runoff method). The SCS Curve Number (CN) for the watershed was estimated based on the aerial survey image, hydrologic soil groups (HSG), and site observations. We calculated weighted average CN of 74 for the watershed and used an antecedent moisture condition (AMC) of 2.

3.5 Time-of-Concentration

The time-of-concentration (Tc) for the dam was based on the longest flow path in the watershed which consisted of several segments including sheet flow, shallow concentrated flow and several reaches of channel flow. Based on these elevations obtained from the 7.5-minute USGS topographic map and the topographic survey of the Mill Pond Dam area, the travel time for each component was determined using the calculations and procedures in TR-55 (Natural Resources Conservation Service (NRCS) Department of Agriculture, Urban Hydrology for Small Watersheds, Technical Release 55). We calculated an estimated Tc for the watershed to be 65 minutes. Calculations are provided in Attachment 3.

3.6 Precipitation

The precipitation used to develop the 100-year storm is based on TR-55. The rainfall distribution maps provided in TR-55 are based on the National Weather Service Technical Paper 40 (TP-40). Based on the location of the watershed on the TP-40 maps (provided in Attachment 4), we selected a “Type III” rainfall distribution and a 100-year, 24-hour rainfall of 6.6 inches for the design storm.

3.7 Rainfall-Runoff Model

The H&H analyses were performed using the U.S. Army Corps of Engineers Hydraulic Engineering Center, Hydraulic Modeling System (HEC-HMS) model, Version 3.3 based on the procedures described in Section 4, Hydrology of the Soil Conservation Service (SCS) National Engineering Handbook.

3.8 Routing

The rainfall-runoff was routed from the watershed to the reservoir in the HEC-HMS model. The reservoir routing for Mill Pond was performed within the model using elevation and surface area values estimated from the 7.5-minute USGS topographic map. Our calculations are provided in Attachment 5.

The discharge rating curve for the existing dam configuration was calculated in the Federal Highway Administration software program HY-8, Version 7.0 and input into the HEC-HMS model as a user-defined elevation-discharge rating curve. We calculated the existing discharge rating curve using HY-8 because of the non-uniform crest and parapet walls. The HY-8 output is provided in Attachment 6.

The discharge rating curve calculations for the proposed conditions were calculated in a spreadsheet to account for the multiple openings, dimensions, and elevations associated with the proposed bench and handrails. We calculated the discharge capacity for the upstream and downstream side of the dam to evaluate the controlling discharge capacity for the dam. The proposed discharge rating curve calculations are provided in Attachment 7. We entered the final elevation versus discharge values for the proposed dam into HEC-HMS.

The starting elevation of the pond (initial elevation) for the 100-year design storm calculations was assumed to be at El. 27.3 feet with an assumed base flow of 1 cfs.

3.9 Antecedent Conditions

We assumed the starting elevation of the reservoir was at the normal maximum pool of El. 27.3 ft. The base flow was assumed to be 1 cfs. The initial abstraction was assumed to be 0.0 inches to simulate saturated soil conditions.

3.10 Model Calibration

A detailed calibration of the rainfall-runoff model is not feasible because the watershed is ungaged and historical information, such as precipitation, peak flow, and peak pond elevation, are not recorded at Mill Pond.

The precipitation intensity and duration that caused the 2001 and 2006 floods is unknown. Gages within several miles of the watershed recorded 4 to 5 inches for both events. The pond overtopped the dam crest during both events so the peak elevation was at least El. 29.5 ft or greater.

We modeled a 5-year storm (3.9 inches of precipitation) in the watershed that resulted in a peak pond elevation of 30.8 feet.

We concluded that the model simulates the rainfall-runoff for the watershed with acceptable and reasonable results based on the results of our 5-year storm analysis. The model computed a peak pond elevation of 30.8 ft from 3.9 inches of precipitation.

4.0 Existing Dam Features

Twin Box Culverts: The existing dam contains two box culverts with an invert elevation of 25.65 to 26.0 ft. Each opening is approximately 2.5 ft wide and 2.5 ft high. It is our understanding that the Town of Rockport maintained the normal pool elevation by placing sandbags or boards to maintain the desired normal pool elevation.

Dam Crest: The current crest elevation is irregular, varies from a low point of approximately 29.5 ft, and ties into the existing natural surrounding contours above El. 32 ft.

Guard Rails/Parapet Walls: The existing dam contains two "parapet walls" along the upstream and downstream face of the dam. The top of the parapet walls are approximately 1.5 ft above the crest of the dam.

5.0 Proposed Dam Features

Twin Box Culverts: The proposed dam will contain two box culverts with an invert elevation of 25.6 ft. The width and height of each box culverts will be about 2.5 ft wide and 2.5 ft high. The top of the box culvert opening will be at El. 28.34 ft, 0.66 ft below the crest elevation. It is our understanding that the Town of Rockport will control the pond elevation by installing stoplogs to maintain the desired normal pool elevation at or near El. 27.3 ft.

Dam Crest: The crest of the dam will be re-established to El. 29.0 ft, about 0.5 ft lower than the existing crest elevation not including the parapet/guard rails. At the Town's request, handrails and benches will replace the parapet walls/guard rails on the upstream and downstream sides of the crest. Our discharge calculations accounted for the flow under, through, and over the proposed handrails and the bench using standard weir and orifice equations. We assumed that the Town will remove stoplogs to allow the box culverts to flow freely before large storm events.

Handrail and Bench System: A system of granite benches will replace the upstream parapet wall and a stone masonry and steel handrail system will replace the downstream parapet wall and provide fall protection (see Drawings). The bench and handrail systems will provide open areas to allow water and debris to safely pass over the dam during significant storm events.

6.0 Design Storm Results and Conclusions

We evaluated three scenarios for the 100-year design storm:

1. Existing Conditions, boards or sandbags in place
2. Proposed Conditions, stoplogs in place
3. Proposed Conditions, stoplogs removed

We also evaluated the 2-, 5-, 10-, 25-, and 50-year storm events for the Proposed Conditions, stoplogs removed scenario.

6.1 100-yr Flood Design Storm

The 100 year storm results for Mill Pond Dam are summarized below in Table 2. The results indicate that under the existing conditions, the peak stage for the 100 year storm is El. 31.5. This would overtop the dam by 2.0 feet for about 6 hours assuming the stoplogs are left in place during the storm.

The proposed dam crest (with the handrails and benches) would be overtopped by approximately 2.0 feet for about 13 hours. The maximum reservoir elevation is 31.0 ft from the 100 year storm assuming the stoplogs are left in place during the storm.

Removing the stoplogs prior to the storm, but maintaining the normal pool at 27.3 ft would reduce the peak pond elevation and maximum overtopping depth and duration of overtopping. Removing the stoplogs on the proposed dam would only decrease the peak stage by .1 ft with a maximum overtopping depth of 1.9 ft. The duration of overtopping is decreased by almost 8 hours be removing the stoplogs.

The HEC-HMS output and hydrographs for the 100-year flood design storm at Mill Pond Dam are provided in Attachment 8.

Table 2. Mill Pond Dam – 100 Year Design Storm Results

Case	Inflow, cfs	Outflow, cfs	Peak Stage, ft	Maximum Overtopping Depth, ft	Duration of Overtopping, hrs
Existing, stoplogs in place	670	667	31.5	2.0	6
Proposed, stoplogs in place	670	669	31.0	2.0	13
Proposed, stoplogs removed	670	665	30.9	1.9	5.3

6.2 Lower Return Period Storms

We evaluated the 2-, 5-, 10-, 25-, and 50-year storms for the proposed Mill Pond Dam with the boards/sand bags in place and removed. The results provided in Table 3 indicate that the dam will be overtopped for each event regardless of the boards/sand bags. Removing the boards/sand bags lowers the peak pond elevations and maximum overtopping depths by 0.1 to 0.3 ft. Removing the boards or sand bags significantly reduces the duration of overtopping by a factor of 2. The results for the boards/sand bags removed are presented below in Table 3.

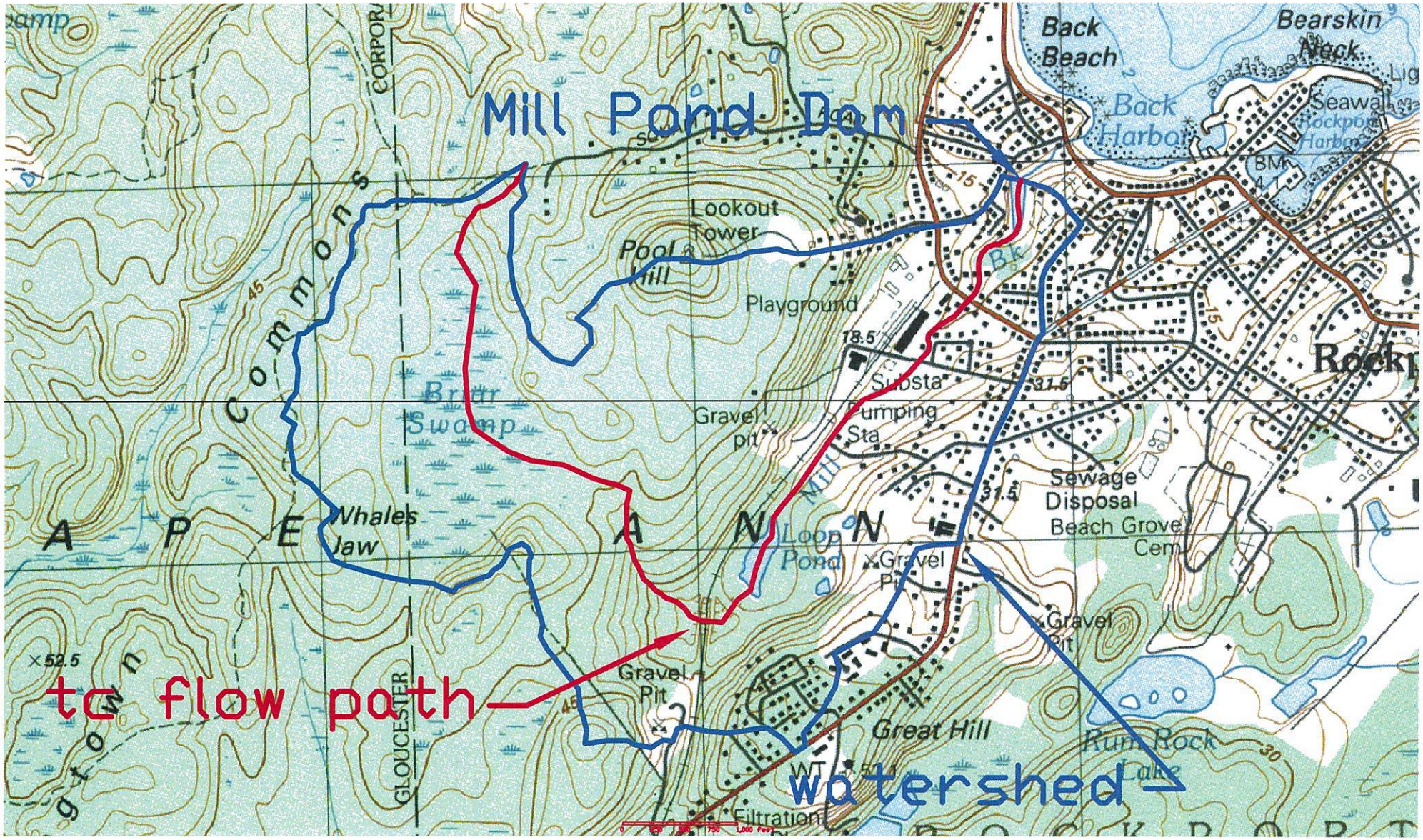
Table 3. Proposed Mill Pond Dam, Boards/sand bags removed: 2, 5, 10, 25, and 50-Year Storm Results

Storm Event, yr	Precipitation, inches	Peak Inflow, Outflow, cfs	Peak Res. Elevation, ft	Duration of Overtopping, hours	Max Overtop Depth, ft
50	5.9	574	30.7	5.0	1.7
25	5.5	520	30.6	4.5	1.6
10	4.6	405	30.3	3.5	1.3
5	3.9	316	30.1	3.0	1.1
2	3.2	233	29.8	2.0	0.8

M:\DATA_DESIGN\071041 Rockport Mill Pond Dam\Ch253\TM2 - H&H\TM2-H&H_2010.02.18.docx

Attachment 1

Watershed Map

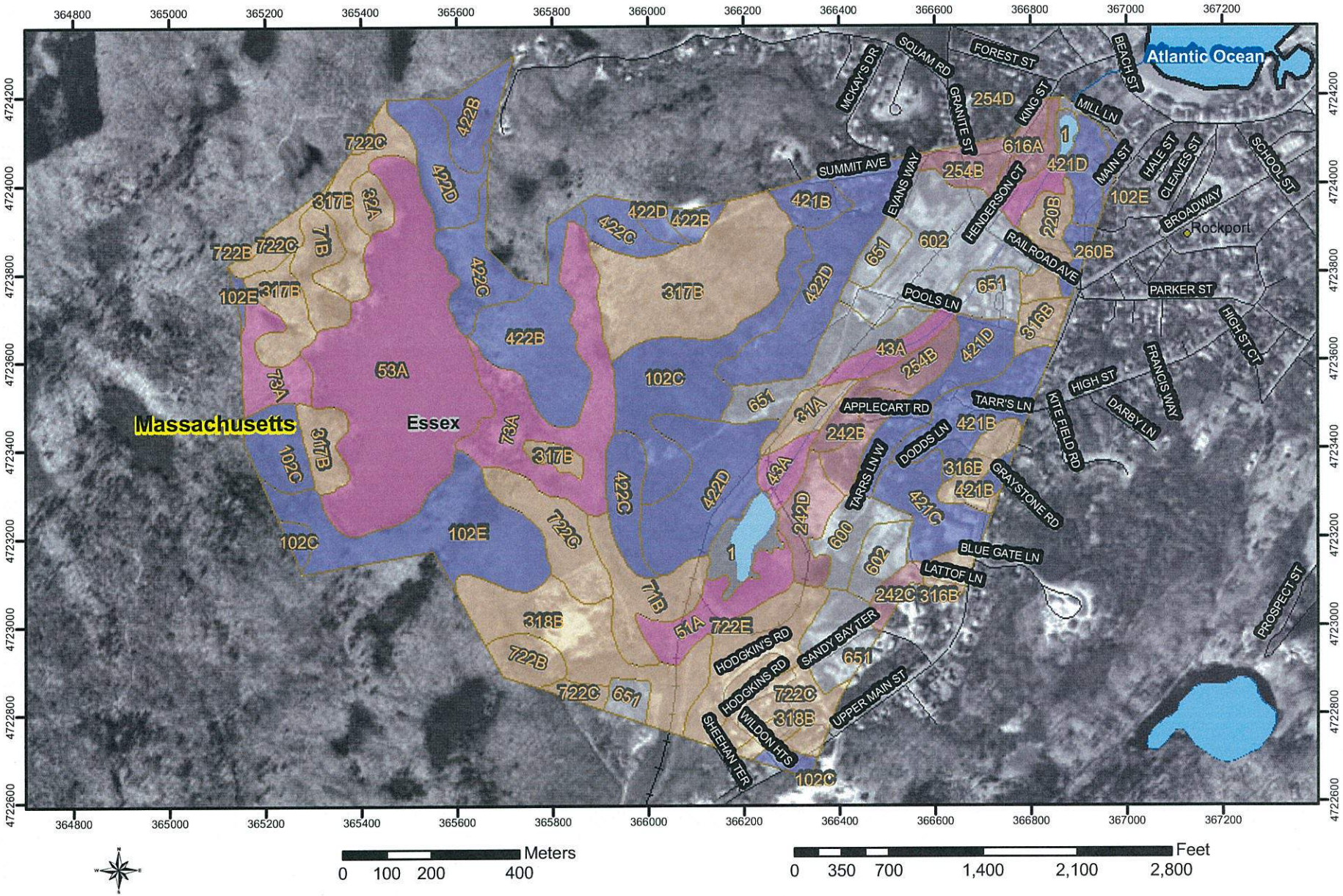

Mill Pond Dam
tc flow path
watershed
Back Beach
Bearskin Neck
Back Harbor
Seawall
Lookout Tower
Pool Hill
Playground
Brutr Swamp
Whales Jaw
Gravel pit
Substa
Pumping Sta
Sewage Disposal
Beach Grove Cem
Loon Pond
Gravel Pit
Great Hill
Run Rock Lake
Filtration
GLOUCESTER
CORPORA
A P E A N N
Commons
Rockp

Attachment 2

Hydrologic Soil Groups, Cover Types, and CN Calculations

HYDROLOGIC SOIL GROUP RATING FOR ESSEX COUNTY, MASSACHUSETTS, SOUTHERN PART

Rockport Mill Pond Dam



USDA Natural Resources Conservation Service

Web Soil Survey 1.1
National Cooperative Soil Survey

4/18/2007
Page 1 of 5

Rockport Mill Pond Dam

MAP LEGEND

Hydrologic Soil Group

{Dominant Condition, <}

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available
- Soil Map Units
- Cities
- Detailed Counties
- Detailed States
- Interstate Highways
- Roads
- Rails
- Water
- Hydrography
- Oceans

MAP INFORMATION

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov

Coordinate System: UTM Zone 19

Soil Survey Area: Essex County, Massachusetts, Southern Part
Spatial Version of Data: 2
Soil Map Compilation Scale: 1:15840

Map comprised of aerial images photographed on these dates:
3/29/1995

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Tables - Hydrologic Soil Group

Summary by Map Unit - Essex County, Massachusetts, Southern Part

Soil Survey Area Map Unit Symbol	Map Unit Name	Rating	Total Acres in AOI	Percent of AOI
1	Water	Null	6.4	1.3
31A	Walpole fine sandy loam, 0 to 3 percent slopes	C	3.4	0.7
32A	Wareham loamy sand, 0 to 3 percent slopes	C	2.5	0.5
43A	Scarboro mucky loamy fine sand, 0 to 1 percent slopes	D	7.1	1.5
51A	Swansea muck, 0 to 1 percent slopes	D	8.1	1.7
53A	Freetown muck, ponded, 0 to 1 percent slopes	D	47.9	9.9
71B	Ridgebury fine sandy loam, 3 to 8 percent slopes, extremely stony	C	14.4	3.0
73A	Whitman loam, 0 to 3 percent slopes, extremely stony	D	26.3	5.4
102C	Chatfield-Hollis-Rock outcrop complex, 3 to 15 percent slopes	B	27.3	5.7
102E	Chatfield-Hollis-Rock outcrop complex, 15 to 35 percent slopes	B	22.4	4.6
220B	Boxford silt loam, 3 to 8 percent slopes	C	5.2	1.1
242B	Hinckley gravelly fine sandy loam, 3 to 8 percent slopes	A	4.0	0.8
242C	Hinckley gravelly fine sandy loam, 8 to 15 percent slopes	A	2.1	0.4
242D	Hinckley gravelly fine sandy loam, 15 to 25 percent slopes	A	5.9	1.2
254B	Merrimac fine sandy loam, 3 to 8 percent slopes	A	11.8	2.4
254D	Merrimac fine sandy loam, 15 to 25 percent slopes	A	2.5	0.5
260B	Sudbury fine sandy loam, 3 to 8 percent slopes	B	1.8	0.4
316B	Scituate fine sandy loam, 3 to 8 percent slopes, very stony	C	8.8	1.8
317B	Scituate fine sandy loam, 3 to 8 percent slopes, extremely stony	C	43.0	8.9

Summary by Map Unit - Essex County, Massachusetts, Southern Part

Soil Survey Area Map Unit Symbol	Map Unit Name	Rating	Total Acres in AOI	Percent of AOI
318B	Scituate fine sandy loam, 3 to 8 percent slopes, extremely bouldery	C	19.7	4.1
421B	Canton fine sandy loam, 3 to 8 percent slopes, very stony	B	15.3	3.2
421C	Canton fine sandy loam, 8 to 15 percent slopes, very stony	B	8.5	1.8
421D	Canton fine sandy loam, 15 to 25 percent slopes, very stony	B	13.6	2.8
422B	Canton fine sandy loam, 3 to 8 percent slopes, extremely stony	B	22.1	4.6
422C	Canton fine sandy loam, 8 to 15 percent slopes, extremely stony	B	17.1	3.5
422D	Canton fine sandy loam, 15 to 25 percent slopes, extremely stony	B	42.0	8.7
600	Pits, gravel	Null	5.1	1.1
602	Urban land	Null	22.7	4.7
616A	Fluvaquents, frequently flooded, 0 to 3 percent slopes	D	2.5	0.5
651	Udorthents, smoothed	Null	23.9	5.0
722B	Annisquam fine sandy loam, 3 to 8 percent slopes, extremely bouldery	C	4.3	0.9
722C	Annisquam fine sandy loam, 8 to 15 percent slopes, extremely bouldery	C	21.6	4.5
722E	Annisquam fine sandy loam, 15 to 35 percent slopes, extremely bouldery	C	12.4	2.6

Description - Hydrologic Soil Group

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Parameter Summary - Hydrologic Soil Group

Aggregation Method: Dominant Condition

Component Percent Cutoff:

Tie-break Rule: Lower

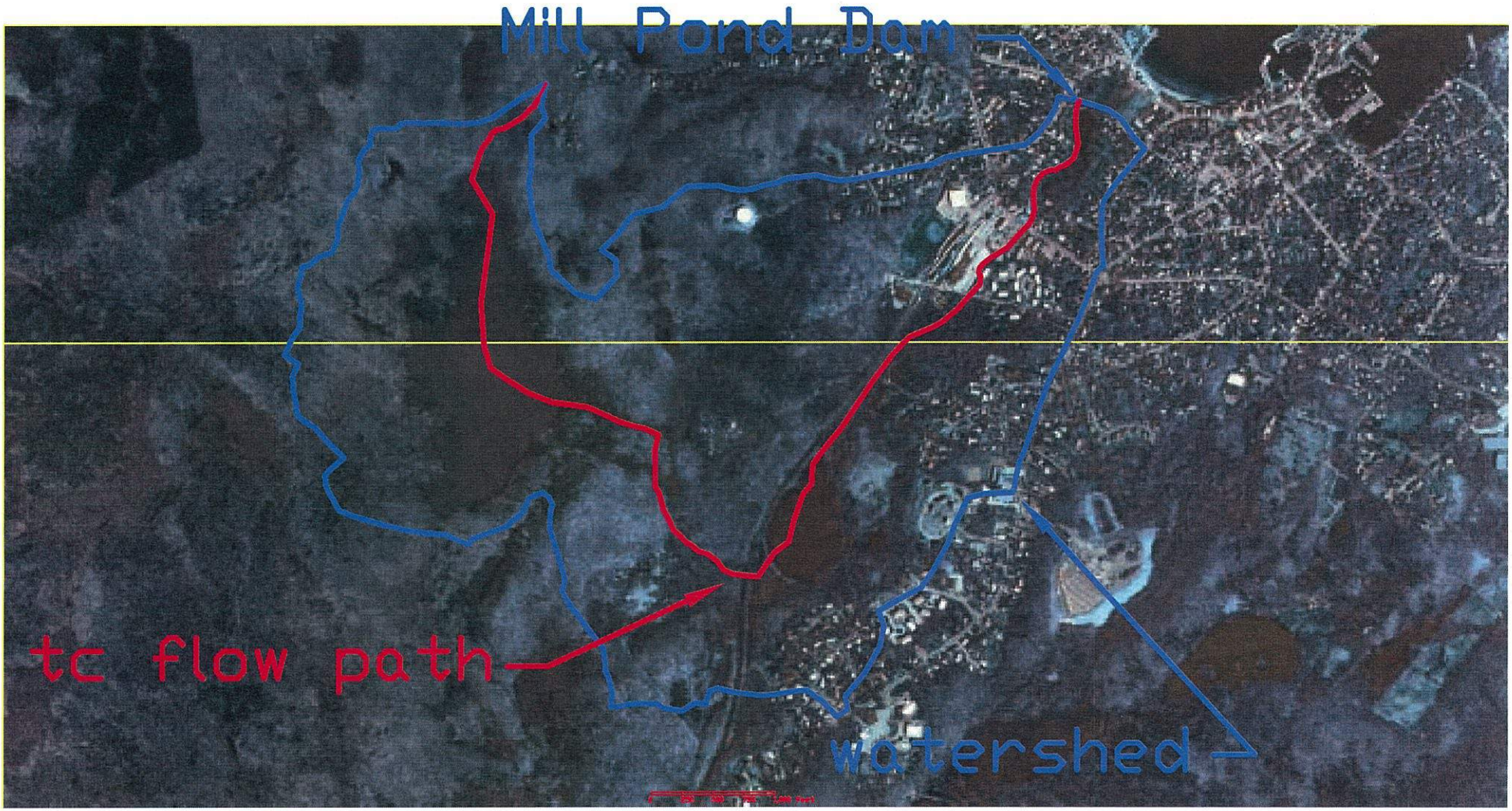
Areas measured in AutoCAD

	sq. meters	sq ft	sq mi	acres
Total	1947768.6248	20954874	0.752	481.058
Briar Swamp	589959.5519	6347020.8	0.228	145.708
Loop Pond	667556.2563	7181837.2	0.258	164.872
Mill Pond	690252.8166	7426015.9	0.266	170.478
Total check	1947768.625	20954874	0.752	481.058

	Cover types	sq. meters	sq ft	sq mi	acres	HSG	Assigned Curve Number	Wt'd CN	acres x CN
1S	**Briar Swamp**	589959.5519	6347020.8	0.228	145.708				
	wetland	186020.3241	2001281.1	0.072	45.943	D	86		3951.1
	wooded	403939.2278	4345739.8	0.156	99.764	B/C	70	**75**	6983.5
				check	**145.708**				
2S	**Loop Pond**	667556.2563	7181837.2	0.258	164.872				
	1/4 acre lots	89329.5749	961043.3	0.034	22.063	A / imp.	83		1831.2
	pond	40695.9639	437823.46	0.016	10.051	pond/urban	98		985.0
	wooded	537530.7175	5782970.5	0.207	132.759	B/C/D	70	**73**	9293.1
				check	**164.872**				
3S	**Mill Pond**	690252.8166	7426015.9	0.266	170.478				
	Wooded	376425.9707	4049741.2	0.145	92.969	B/C	70		6507.8
	1/4 acre lots	221124.2814	2378943.5	0.085	54.613	A/B/C	75		4096.0
	commercial	48440.9338	521146.94	0.019	11.964	urban	94		1124.6
	grass field	14541.04	156438.32	0.006	3.591	C	74		265.8
	wooded	24466.9322	263225.04	0.009	6.043	B/C/D	70		423.0
	pond	5253.6585	56520.96	0.002	1.298	urban	94	**74**	122.0
				check	**170.478**			**Wt'd CN**	**74.0**
						481.058			

Attachment 3

Time of Concentration (T_c) Calculations


Mill Pond Dam
tc flow path
watershed

Subcatchment 11S: Mill Pond Drainage Area

Runoff = 712.06 cfs @ 15.91 hrs, Volume= 146.499 af, Depth> 3.65"

Runoff by SCS TR-20 method, UH=SCS, Time Span= 5.00-30.00 hrs, dt= 0.02 hrs
Type III 24-hr 30.00 hrs 100-yr Rainfall=6.60"

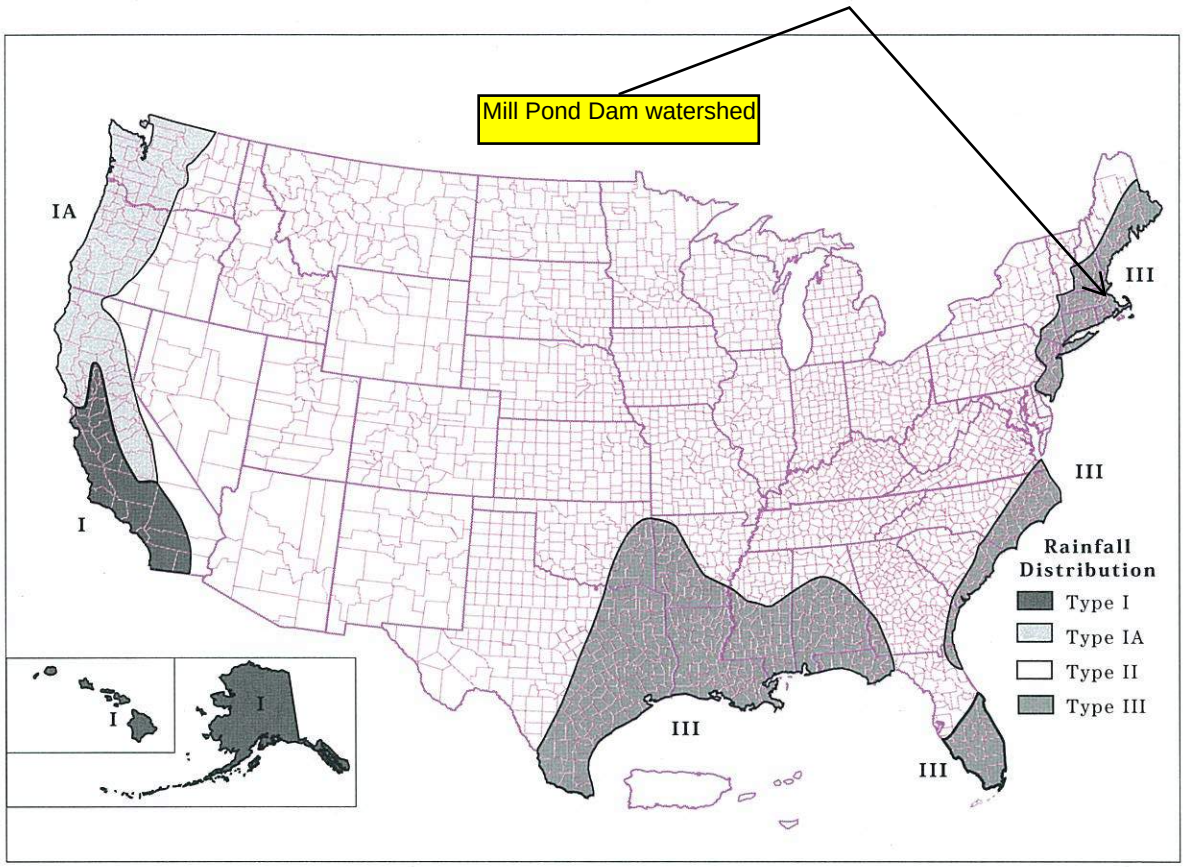
Area (ac)	CN	Description
481.058	74	Wt'd Average
481.058		Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
32.1	150	0.0660	0.08		**Sheet Flow, SF 1-1** Woods: Dense underbrush n= 0.800 P2= 3.20"
11.1	692	0.0430	1.04		**Shallow Concentrated Flow, SCF 1-2** Woodland Kv= 5.0 fps
7.7	2,626		5.67		**Lake or Reservoir, Swamp 1-3** Mean Depth= 1.00'
5.7	3,295	0.0231	9.71	323.54	**Parabolic Channel, Loop Pond Reach** W=10.00' D=5.00' Area=33.3 sf Perim=14.8' n= 0.040
8.8	3,882	0.0134	7.39	246.42	**Parabolic Channel, Mill Brook Reach** W=10.00' D=5.00' Area=33.3 sf Perim=14.8' n= 0.040
65.4	10,645	Total			

Attachment 4

NRCS (SCS) 24-Hour Precipitation Maps

Figure B-2 Approximate geographic boundaries for NRCS (SCS) rainfall distributions



Rainfall data sources

This section lists the most current 24-hour rainfall data published by the National Weather Service (NWS) for various parts of the country. Because NWS Technical Paper 40 (TP-40) is out of print, the 24-hour rainfall maps for areas east of the 105th meridian are included here as figures B-3 through B-8. For the area generally west of the 105th meridian, TP-40 has been superseded by NOAA Atlas 2, the Precipitation-Frequency Atlas of the Western United States, published by the National Ocean and Atmospheric Administration.

East of 105th meridian

Hershfield, D.M. 1961. Rainfall frequency atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years. U.S. Dept. Commerce, Weather Bur. Tech. Pap. No. 40. Washington, DC. 155 p.

West of 105th meridian

Miller, J.F., R.H. Frederick, and R.J. Tracey. 1973. Precipitation-frequency atlas of the Western United States. Vol. I Montana; Vol. II, Wyoming; Vol III, Colorado; Vol. IV, New Mexico; Vol V, Idaho; Vol. VI, Utah; Vol. VII, Nevada; Vol. VIII, Arizona; Vol. IX, Washington; Vol. X, Oregon; Vol. XI, California. U.S. Dept. of Commerce, National Weather Service, NOAA Atlas 2. Silver Spring, MD.

Alaska

Miller, John F. 1963. Probable maximum precipitation and rainfall-frequency data for Alaska for areas to 400 square miles, durations to 24 hours and return periods from 1 to 100 years. U.S. Dept. of Commerce, Weather Bur. Tech. Pap. No. 47. Washington, DC. 69 p.

Hawaii

Weather Bureau. 1962. Rainfall-frequency atlas of the Hawaiian Islands for areas to 200 square miles, durations to 24 hours and return periods from 1 to 100 years. U.S. Dept. Commerce, Weather Bur. Tech. Pap. No. 43. Washington, DC. 60 p.

Puerto Rico and Virgin Islands

Weather Bureau. 1961. Generalized estimates of probable maximum precipitation and rainfall-frequency data for Puerto Rico and Virgin Islands for areas to 400 square miles, durations to 24 hours, and return periods from 1 to 100 years. U.S. Dept. Commerce, Weather Bur. Tech. Pap. No. 42. Washington, DC. 94 P.

Figure B-3 2-year, 24-hr rainfall

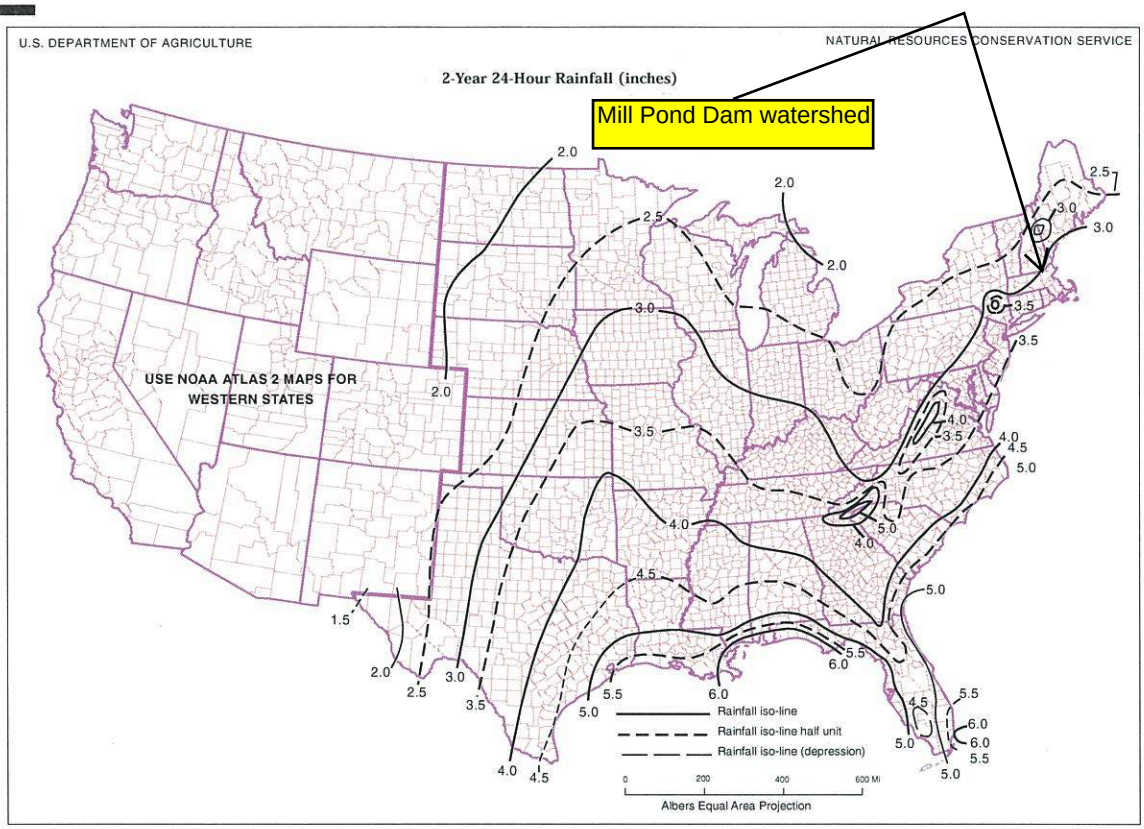


Figure B-4 5-year, 24-hour rainfall

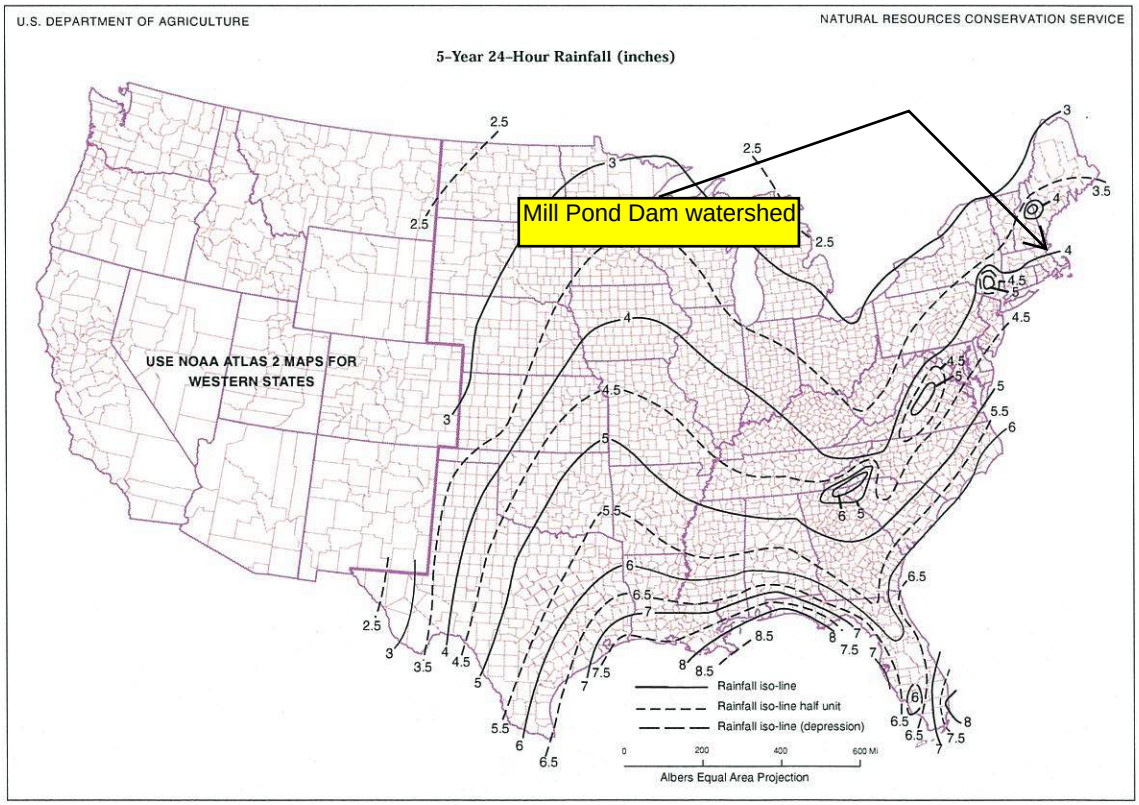


Figure B-5 10-year, 24-hour rainfall

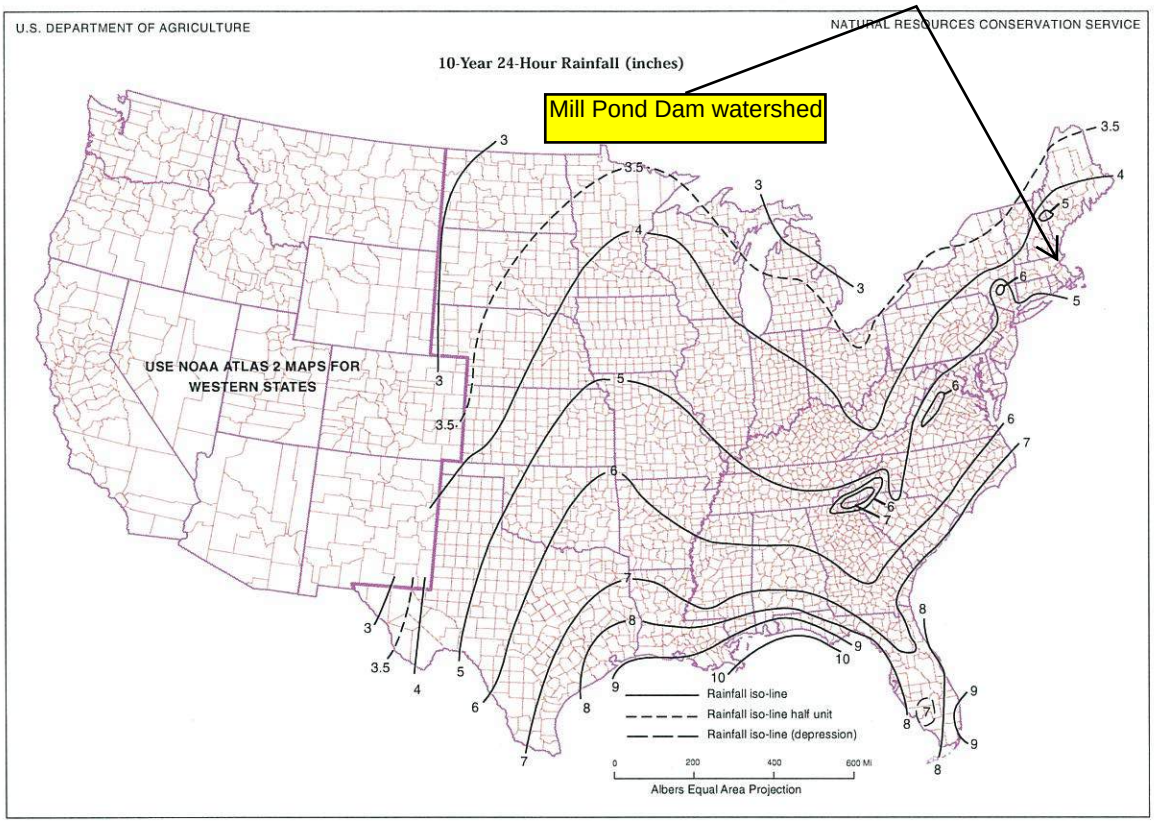


Figure B-6 25-year, 24-hour rainfall

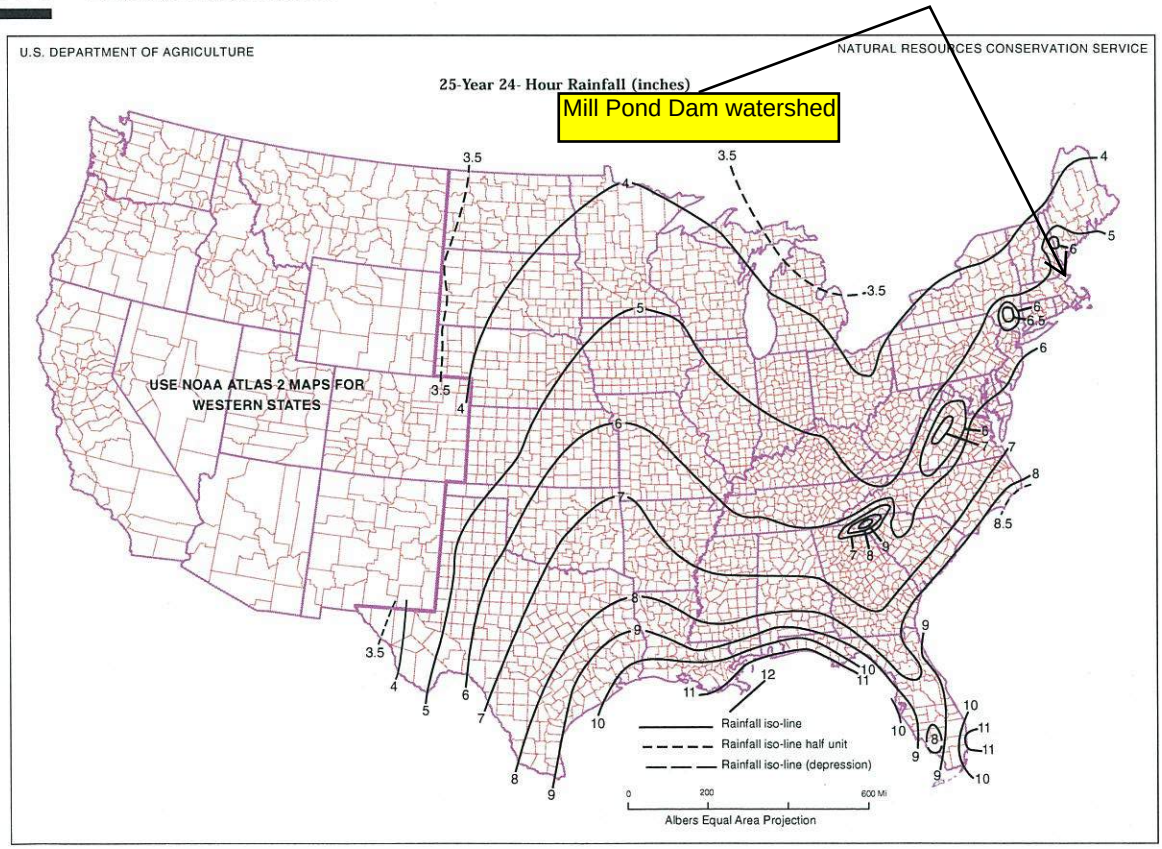


Figure B-7 50-year, 24-hour rainfall

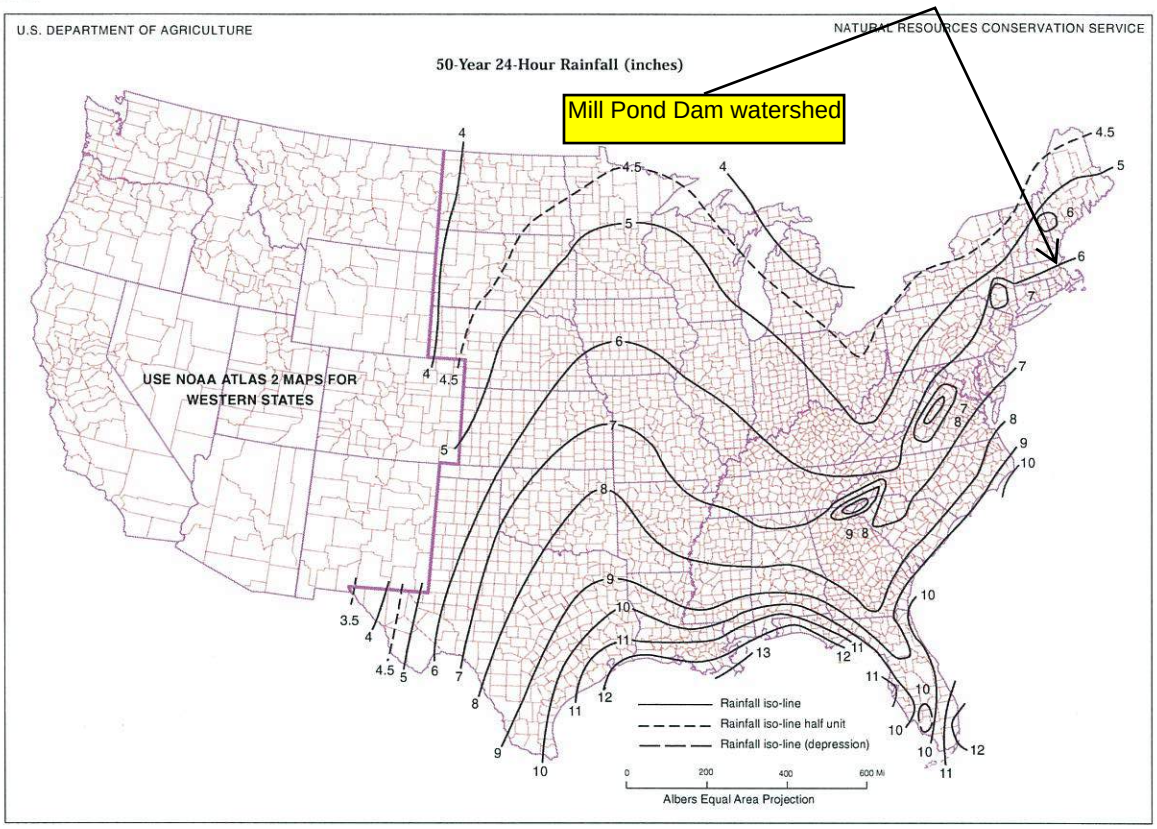
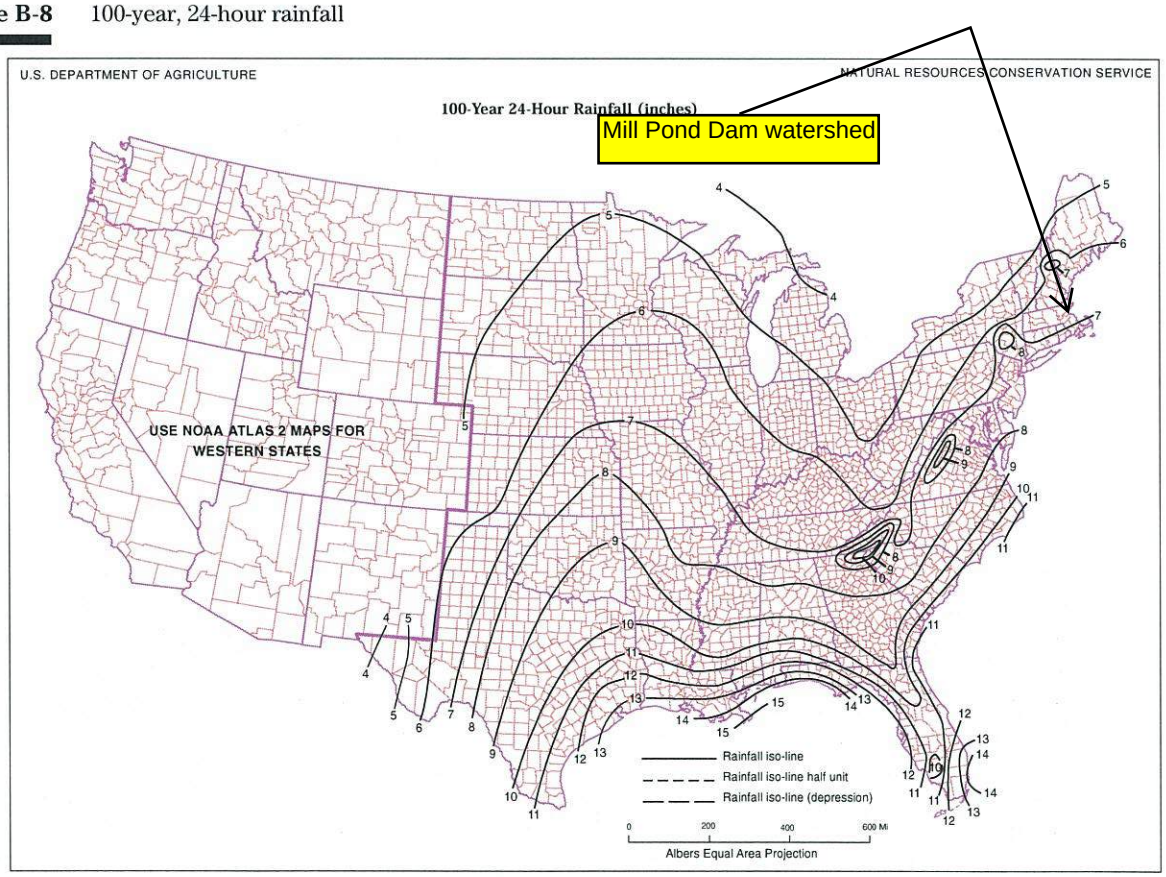


Figure B-8 100-year, 24-hour rainfall

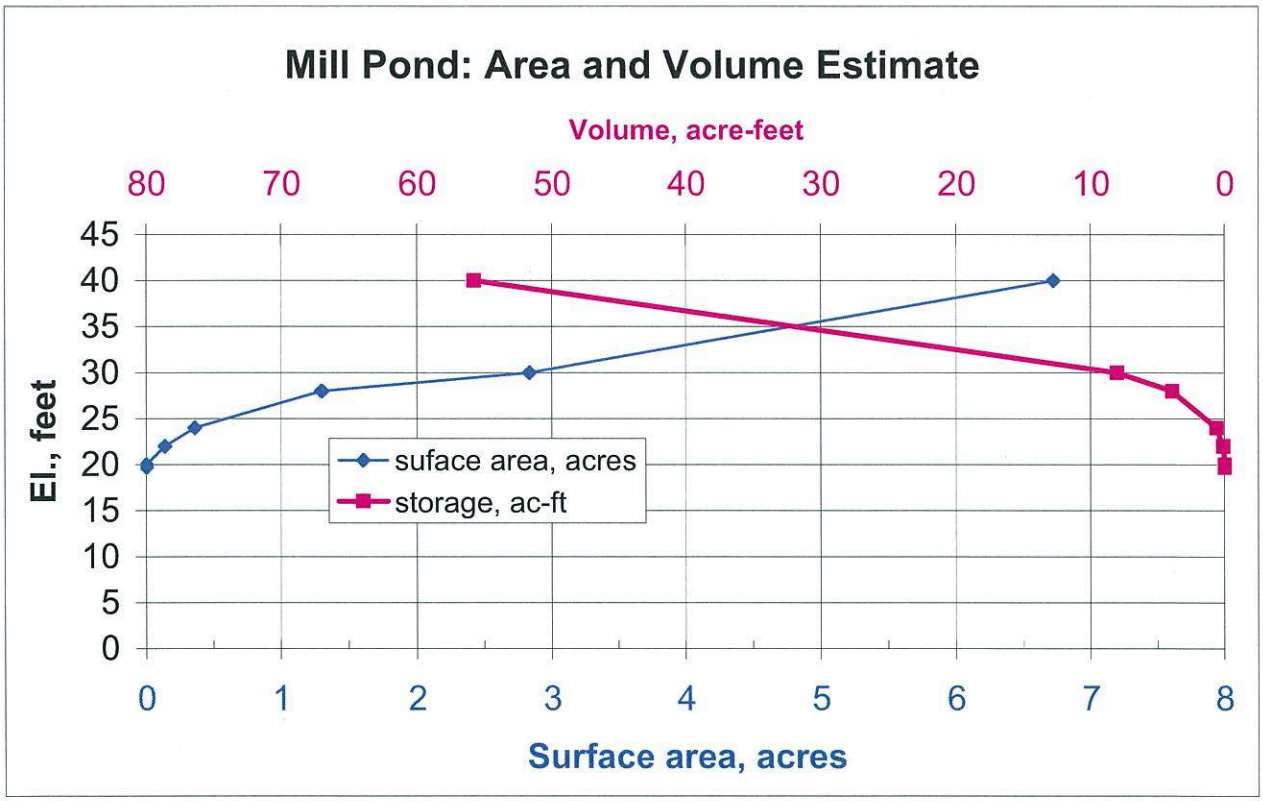


Attachment 5

Reservoir Storage Data

MILL POND SURFACE AREA and VOLUME ESTIMATE

Source	El., ft	sq meters	sq ft	ace area, a	sq feet	storage, ac-ft	Comments
Autocad	19.7		0	0.00	0	0	
Autocad	20		18	0.00	3	0	
Autocad	22		5980	0.14	6,000	0.14	*estimated*
Autocad	24		15543	0.36	27,525	0.63	*estimated*
USGS	28	5253.6585	56521	1.30	171,653	3.94	
USGS	30	11457.2468	123262	2.83	351,436	8.07	
USGS	40	27206	292693	6.72	2,431,261	55.81	



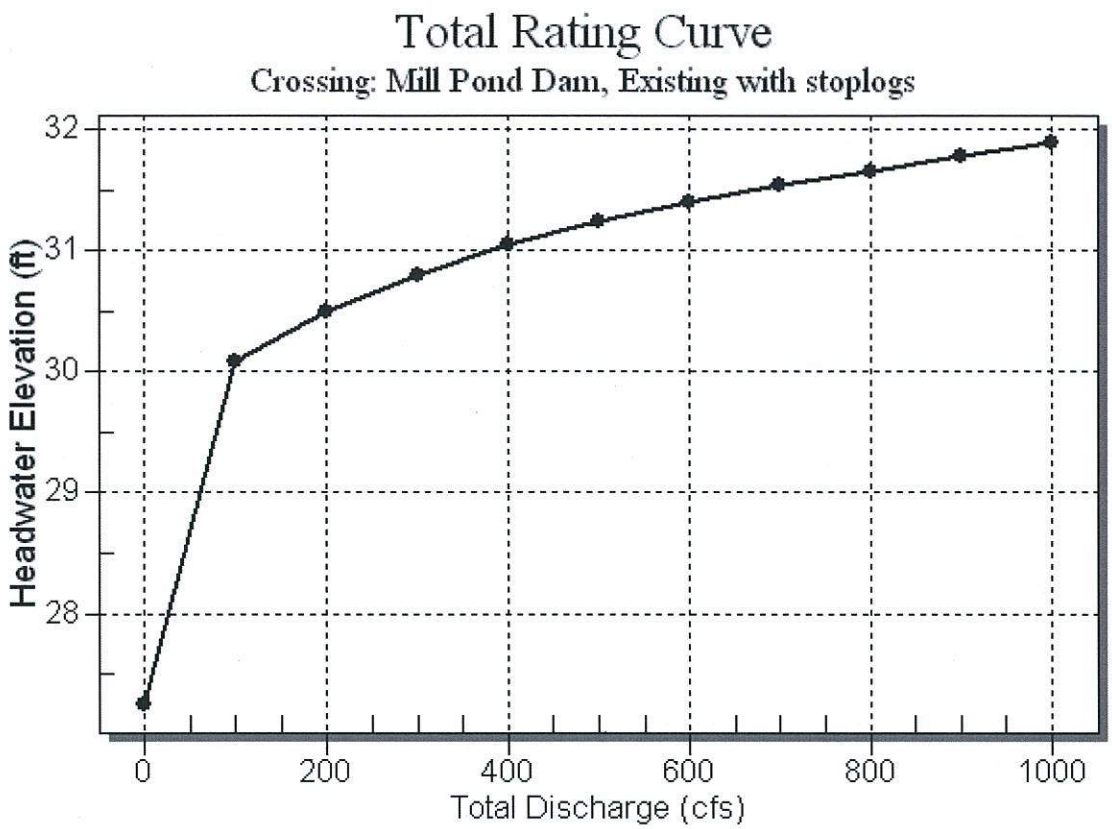
Attachment 6

Existing Discharge Rating Curve Calculations

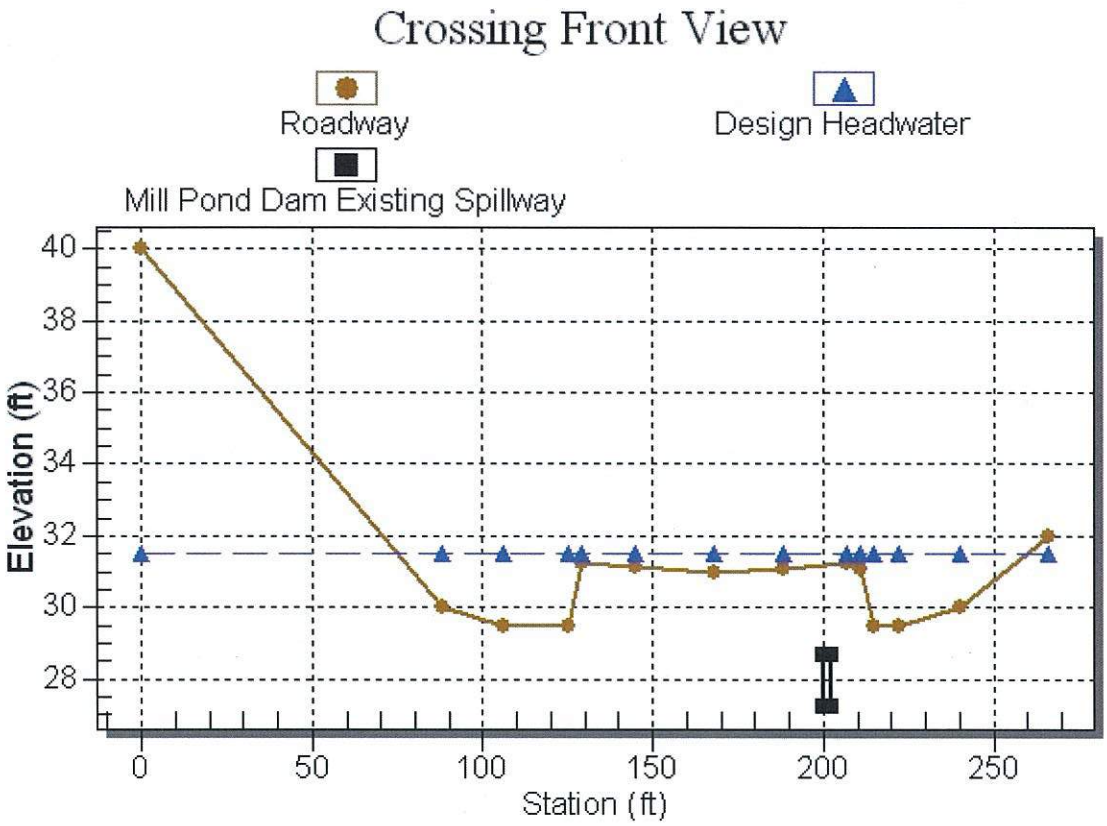
Table 1 - Summary of Culvert Flows at Crossing: Mill Pond Dam, Existing w/stoplogs

Headwater Elevation (ft)	Total Discharge (cfs)	Mill Pond Dam Existing Spillway Discharge (cfs)	Roadway Discharge (cfs)	Iterations
27.25	0.00	0.00	0.00	1
30.09	100.00	46.89	53.07	7
30.50	200.00	51.87	147.88	5
30.80	300.00	55.24	244.45	6
31.05	400.00	57.84	341.47	6
31.25	500.00	59.80	439.56	5
31.40	600.00	61.31	538.13	4
31.54	700.00	62.54	637.30	4
31.66	800.00	63.60	735.92	3
31.78	900.00	64.62	835.03	3
31.89	1000.00	65.54	934.43	3

Rating Curve Plot for Crossing: Mill Pond Dam, Existing with stoplogs



Crossing Front View (Roadway Profile): Mill Pond Dam, Existing with stoplogs



Culvert Notes: Mill Pond Dam Existing Spillway

Table 2 - Culvert Summary Table: Mill Pond Dam Existing Spillway

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	27.25	0.000	0.000	0-NF	0.000	0.000	0.000	0.000	0.000	0.000
100.00	46.89	30.09	2.841	1.323	5-S2n	0.512	1.400	0.767	1.463	12.227	4.111
200.00	51.87	30.50	3.251	1.585	5-S2n	0.550	1.400	0.832	1.925	12.470	4.891
300.00	55.24	30.80	3.553	1.777	5-S2n	0.574	1.400	0.873	2.256	12.655	5.414
400.00	57.84	31.05	3.801	1.934	5-S2n	0.592	1.400	0.904	2.524	12.797	5.818
500.00	59.80	31.25	3.996	2.057	5-S2n	0.605	1.400	0.926	2.754	12.917	6.147
600.00	61.31	31.40	4.150	2.154	5-S2n	0.616	1.400	0.942	2.955	13.016	6.437
700.00	62.54	31.54	4.285	2.236	5-S2n	0.624	1.400	0.955	3.137	13.097	6.686
800.00	63.60	31.66	4.409	2.308	5-S2n	0.632	1.400	0.966	3.302	13.169	6.919
900.00	64.62	31.78	4.529	2.377	5-S2n	0.639	1.400	0.976	3.455	13.242	7.126
1000.00	65.54	31.89	4.638	2.441	5-S2n	0.645	1.400	0.985	3.599	13.307	7.313

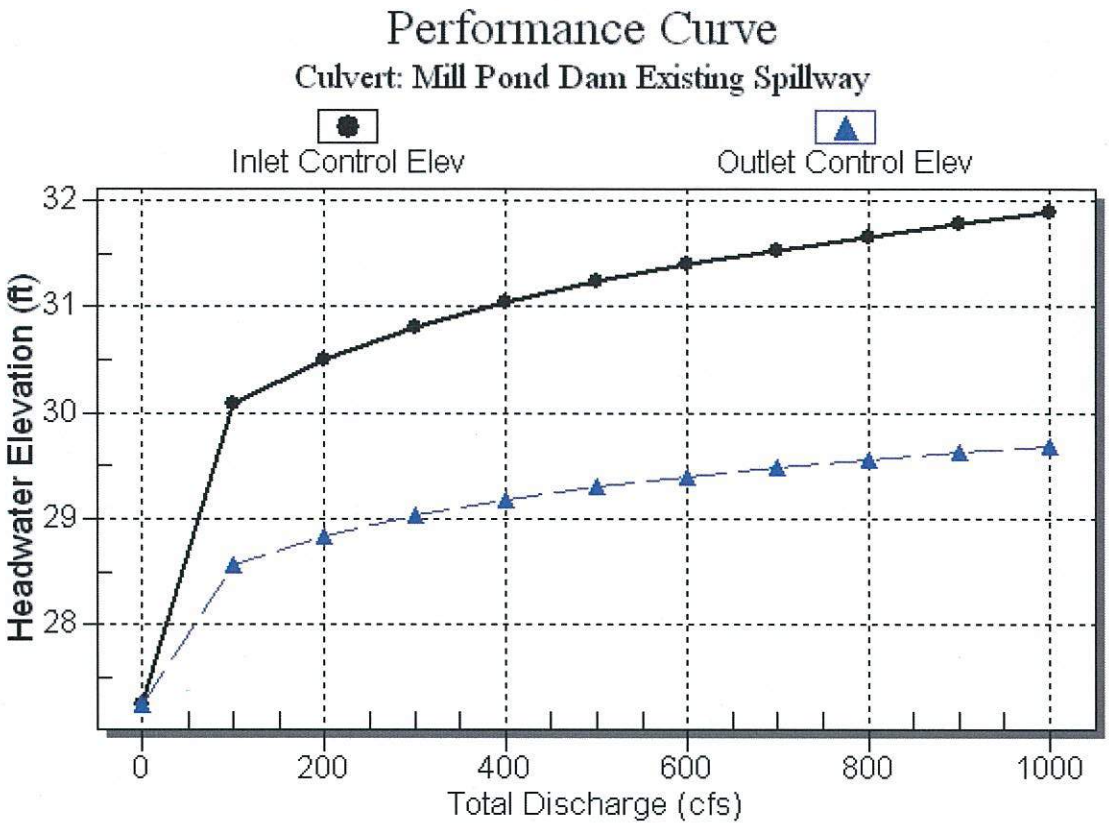
****

Inlet Elevation (invert): 27.25 ft, Outlet Elevation (invert): 26.00 ft

Culvert Length: 15.05 ft, Culvert Slope: 0.0833

****

Culvert Performance Curve Plot: Mill Pond Dam Existing Spillway



HY-8 Culvert Analysis Report

Project Notes

Project Title:

Designer:

Project Date: Tuesday, May 15, 2007

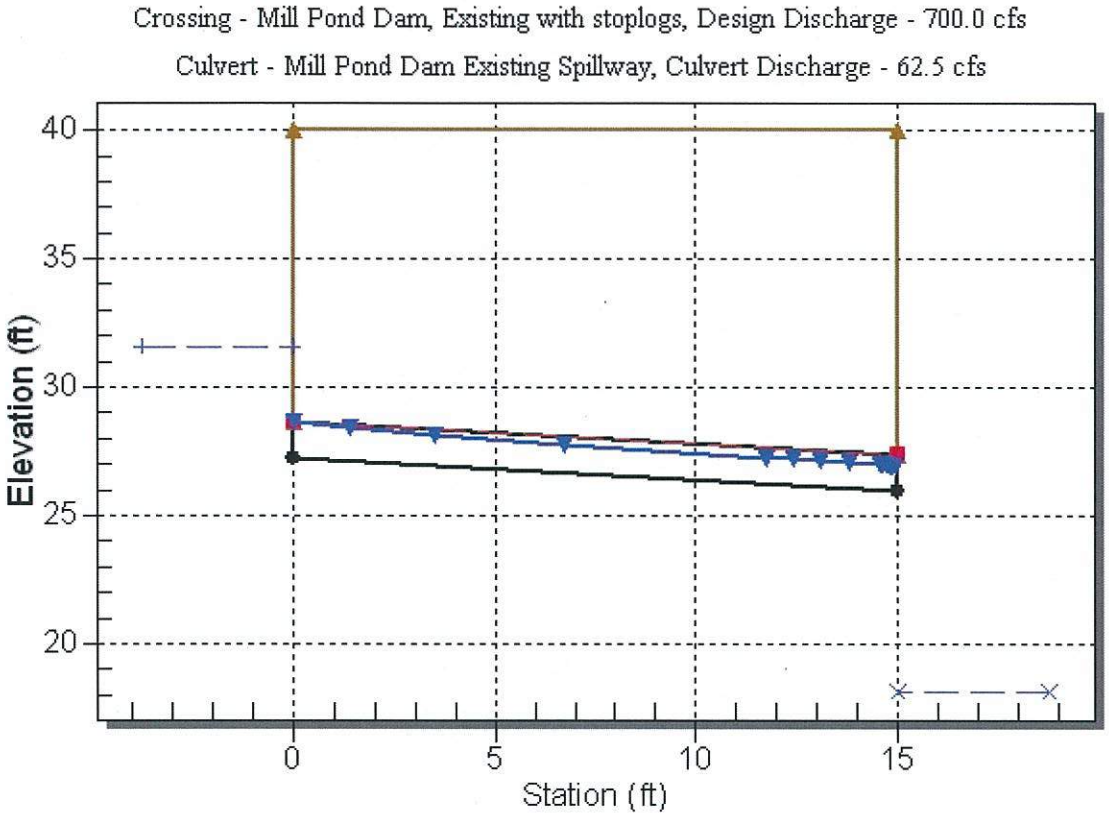
Notes:

Project Units: U.S. Customary Units

Outlet Control Option: Profiles

Crossing Notes: Mill Pond Dam, Existing with stoplogs

Water Surface Profile Plot for Culvert: Mill Pond Dam Existing Spillway



Site Data - Mill Pond Dam Existing Spillway

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 27.25 ft

Outlet Station: 15.00 ft

Outlet Elevation: 26.00 ft

Number of Barrels: 2

Culvert Data Summary - Mill Pond Dam Existing Spillway

Barrel Shape: Concrete Box

Barrel Span: 2.50 ft

Barrel Rise: 1.40 ft

Barrel Material: Concrete

Barrel Manning's n: 0.0120

Inlet Type: Conventional

Inlet Edge Condition: Square Edge (90°) Headwall

Table 3 - Downstream Channel Rating Curve (Crossing: Mill Pond Dam, Existing with stoplogs)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
0.00	15.00	0.00	0.00	0.00	0.00
100.00	16.46	1.46	4.11	1.57	0.82
200.00	16.92	1.92	4.89	2.07	0.86
300.00	17.26	2.26	5.41	2.42	0.88
400.00	17.52	2.52	5.82	2.71	0.90
500.00	17.75	2.75	6.15	2.96	0.91
600.00	17.95	2.95	6.44	3.17	0.92
700.00	18.14	3.14	6.69	3.37	0.93
800.00	18.30	3.30	6.92	3.54	0.94
900.00	18.46	3.46	7.13	3.71	0.94
1000.00	18.60	3.60	7.31	3.86	0.95

Tailwater Channel Data - Mill Pond Dam, Existing with stoplogs

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 2.00 ft

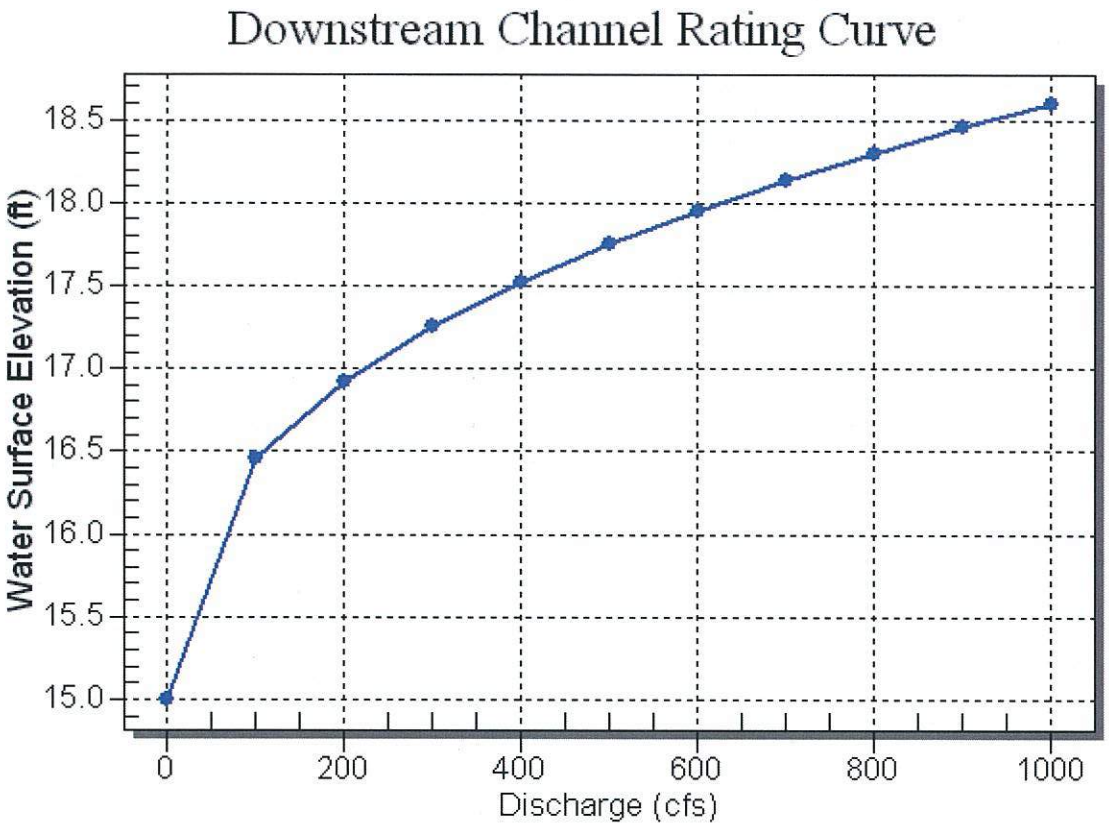
Side Slope (H:V): 10.00 (_:1)

Channel Slope: 0.0172

Channel Manning's n: 0.0400

Channel Invert Elevation: 15.00 ft

Tailwater Rating Curve Plot for Crossing: Mill Pond Dam, Existing with stoplogs



Roadway Data for Crossing: Mill Pond Dam, Existing with stoplogs

Roadway Profile Shape: Irregular Roadway Shape (coordinates)

Irregular Roadway Cross-Section:

Coord No.	Station (ft)	Elevation (ft)
1	0.00	40.00
2	88.00	30.00
3	106.00	29.50
4	125.00	29.50
5	129.00	31.24
6	145.00	31.15
7	168.00	30.98
8	188.00	31.09
9	207.00	31.24
10	211.00	31.08
11	214.50	29.50
12	222.00	29.50
13	240.00	30.00
14	266.00	32.00
15	314.00	40.00

Roadway Surface: Gravel

Roadway Top Width: 15.00 ft

Attachment 7

Proposed Discharge Rating Curve Calculations

MILL POND DAM

UPSTREAM SLOPING SECTION OF OVERTOPPING FLOWS

	Left Side					Right Side					Overtopping
	el.1=	29	el.2=	32.5		el.1=	29	el.2=	32.5		
	Lt=	60	slope=	0.0583		Lt=	40	slope=	0.0875		
WSEL	**H1, ft**	**yc1, ft**	**L1, ft**	**A1**	**Q1, cfs**	**H1, ft**	**yc1, ft**	**L1, ft**	**A1**	**Q1, cfs**	**Total, cfs**
29.00	0.0	0.0	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0
29.10	0.1	0.1	1.4	0.1	0	0.1	0.1	1.0	0.0	0	0
29.20	0.2	0.2	2.9	0.2	0	0.2	0.2	1.9	0.2	0	1
29.30	0.3	0.3	4.3	0.5	1	0.3	0.3	2.9	0.4	1	2
29.40	0.4	0.3	5.7	1.0	2	0.4	0.3	3.8	0.6	1	4
29.50	0.5	0.4	7.1	1.5	4	0.5	0.4	4.8	1.0	3	6
29.60	0.6	0.5	8.6	2.1	6	0.6	0.5	5.7	1.4	4	10
29.70	0.7	0.6	10.0	2.9	9	0.7	0.6	6.7	1.9	6	15
29.80	0.8	0.7	11.4	3.8	12	0.8	0.7	7.6	2.5	8	21
29.90	0.9	0.8	12.9	4.8	17	0.9	0.8	8.6	3.2	11	28
30.00	1.0	0.8	14.3	6.0	22	1.0	0.8	9.5	4.0	15	36
30.10	1.1	0.9	15.7	7.2	28	1.1	0.9	10.5	4.8	18	46
30.20	1.2	1.0	17.1	8.6	34	1.2	1.0	11.4	5.7	23	57
30.30	1.3	1.1	18.6	10.1	42	1.3	1.1	12.4	6.7	28	70
30.40	1.4	1.2	20.0	11.7	51	1.4	1.2	13.3	7.8	34	84
30.50	1.5	1.3	21.4	13.4	60	1.5	1.3	14.3	8.9	40	100
30.60	1.6	1.3	22.9	15.2	71	1.6	1.3	15.2	10.2	47	118
30.70	1.7	1.4	24.3	17.2	82	1.7	1.4	16.2	11.5	55	137
30.80	1.8	1.5	25.7	19.3	95	1.8	1.5	17.1	12.9	63	158
30.90	1.9	1.6	27.1	21.5	108	1.9	1.6	18.1	14.3	72	181
31.00	2.0	1.7	28.6	23.8	123	2.0	1.7	19.0	15.9	82	206
31.10	2.1	1.8	30.0	26.3	139	2.1	1.8	20.0	17.5	93	232
31.20	2.2	1.8	31.4	28.8	157	2.2	1.8	21.0	19.2	104	261
31.30	2.3	1.9	32.9	31.5	175	2.3	1.9	21.9	21.0	117	292
31.40	2.4	2.0	34.3	34.3	195	2.4	2.0	22.9	22.9	130	324
31.50	2.5	2.1	35.7	37.2	215	2.5	2.1	23.8	24.8	144	359
31.60	2.6	2.2	37.1	40.2	238	2.6	2.2	24.8	26.8	158	396
31.70	2.7	2.3	38.6	43.4	261	2.7	2.3	25.7	28.9	174	435
31.80	2.8	2.3	40.0	46.7	286	2.8	2.3	26.7	31.1	191	477
31.90	2.9	2.4	41.4	50.1	312	2.9	2.4	27.6	33.4	208	520
32.00	3.0	2.5	42.9	53.6	340	3.0	2.5	28.6	35.7	227	566
32.10	3.1	2.6	44.3	57.2	369	3.1	2.6	29.5	38.1	246	615
32.20	3.2	2.7	45.7	61.0	399	3.2	2.7	30.5	40.6	266	666
32.30	3.3	2.8	47.1	64.8	431	3.3	2.8	31.4	43.2	288	719
32.40	3.4	2.8	48.6	68.8	465	3.4	2.8	32.4	45.9	310	775
32.50	3.5	2.9	50.0	72.9	500	3.5	2.9	33.3	48.6	333	833
33.00	4.0	3.3	55.7	90.5	655	4.0	3.3	37.1	60.4	437	1,091

MILL POND DAM

DOWNSTREAM SLOPING SECTION OF OVERTOPPING FLOWS

						Right Side					Overtopping
	el.1=		el.2=			el.1=	29	el.2=	32.5		
	Lt=		slope=	#DIV/0!		Lt=	35	slope=	0.1000		
WSEL	**H1, ft**	**yc1, ft**	**L1, ft**	**A1**	**Q1, cfs**	**H1, ft**	**yc1, ft**	**L1, ft**	**A1**	**Q1, cfs**	**Total, cfs**
29.00	29.0	19.3	0.0	0.0	0	0.0	0.0	0.0	0.0	0	0
29.10	29.1	19.4	0.0	0.0		0.1	0.1	0.8	0.0	0	0
29.20	29.2	19.5	0.0	0.0		0.2	0.2	1.7	0.1	0	0
29.30	29.3	19.5	0.0	0.0		0.3	0.3	2.5	0.3	1	1
29.40	29.4	19.6	0.0	0.0		0.4	0.3	3.3	0.6	1	1
29.50	29.5	19.7	0.0	0.0		0.5	0.4	4.2	0.9	2	2
29.60	29.6	19.7	0.0	0.0		0.6	0.5	5.0	1.3	4	4
29.70	29.7	19.8	0.0	0.0		0.7	0.6	5.8	1.7	5	5
29.80	29.8	19.9	0.0	0.0		0.8	0.7	6.7	2.2	7	7
29.90	29.9	19.9	0.0	0.0		0.9	0.8	7.5	2.8	10	10
30.00	30.0	20.0	0.0	0.0		1.0	0.8	8.3	3.5	13	13
30.10	30.1	20.1	0.0	0.0		1.1	0.9	9.2	4.2	16	16
30.20	30.2	20.1	0.0	0.0		1.2	1.0	10.0	5.0	20	20
30.30	30.3	20.2	0.0	0.0		1.3	1.1	10.8	5.9	25	25
30.40	30.4	20.3	0.0	0.0		1.4	1.2	11.7	6.8	29	29
30.50	30.5	20.3	0.0	0.0		1.5	1.3	12.5	7.8	35	35
30.60	30.6	20.4	0.0	0.0		1.6	1.3	13.3	8.9	41	41
30.70	30.7	20.5	0.0	0.0		1.7	1.4	14.2	10.0	48	48
30.80	30.8	20.5	0.0	0.0		1.8	1.5	15.0	11.3	55	55
30.90	30.9	20.6	0.0	0.0		1.9	1.6	15.8	12.5	63	63
31.00	31.0	20.7	0.0	0.0		2.0	1.7	16.7	13.9	72	72
31.10	31.1	20.7	0.0	0.0		2.1	1.8	17.5	15.3	81	81
31.20	31.2	20.8	0.0	0.0		2.2	1.8	18.3	16.8	91	91
31.30	31.3	20.9	0.0	0.0		2.3	1.9	19.2	18.4	102	102
31.40	31.4	20.9	0.0	0.0		2.4	2.0	20.0	20.0	113	113
31.50	31.5	21.0	0.0	0.0		2.5	2.1	20.8	21.7	126	126
31.60	31.6	21.1	0.0	0.0		2.6	2.2	21.7	23.5	139	139
31.70	31.7	21.1	0.0	0.0		2.7	2.3	22.5	25.3	152	152
31.80	31.8	21.2	0.0	0.0		2.8	2.3	23.3	27.2	167	167
31.90	31.9	21.3	0.0	0.0		2.9	2.4	24.2	29.2	182	182
32.00	32.0	21.3	0.0	0.0		3.0	2.5	25.0	31.3	198	198
32.10	32.1	21.4	0.0	0.0		3.1	2.6	25.8	33.4	215	215
32.20	32.2	21.5	0.0	0.0		3.2	2.7	26.7	35.6	233	233
32.30	32.3	21.5	0.0	0.0		3.3	2.8	27.5	37.8	252	252
32.40	32.4	21.6	0.0	0.0		3.4	2.8	28.3	40.1	271	271
32.50	32.5	21.7	0.0	0.0		3.5	2.9	29.2	42.5	291	291
33.00	33.0	22.0	0.0	0.0		4.0	3.3	32.5	52.8	382	382

Client: Town of Rockport
Project: Mill Pond Dam
Project No.: 07104-1

UPSTREAM WALL Discharge Rating Curve Calculation

	1. Double Box Culvert Discharge, Weir and Orifice Flow Estimate						
Res.	**Crest El. Ft**	**breadth, ft**	**Length, ft**	**h/Lb**	**c for 3/4:1**	**Head, ft**	**Q=clh^1.5, cfs**
Elev,	**Top Open**	**Bottom Open**	**Length, ft**	**Area**	**orifice c**	**Head, ft**	**Q=cA(2gh)^0.5**
25.6	25.60	15.0	5.0	0.00	2.63	0.0	0
25.7	25.60	15.0	5.0	0.01	2.63	0.1	0
25.8	25.60	15.0	5.0	0.01	2.63	0.2	1
25.9	25.60	15.0	5.0	0.02	2.63	0.3	2
26.0	25.60	15.0	5.0	0.03	2.63	0.4	3
26.1	25.60	15.0	5.0	0.03	2.63	0.5	5
26.2	25.60	15.0	5.0	0.04	2.63	0.6	6
26.3	25.60	15.0	5.0	0.05	2.63	0.7	8
26.4	25					0.8	9
26.5	25.60	15.0	5.0	0.06	2.63	0.9	11
26.6	25.60	15.0	5.0	0.07	2.63	1.0	13
26.7	25.60	15.0	5.0	0.07	2.63	1.1	15
26.8	25.60	15.0	5.0	0.08	2.63	1.2	17
26.9	25.60	15.0	5.0	0.09	2.63	1.3	19
27.0	25.60	15.0	5.0	0.09	2.63	1.4	22
27.1	25.60	15.0	5.0	0.10	2.63	1.5	24
27.2	25.60	15.0	5.0	0.11	2.63	1.6	27
27.25	25.60	15.0	5.0	0.11	2.63	1.7	28
27.40	25.60	15.0	5.0	0.12	2.63	1.8	32
27.5	25.60	15.0	5.0	0.13	2.63	1.9	34
27.6	25.60	15.0	5.0	0.13	2.63	2.0	37
27.7	25.60	15.0	5.0	0.14	2.63	2.1	40
27.8	25.60	15.0	5.0	0.15	2.63	2.2	43
27.9	25.60	15.0	5.0	0.15	2.63	2.3	46
28.0	25.60	15.0	5.0	0.16	2.63	2.4	49
28.1	25.60	15.0	5.0	0.17	2.63	2.5	52
28.2	25.60	15.0	5.0	0.17	2.63	2.6	55
28.3	25.60	15.0	5.0	0.18	2.63	2.7	58
28.4	25.60	15.0	5.0	0.19	2.63	2.8	62
28.5	25.60	15.0	5.0	0.19	2.63	2.9	65
28.6	28.50	25.60	5.0	14.5	0.60	1.6	87
28.7	28.50	25.6	5.0	14.5	0.60	1.7	90
28.8	28.50	25.6	5.0	14.5	0.60	1.8	92
28.9	28.50	25.6	5.0	14.5	0.60	1.9	95
29.0	28.50	25.6	5.0	14.5	0.60	2.0	97
29.1	28.50	25.6	5.0	14.5	0.60	2.1	100
29.2	28.50	25.6	5.0	14.5	0.60	2.2	102
29.3	28.50	25.6	5.0	14.5	0.60	2.3	105
29.4	28.50	25.6	5.0	14.5	0.60	2.4	107
29.5	28.50	25.6	5.0	14.5	0.60	2.5	109
29.6	28.50	25.6	5.0	14.5	0.60	2.6	111
29.7	28.50	25.6	5.0	14.5	0.60	2.7	114
29.8	28.50	25.6	5.0	14.5	0.60	2.8	116
29.9	28.50	25.6	5.0	14.5	0.60	2.9	118
30.0	28.50	25.6	5.0	14.5	0.60	3.0	120
30.1	28.50	25.6	5.0	14.5	0.60	3.1	122
30.2	28.50	25.6	5.0	14.5	0.60	3.2	124
30.3	28.50	25.6	5.0	14.5	0.60	3.3	126
30.4	28.50	25.6	5.0	14.5	0.60	3.4	128
30.5	28.50	25.6	5.0	14.5	0.60	3.5	130
30.6	28.50	25.6	5.0	14.5	0.60	3.6	132
30.7	28.50	25.6	5.0	14.5	0.60	3.7	133
30.8	28.50	25.6	5.0	14.5	0.60	3.8	135
30.9	28.50	25.6	5.0	14.5	0.60	3.9	137
31.0	28.50	25.6	5.0	14.5	0.60	4.0	139
31.1	28.50	25.6	5.0	14.5	0.60	4.1	141
31.2	28.50	25.6	5.0	14.5	0.60	4.2	142
31.3	28.50	25.6	5.0	14.5	0.60	4.3	144
31.4	28.50	25.6	5.0	14.5	0.60	4.4	146
31.5	28.50	25.6	5.0	14.5	0.60	4.5	147
31.6	28.50	25.6	5.0	14.5	0.60	4.6	149
31.7	28.50	25.6	5.0	14.5	0.60	4.7	151
31.8	28.50	25.6	5.0	14.5	0.60	4.8	152
31.9	28.50	25.6	5.0	14.5	0.60	4.9	154
32.0	28.50	25.6	5.0	14.5	0.60	5.0	155
32.1	28.50	25.6	5.0	14.5	0.60	5.1	157
32.2	28.50	25.6	5.0	14.5	0.60	5.2	158
32.3	28.50	25.6	5.0	14.5	0.60	5.3	160
32.4	28.50	25.6	5.0	14.5	0.60	5.4	161
32.5	28.50	25.6	5.0	14.5	0.60	5.5	163
33.0	28.50	25.6	5.0	14.5	0.60	6.0	170

Assume stop logs removed during storm.

UPSTREAI

Res. Elev,	2. Flow under or through lowest railing over box culvert						
	Crest El. Ft	breadth, ft	Length, ft	h/Lb	c	Head, ft	Q=clh^1.5, cfs
	Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5
25.6							
25.7							
25.8							
25.9							
26.0							
26.1							
26.2							
26.3							
26.4							
26.5							
26.6							
26.7							
26.8							
26.9							
27.0							
27.1							
27.2							
27.25							
27.40							
27.5							
27.6							
27.7							
27.8							
27.9							
28.0							
28.1							
28.2							
28.3							
28.4							
28.5							
28.6							
28.7							
28.8							
28.9							
29.0	29.0	15.0	7.7	0.00	2.63	0.0	0
29.1	29.0	15.0	7.7	0.01	2.63	0.1	1
29.2	29.0	15.0	7.7	0.01	2.63	0.2	2
29.3	29.0	15.0	7.7	0.02	2.63	0.3	3
29.4	29.0	15.0	7.7	0.03	2.63	0.4	5
29.5	29.4	29.0	7.7	3.1	0.60	0.3	8
29.6	29.4	29.0	7.7	3.1	0.60	0.4	9
29.7	29.4	29.0	7.7	3.1	0.60	0.5	10
29.8	29.4	29.0	7.7	3.1	0.60	0.6	11
29.9	29.4	29.0	7.7	3.1	0.60	0.7	12
30.0	29.4	29.0	7.7	3.1	0.60	0.8	13
30.1	29.4	29.0	7.7	3.1	0.60	0.9	14
30.2	29.4	29.0	7.7	3.1	0.60	1.0	15
30.3	29.4	29.0	7.7	3.1	0.60	1.1	16
30.4	29.4	29.0	7.7	3.1	0.60	1.2	16
30.5	29.4	29.0	7.7	3.1	0.60	1.3	17
30.6	29.4	29.0	7.7	3.1	0.60	1.4	18
30.7	29.4	29.0	7.7	3.1	0.60	1.5	18
30.8	29.4	29.0	7.7	3.1	0.60	1.6	19
30.9	29.4	29.0	7.7	3.1	0.60	1.7	19
31.0	29.4	29.0	7.7	3.1	0.60	1.8	20
31.1	29.4	29.0	7.7	3.1	0.60	1.9	20
31.2	29.4	29.0	7.7	3.1	0.60	2.0	21
31.3	29.4	29.0	7.7	3.1	0.60	2.1	21
31.4	29.4	29.0	7.7	3.1	0.60	2.2	22
31.5	29.4	29.0	7.7	3.1	0.60	2.3	22
31.6	29.4	29.0	7.7	3.1	0.60	2.4	23
31.7	29.4	29.0	7.7	3.1	0.60	2.5	23
31.8	29.4	29.0	7.7	3.1	0.60	2.6	24
31.9	29.4	29.0	7.7	3.1	0.60	2.7	24
32.0	29.4	29.0	7.7	3.1	0.60	2.8	25
32.1	29.4	29.0	7.7	3.1	0.60	2.9	25
32.2	29.4	29.0	7.7	3.1	0.60	3.0	26
32.3	29.4	29.0	7.7	3.1	0.60	3.1	26
32.4	29.4	29.0	7.7	3.1	0.60	3.2	26
32.5	29.4	29.0	7.7	3.1	0.60	3.3	27
33.0	29.4	29.0	7.7	3.1	0.60	3.8	29

UPSTREAI

Res. Elev,		3. Flow under or through middle railing over box culvert						
		Crest El. Ft	breadth, ft	Length, ft	h/Lb	c	Head, ft	Q=clh^1.5, cfs
		Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5
25.6								
25.7								
25.8								
25.9								
26.0								
26.1								
26.2								
26.3								
26.4								
26.5								
26.6								
26.7								
26.8								
26.9								
27.0								
27.1								
27.2								
27.25								
27.40								
27.5								
27.6								
27.7								
27.8								
27.9								
28.0								
28.1								
28.2								
28.3								
28.4								
28.5								
28.6								
28.7								
28.8								
28.9								
29.0								
29.1								
29.2								
29.3								
29.4								
29.5								
29.6		29.6	0.2	7.7	0.00	2.63	0.0	0
29.7		29.6	0.2	7.7	0.48	2.69	0.1	1
29.8		29.6	0.2	7.7	0.96	2.97	0.2	2
29.9		29.6	0.2	7.7	1.44	3.27	0.3	4
30.0		29.6	0.2	7.7	1.92	3.50	0.4	7
30.1		29.6	0.2	7.7	2.40	3.65	0.5	10
30.2		29.6	0.2	7.7	2.88	3.65	0.6	13
30.3		29.6	0.2	7.7	3.36	3.65	0.7	16
30.4		29.6	0.2	7.7	3.84	3.65	0.8	20
30.5		29.6	0.2	7.7	4.32	3.65	0.9	24
30.6		29.6	0.2	7.7	4.80	3.65	1.0	28
30.7		29.6	0.2	7.7	5.28	3.65	1.1	32
30.8		29.6	0.2	7.7	5.76	3.65	1.2	37
30.9		29.6	0.2	7.7	6.24	3.65	1.3	42
31.0	6.46	30.9	29.6	7.7	10.0	0.60	0.8	42
31.1		30.9	29.6	7.7	10.0	0.60	0.9	44
31.2		30.9	29.6	7.7	10.0	0.60	1.0	47
31.3		30.9	29.6	7.7	10.0	0.60	1.1	49
31.4		30.9	29.6	7.7	10.0	0.60	1.2	52
31.5		30.9	29.6	7.7	10.0	0.60	1.3	54
31.6		30.9	29.6	7.7	10.0	0.60	1.4	56
31.7		30.9	29.6	7.7	10.0	0.60	1.5	58
31.8		30.9	29.6	7.7	10.0	0.60	1.6	60
31.9		30.9	29.6	7.7	10.0	0.60	1.7	62
32.0		30.9	29.6	7.7	10.0	0.60	1.8	64
32.1		30.9	29.6	7.7	10.0	0.60	1.9	65
32.2		30.9	29.6	7.7	10.0	0.60	2.0	67
32.3		30.9	29.6	7.7	10.0	0.60	2.1	69
32.4		30.9	29.6	7.7	10.0	0.60	2.2	71
32.5		30.9	29.6	7.7	10.0	0.60	2.3	72
33.0		30.9	29.6	7.7	10.0	0.60	2.8	80

UPSTREAI

Res. Elev,		4. Flow under or through Granite Slab Bench						
		Crest El. Ft	breadth, ft	Length, ft	h/Lb	c	Head, ft	Q=clh^1.5, cfs
		Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5
25.6								
25.7								
25.8								
25.9								
26.0								
26.1								
26.2								
26.3								
26.4								
26.5								
26.6								
26.7								
26.8								
26.9								
27.0								
27.1								
27.2								
27.25								
27.40								
27.5								
27.6								
27.7								
27.8								
27.9								
28.0								
28.1								
28.2								
28.3								
28.4								
28.5								
28.6								
28.7								
28.8								
28.9								
29.0		29.0	15.0	40.0	0.00	2.63	0.0	0
29.1		29.0	15.0	40.0	0.01	2.63	0.1	3
29.2		29.0	15.0	40.0	0.01	2.63	0.2	9
29.3		29.0	15.0	40.0	0.02	2.63	0.3	17
29.4		29.0	15.0	40.0	0.03	2.63	0.4	27
29.5		29.0	15.0	40.0	0.03	2.63	0.5	37
29.6		29.0	15.0	40.0	0.04	2.63	0.6	49
29.7		29.0	15.0	40.0	0.05	2.63	0.7	62
29.8		29.0	15.0	40.0	0.05	2.63	0.8	75
29.9		29.0	15.0	40.0	0.06	2.63	0.9	90
30.0		29.0	15.0	40.0	0.07	2.63	1.0	105
30.1		29.0	15.0	40.0	0.07	2.63	1.1	121
30.2		29.0	15.0	40.0	0.08	2.63	1.2	138
30.3		29.0	15.0	40.0	0.09	2.63	1.3	156
30.4		29.0	15.0	40.0	0.09	2.63	1.4	174
30.5		29.0	15.0	40.0	0.10	2.63	1.5	193
30.6		29.0	15.0	40.0	0.11	2.63	1.6	213
30.7		29.0	15.0	40.0	0.11	2.63	1.7	233
30.8		29.0	15.0	40.0	0.12	2.63	1.8	254
30.9		29.0	15.0	40.0	0.13	2.63	1.9	275
31.0	4.17	30.9	29.0	40.0	76.0	0.60	1.1	375
31.1		30.9	29.0	40.0	76.0	0.60	1.2	392
31.2		30.9	29.0	40.0	76.0	0.60	1.3	409
31.3		30.9	29.0	40.0	76.0	0.60	1.4	425
31.4		30.9	29.0	40.0	76.0	0.60	1.5	441
31.5		30.9	29.0	40.0	76.0	0.60	1.6	456
31.6		30.9	29.0	40.0	76.0	0.60	1.7	470
31.7		30.9	29.0	40.0	76.0	0.60	1.8	484
31.8		30.9	29.0	40.0	76.0	0.60	1.9	498
31.9		30.9	29.0	40.0	76.0	0.60	2.0	511
32.0		30.9	29.0	40.0	76.0	0.60	2.1	524
32.1		30.9	29.0	40.0	76.0	0.60	2.2	537
32.2		30.9	29.0	40.0	76.0	0.60	2.3	549
32.3		30.9	29.0	40.0	76.0	0.60	2.4	561
32.4		30.9	29.0	40.0	76.0	0.60	2.5	573
32.5		30.9	29.0	40.0	76.0	0.60	2.6	584
33.0		30.9	29.0	40.0	76.0	0.60	3.1	639

UPSTREAI		5. Flow Over Bench							Slope Area
Res. Elev,		Crest El. Ft	breadth, ft	Length, ft	h/Lb	c	Head, ft	Q=clh^1.5, cfs	LEFT and RT Sides
25.6									
25.7									
25.8									
25.9									
26.0									
26.1									
26.2									
26.3									
26.4									
26.5									
26.6									
26.7									
26.8									
26.9									
27.0									
27.1									
27.2									
27.25									
27.40									
27.5									
27.6									
27.7									
27.8									
27.9									
28.0									
28.1									
28.2									
28.3									
28.4									
28.5									
28.6									
28.7									
28.8									
28.9									
29.0									0
29.1									0
29.2									1
29.3									2
29.4									4
29.5									6
29.6									10
29.7									15
29.8									21
29.9									28
30.0									36
30.1									46
30.2									57
30.3									70
30.4									84
30.5									100
30.6									118
30.7									137
30.8									158
30.9									181
31.0	4.93								206
31.1									232
31.2									261
31.3									292
31.4		31.4	2.0	81.3	0.00	2.63	0.0	0	324
31.5		31.4	2.0	81.3	0.05	2.63	0.1	7	359
31.6		31.4	2.0	81.3	0.10	2.63	0.2	19	396
31.7		31.4	2.0	81.3	0.15	2.63	0.3	35	435
31.8		31.4	2.0	81.3	0.20	2.63	0.4	54	477
31.9		31.4	2.0	81.3	0.25	2.63	0.5	76	520
32.0		31.4	2.0	81.3	0.30	2.64	0.6	100	566
32.1		31.4	2.0	81.3	0.35	2.65	0.7	126	615
32.2		31.4	2.0	81.3	0.40	2.66	0.8	155	666
32.3		31.4	2.0	81.3	0.45	2.68	0.9	186	719
32.4		31.4	2.0	81.3	0.50	2.70	1.0	220	775
32.5		31.4	2.0	81.3	0.55	2.72	1.1	255	833
33.0		31.4	2.0	81.3	0.80	2.85	1.6	469	1,091

UPSTREAI

Res. Elev,	TOTAL Q, cfs	ELEV, ft
25.6	0	25.6
25.7	0	25.7
25.8	1	25.8
25.9	2	25.9
26.0	3	26.0
26.1	5	26.1
26.2	6	26.2
26.3	8	26.3
26.4	9	26.4
26.5	11	26.5
26.6	13	26.6
26.7	15	26.7
26.8	17	26.8
26.9	19	26.9
27.0	22	27.0
27.1	24	27.1
27.2	27	27.2
27.25	28	27.25
27.40	32	27.40
27.5	34	27.5
27.6	37	27.6
27.7	40	27.7
27.8	43	27.8
27.9	46	27.9
28.0	49	28.0
28.1	52	28.1
28.2	55	28.2
28.3	58	28.3
28.4	62	28.4
28.5	65	28.5
28.6	87	28.6
28.7	90	28.7
28.8	92	28.8
28.9	95	28.9
29.0	97	29.0
29.1	104	29.1
29.2	114	29.2
29.3	127	29.3
29.4	142	29.4
29.5	161	29.5
29.6	180	29.6
29.7	201	29.7
29.8	225	29.8
29.9	252	29.9
30.0	281	30.0
30.1	313	30.1
30.2	347	30.2
30.3	384	30.3
30.4	422	30.4
30.5	464	30.5
30.6	507	30.6
30.7	554	30.7
30.8	602	30.8
30.9	654	30.9
31.0	781	31.0
31.1	830	31.1
31.2	880	31.2
31.3	931	31.3
31.4	984	31.4
31.5	1045	31.5
31.6	1113	31.6
31.7	1186	31.7
31.8	1264	31.8
31.9	1347	31.9
32.0	1434	32.0
32.1	1525	32.1
32.2	1621	32.2
32.3	1721	32.3
32.4	1825	32.4
32.5	1934	32.5
33.0	2478	33.0

Client: Town of Rockport
Project: Mill Pond Dam
Project No.: 07104-1

DOWNSTREAM WALL Discharge Rating Curve Calculation

	1. Double Box Culvert Discharge, Weir and Orifice Flow Estimate						
Res.	**Crest El. Ft**	**breadth, ft**	**Length, ft**	**h/Lb**	**c for 3/4:1**	**Head, ft**	**Q=clh^1.5, cfs**
Elev,	**Top Open**	**Bottom Open**	**Length, ft**	**Area**	**orifice c**	**Head, ft**	**Q=cA(2gh)^0.5**
25.6	25.6	15.0	5.0	0.00	2.63	0.0	0
25.7	25.6	15.0	5.0	0.01	2.63	0.1	0
25.8	25.6	15.0	5.0	0.01	2.63	0.2	1
25.9	25.6	15.0	5.0	0.02	2.63	0.3	2
26.0	25.6	15.0	5.0	0.03	2.63	0.4	3
26.1	25.6	15.0	5.0	0.03	2.63	0.5	5
26.2	25.6	15.0	5.0	0.04	2.63	0.6	6
26.3	25.6	15.0	5.0	0.05	2.63	0.7	8
26.4	25.6	15.0	5.0	0.05	2.63	0.8	9
26.5	25.6	15.0	5.0	0.06	2.63	0.9	11
26.6	25.6	15.0	5.0	0.07	2.63	1.0	13
26.7	25.6	15.0	5.0	0.07	2.63	1.1	15
26.8	25.6	15.0	5.0	0.08	2.63	1.2	17
26.9	25.6	15.0	5.0	0.09	2.63	1.3	19
27.0	25.6	15.0	5.0	0.09	2.63	1.4	22
27.1	25.6	15.0	5.0	0.10	2.63	1.5	24
27.2	25.6	15.0	5.0	0.11	2.63	1.6	27
27.3	25.6	15.0	5.0	0.11	2.63	1.7	29
27.4	25.6	15.0	5.0	0.12	2.63	1.8	32
27.5	25.6	15.0	5.0	0.13	2.63	1.9	34
27.6	25.6	15.0	5.0	0.13	2.63	2.0	37
27.7	25.6	15.0	5.0	0.14	2.63	2.1	40
27.8	25.6	15.0	5.0	0.15	2.63	2.2	43
27.9	25.6	15.0	5.0	0.15	2.63	2.3	46
28.0	25.6	15.0	5.0	0.16	2.63	2.4	49
28.1	28.1	25.6	5.0	12.5	0.60	1.3	67
28.2	28.1	25.6	5.0	12.5	0.60	1.4	70
28.3	28.1	25.6	5.0	12.5	0.60	1.5	72
28.4	28.1	25.6	5.0	12.5	0.60	1.6	75
28.5	28.1	25.6	5.0	12.5	0.60	1.7	77
28.6	28.1	25.6	5.0	12.5	0.60	1.8	80
28.7	28.1	25.6	5.0	12.5	0.60	1.9	82
28.8	28.1	25.6	5.0	12.5	0.60	2.0	84
28.9	28.1	25.6	5.0	12.5	0.60	2.1	86
29.0	28.1	25.6	5.0	12.5	0.60	2.2	88
29.1	28.1	25.6	5.0	12.5	0.60	2.3	90
29.2	28.1	25.6	5.0	12.5	0.60	2.4	92
29.3	28.1	25.6	5.0	12.5	0.60	2.5	94
29.4	28.1	25.6	5.0	12.5	0.60	2.6	96
29.5	28.1	25.6	5.0	12.5	0.60	2.7	98
29.6	28.1	25.6	5.0	12.5	0.60	2.8	100
29.7	28.1	25.6	5.0	12.5	0.60	2.9	102
29.8	28.1	25.6	5.0	12.5	0.60	3.0	103
29.9	28.1	25.6	5.0	12.5	0.60	3.1	105
30.0	28.1	25.6	5.0	12.5	0.60	3.2	107
30.1	28.1	25.6	5.0	12.5	0.60	3.3	109
30.2	28.1	25.6	5.0	12.5	0.60	3.4	110
30.3	28.1	25.6	5.0	12.5	0.60	3.5	112
30.4	28.1	25.6	5.0	12.5	0.60	3.6	113
30.5	28.1	25.6	5.0	12.5	0.60	3.7	115
30.6	28.1	25.6	5.0	12.5	0.60	3.8	117
30.7	28.1	25.6	5.0	12.5	0.60	3.9	118
30.8	28.1	25.6	5.0	12.5	0.60	4.0	120
30.9	28.1	25.6	5.0	12.5	0.60	4.1	121
31.0	28.1	25.6	5.0	12.5	0.60	4.2	123
31.1	28.1	25.6	5.0	12.5	0.60	4.3	124
31.2	28.1	25.6	5.0	12.5	0.60	4.4	126
31.3	28.1	25.6	5.0	12.5	0.60	4.5	127
31.4	28.1	25.6	5.0	12.5	0.60	4.6	128
31.5	28.1	25.6	5.0	12.5	0.60	4.7	130
31.6	28.1	25.6	5.0	12.5	0.60	4.8	131
31.7	28.1	25.6	5.0	12.5	0.60	4.9	133
31.8	28.1	25.6	5.0	12.5	0.60	5.0	134
31.9	28.1	25.6	5.0	12.5	0.60	5.1	135
32.0	28.1	25.6	5.0	12.5	0.60	5.2	137
32.1	28.1	25.6	5.0	12.5	0.60	5.3	138
32.2	28.1	25.6	5.0	12.5	0.60	5.4	139
32.3	28.1	25.6	5.0	12.5	0.60	5.5	141
32.4	28.1	25.6	5.0	12.5	0.60	5.6	142
32.5	28.1	25.6	5.0	12.5	0.60	5.7	143
33.0	28.1	25.6	5.0	12.5	0.60	6.2	149

DOWNSTREA

Res.	2. Flow under or through lowest railing						
	Crest El. Ft	breadth, ft	Length, ft	h/Lb	c for 3/4:1	Head, ft	Q=clh^1.5, cfs
Elev,	Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5
25.6							
25.7							
25.8							
25.9							
26.0							
26.1							
26.2							
26.3							
26.4							
26.5							
26.6							
26.7							
26.8							
26.9							
27.0							
27.1							
27.2							
27.3							
27.4							
27.5							
27.6							
27.7							
27.8							
27.9							
28.0							
28.1							
28.2							
28.3							
28.4							
28.5							
28.6							
28.7							
28.8							
28.9							
29.0	29.0	15.0	63.0	0.00	2.63	0.0	0
29.1	29.0	15.0	63.0	0.01	2.63	0.1	5
29.2	29.0	15.0	63.0	0.01	2.63	0.2	15
29.3	29.0	15.0	63.0	0.02	2.63	0.3	27
29.4	29.0	15.0	63.0	0.03	2.63	0.4	42
29.5	29.4	29.0	63.0	25.2	0.60	0.3	66
29.6	29.4	29.0	63.0	25.2	0.60	0.4	77
29.7	29.4	29.0	63.0	25.2	0.60	0.5	86
29.8	29.4	29.0	63.0	25.2	0.60	0.6	94
29.9	29.4	29.0	63.0	25.2	0.60	0.7	102
30.0	29.4	29.0	63.0	25.2	0.60	0.8	109
30.1	29.4	29.0	63.0	25.2	0.60	0.9	115
30.2	29.4	29.0	63.0	25.2	0.60	1.0	121
30.3	29.4	29.0	63.0	25.2	0.60	1.1	127
30.4	29.4	29.0	63.0	25.2	0.60	1.2	133
30.5	29.4	29.0	63.0	25.2	0.60	1.3	138
30.6	29.4	29.0	63.0	25.2	0.60	1.4	144
30.7	29.4	29.0	63.0	25.2	0.60	1.5	149
30.8	29.4	29.0	63.0	25.2	0.60	1.6	153
30.9	29.4	29.0	63.0	25.2	0.60	1.7	158
31.0	29.4	29.0	63.0	25.2	0.60	1.8	163
31.1	29.4	29.0	63.0	25.2	0.60	1.9	167
31.2	29.4	29.0	63.0	25.2	0.60	2.0	172
31.3	29.4	29.0	63.0	25.2	0.60	2.1	176
31.4	29.4	29.0	63.0	25.2	0.60	2.2	180
31.5	29.4	29.0	63.0	25.2	0.60	2.3	184
31.6	29.4	29.0	63.0	25.2	0.60	2.4	188
31.7	29.4	29.0	63.0	25.2	0.60	2.5	192
31.8	29.4	29.0	63.0	25.2	0.60	2.6	196
31.9	29.4	29.0	63.0	25.2	0.60	2.7	199
32.0	29.4	29.0	63.0	25.2	0.60	2.8	203
32.1	29.4	29.0	63.0	25.2	0.60	2.9	207
32.2	29.4	29.0	63.0	25.2	0.60	3.0	210
32.3	29.4	29.0	63.0	25.2	0.60	3.1	214
32.4	29.4	29.0	63.0	25.2	0.60	3.2	217
32.5	29.4	29.0	63.0	25.2	0.60	3.3	220
33.0	29.4	29.0	63.0	25.2	0.60	3.8	237

DOWNSTREA

Res. Elev,		3. Flow under or through middle railing						
		Crest El. Ft	breadth, ft	Length, ft	h/Lb	c for 3/4:1	Head, ft	Q=clh^1.5, cfs
		Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5
25.6								
25.7								
25.8								
25.9								
26.0								
26.1								
26.2								
26.3								
26.4								
26.5								
26.6								
26.7								
26.8								
26.9								
27.0								
27.1								
27.2								
27.3								
27.4								
27.5								
27.6								
27.7								
27.8								
27.9								
28.0								
28.1								
28.2								
28.3								
28.4								
28.5								
28.6								
28.7								
28.8								
28.9								
29.0								
29.1								
29.2								
29.3								
29.4								
29.5								
29.6		29.6	0.2	63.0	0.00	2.63	0.0	0
29.7		29.6	0.2	63.0	0.48	2.69	0.1	5
29.8		29.6	0.2	63.0	0.96	2.97	0.2	17
29.9		29.6	0.2	63.0	1.44	3.27	0.3	34
30.0		29.6	0.2	63.0	1.92	3.50	0.4	56
30.1		29.6	0.2	63.0	2.40	3.65	0.5	81
30.2		29.6	0.2	63.0	2.88	3.65	0.6	107
30.3		29.6	0.2	63.0	3.36	3.65	0.7	135
30.4		29.6	0.2	63.0	3.84	3.65	0.8	165
30.5		29.6	0.2	63.0	4.32	3.65	0.9	196
30.6		29.6	0.2	63.0	4.80	3.65	1.0	230
30.7		29.6	0.2	63.0	5.28	3.65	1.1	265
30.8		29.6	0.2	63.0	5.76	3.65	1.2	302
30.9		29.6	0.2	63.0	6.24	3.65	1.3	341
31.0	6.5	30.9	29.6	63.0	81.9	0.60	0.8	342
31.1		30.9	29.6	63.0	81.9	0.60	0.9	364
31.2		30.9	29.6	63.0	81.9	0.60	1.0	384
31.3		30.9	29.6	63.0	81.9	0.60	1.1	404
31.4		30.9	29.6	63.0	81.9	0.60	1.2	423
31.5		30.9	29.6	63.0	81.9	0.60	1.3	441
31.6		30.9	29.6	63.0	81.9	0.60	1.4	458
31.7		30.9	29.6	63.0	81.9	0.60	1.5	475
31.8		30.9	29.6	63.0	81.9	0.60	1.6	491
31.9		30.9	29.6	63.0	81.9	0.60	1.7	507
32.0		30.9	29.6	63.0	81.9	0.60	1.8	522
32.1		30.9	29.6	63.0	81.9	0.60	1.9	536
32.2		30.9	29.6	63.0	81.9	0.60	2.0	551
32.3		30.9	29.6	63.0	81.9	0.60	2.1	565
32.4		30.9	29.6	63.0	81.9	0.60	2.2	578
32.5		30.9	29.6	63.0	81.9	0.60	2.3	592
33.0		30.9	29.6	63.0	81.9	0.60	2.8	654

DOWNSTREA

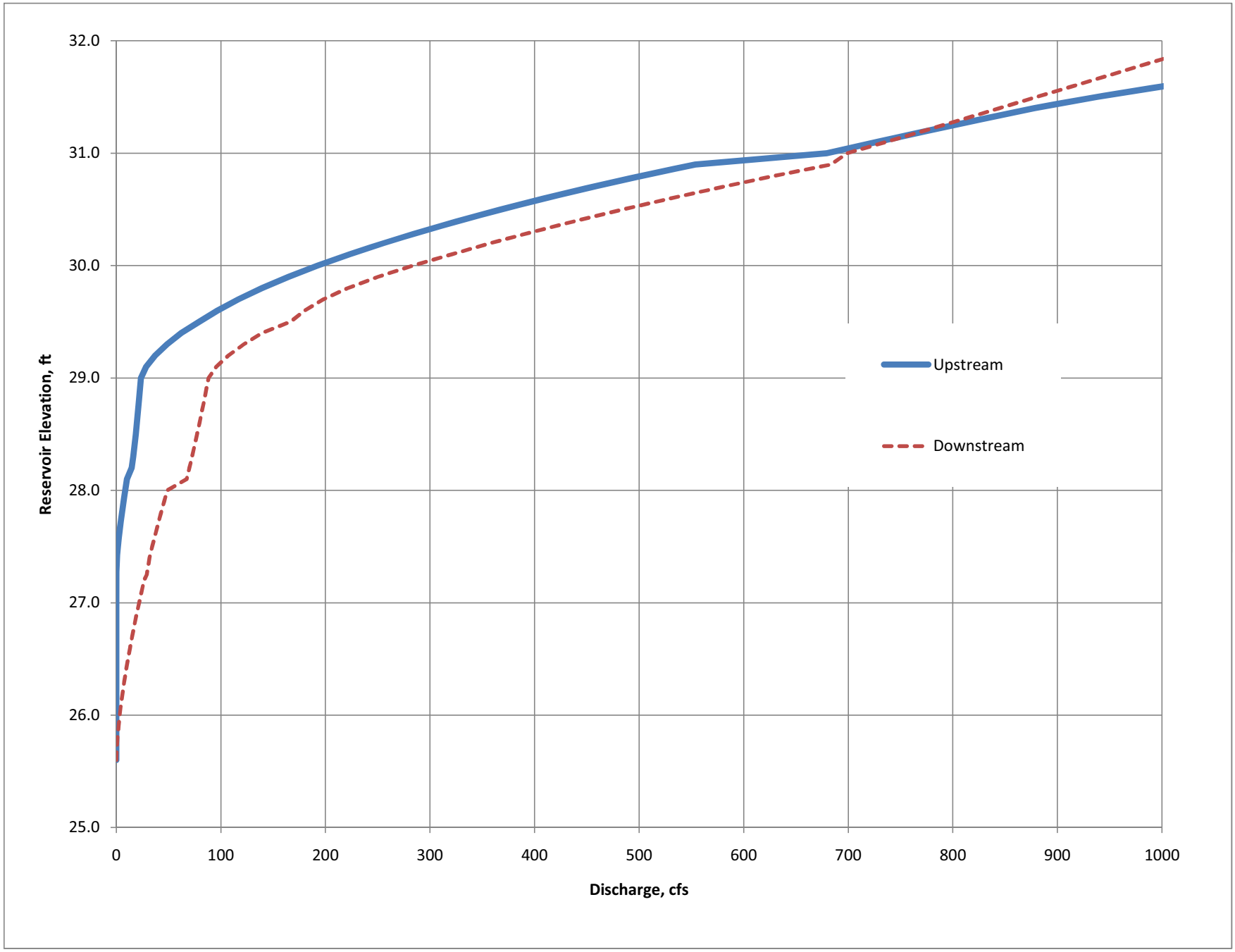
Res. Elev,		4. Flow thru existing walkway							
		Crest El. Ft	breadth, ft	Length, ft	h/Lb	c for 3/4:1	Head, ft	Q=clh^1.5, cfs	
		Top Open	Bottom Open	Length, ft	Area	orifice c	Head, ft	Q=cA(2gh)^0.5	RIGHT SIDE ONLY
25.6									
25.7									
25.8									
25.9									
26.0									
26.1									
26.2									
26.3									
26.4									
26.5									
26.6									
26.7									
26.8									
26.9									
27.0									
27.1									
27.2									
27.3									
27.4									
27.5									
27.6									
27.7									
27.8									
27.9									
28.0									
28.1									
28.2									
28.3									
28.4									
28.5									
28.6									
28.7									
28.8									
28.9									
29.0									0
29.1									0
29.2									0
29.3									1
29.4									1
29.5		29.5	20.0	6.0	0.00	2.63	0.0	0	2
29.6		29.5	20.0	6.0	0.01	2.63	0.1	0	4
29.7		29.5	20.0	6.0	0.01	2.63	0.2	1	5
29.8		29.5	20.0	6.0	0.02	2.63	0.3	3	7
29.9		29.5	20.0	6.0	0.02	2.63	0.4	4	10
30.0		29.5	20.0	6.0	0.03	2.63	0.5	6	13
30.1		29.5	20.0	6.0	0.03	2.63	0.6	7	16
30.2		29.5	20.0	6.0	0.04	2.63	0.7	9	20
30.3		29.5	20.0	6.0	0.04	2.63	0.8	11	25
30.4		29.5	20.0	6.0	0.05	2.63	0.9	13	29
30.5		29.5	20.0	6.0	0.05	2.63	1.0	16	35
30.6		29.5	20.0	6.0	0.06	2.63	1.1	18	41
30.7		29.5	20.0	6.0	0.06	2.63	1.2	21	48
30.8		29.5	20.0	6.0	0.07	2.63	1.3	23	55
30.9		29.5	20.0	6.0	0.07	2.63	1.4	26	63
31.0	4.17	29.5	20.0	6.0	0.08	2.63	1.5	29	72
31.1		29.5	20.0	6.0	0.08	2.63	1.6	32	81
31.2		29.5	20.0	6.0	0.09	2.63	1.7	35	91
31.3		29.5	20.0	6.0	0.09	2.63	1.8	38	102
31.4		29.5	20.0	6.0	0.10	2.63	1.9	41	113
31.5		29.5	20.0	6.0	0.10	2.63	2.0	45	126
31.6		29.5	20.0	6.0	0.11	2.63	2.1	48	139
31.7		29.5	20.0	6.0	0.11	2.63	2.2	51	152
31.8		29.5	20.0	6.0	0.12	2.63	2.3	55	167
31.9		29.5	20.0	6.0	0.12	2.63	2.4	59	182
32.0		29.5	20.0	6.0	0.13	2.63	2.5	62	198
32.1		29.5	20.0	6.0	0.13	2.63	2.6	66	215
32.2		29.5	20.0	6.0	0.14	2.63	2.7	70	233
32.3		29.5	20.0	6.0	0.14	2.63	2.8	74	252
32.4		29.5	20.0	6.0	0.15	2.63	2.9	78	271
32.5		29.5	20.0	6.0	0.15	2.63	3.0	82	291
33.0		29.5	20.0	6.0	0.18	2.63	3.5	103	382

DOWNSTREA

Res. Elev,	TOTAL	ELEV
25.6	0	25.60
25.7	0	25.70
25.8	1	25.80
25.9	2	25.90
26.0	3	26.00
26.1	5	26.10
26.2	6	26.20
26.3	8	26.30
26.4	9	26.40
26.5	11	26.50
26.6	13	26.60
26.7	15	26.70
26.8	17	26.80
26.9	19	26.90
27.0	22	27.00
27.1	24	27.10
27.2	27	27.20
27.3	29	27.30
27.4	32	27.40
27.5	34	27.50
27.6	37	27.60
27.7	40	27.70
27.8	43	27.80
27.9	46	27.90
28.0	49	28.00
28.1	67	28.10
28.2	70	28.20
28.3	72	28.30
28.4	75	28.40
28.5	77	28.50
28.6	80	28.60
28.7	82	28.70
28.8	84	28.80
28.9	86	28.90
29.0	88	29.00
29.1	96	29.10
29.2	107	29.20
29.3	122	29.30
29.4	139	29.40
29.5	167	29.50
29.6	180	29.60
29.7	198	29.70
29.8	221	29.80
29.9	250	29.90
30.0	284	30.00
30.1	321	30.10
30.2	358	30.20
30.3	398	30.30
30.4	440	30.40
30.5	485	30.50
30.6	531	30.60
30.7	580	30.70
30.8	631	30.80
30.9	683	30.90
31.0	699	31.00
31.1	736	31.10
31.2	773	31.20
31.3	809	31.30
31.4	845	31.40
31.5	880	31.50
31.6	916	31.60
31.7	952	31.70
31.8	987	31.80
31.9	1023	31.90
32.0	1060	32.00
32.1	1096	32.10
32.2	1133	32.20
32.3	1170	32.30
32.4	1208	32.40
32.5	1246	32.50
33.0	1422	33.00

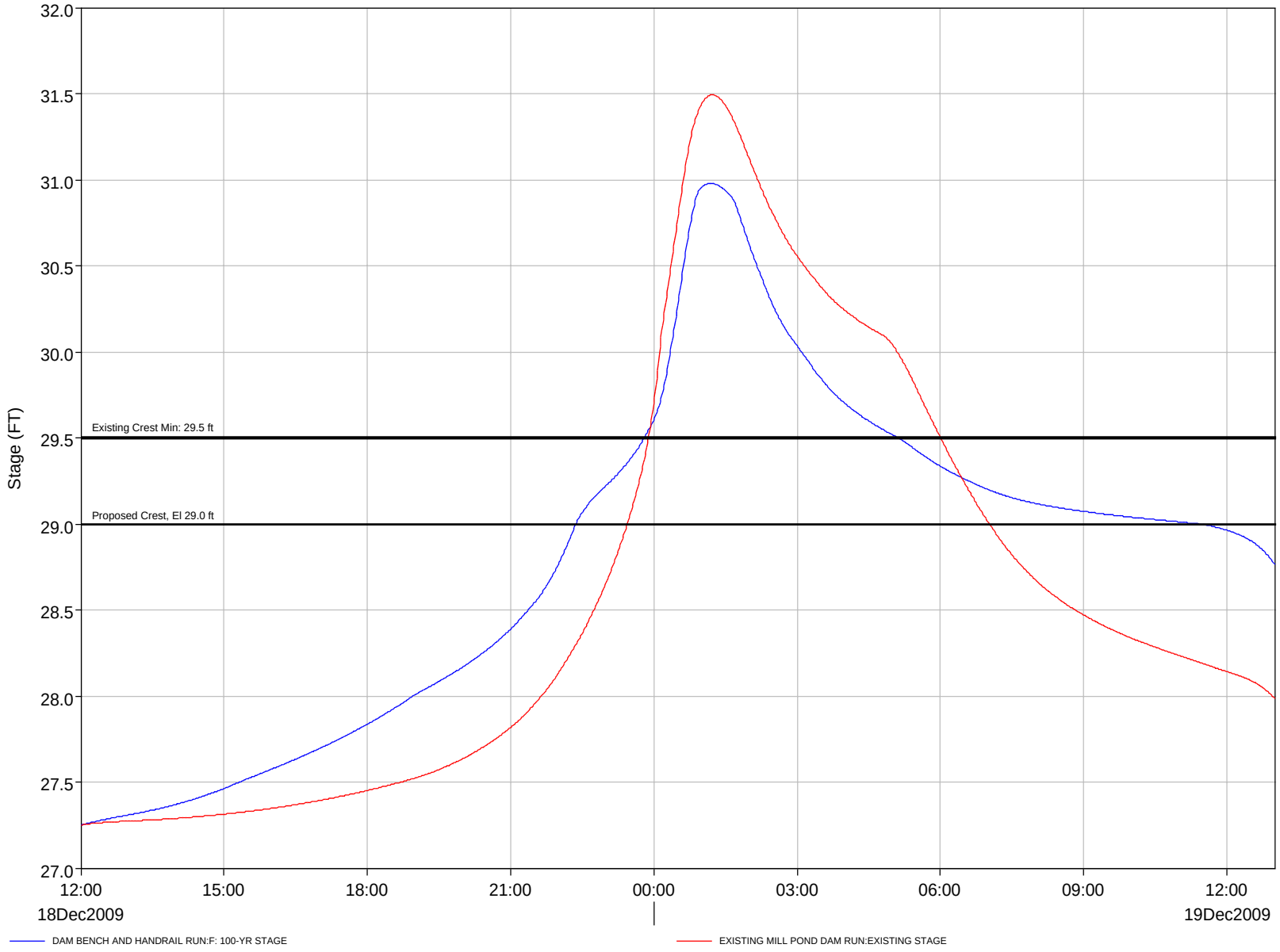
Summar of Discharge Rating Curve Calcs: Mill Pond Dam

	CONTROL	FLOW, CFS	
RES. Elev	(MIN CFS)	Upstream	Downstream
25.6	0	0	0
25.7	0.4	0	0
25.8	1	1	1
25.9	2	2	2
26.0	3	3	3
26.1	5	5	5
26.2	6	6	6
26.3	8	8	8
26.4	9	9	9
26.5	11	11	11
26.6	13	13	13
26.7	15	15	15
26.8	17	17	17
26.9	19	19	19
27.0	22	22	22
27.1	24	24	24
27.2	27	27	27
27.3	28	28	29
27.4	32	32	32
27.5	34	34	34
27.6	37	37	37
27.7	40	40	40
27.8	43	43	43
27.9	46	46	46
28.0	49	49	49
28.1	52	52	67
28.2	55	55	70
28.3	58	58	72
28.4	62	62	75
28.5	65	65	77
28.6	80	87	80
28.7	82	90	82
28.8	84	92	84
28.9	86	95	86
29.0	88	97	88
29.1	96	104	96
29.2	107	114	107
29.3	122	127	122
29.4	139	142	139
29.5	161	161	167
29.6	180	180	180
29.7	198	201	198
29.8	221	225	221
29.9	250	252	250
30.0	281	281	284
30.1	313	313	321
30.2	347	347	358
30.3	384	384	398
30.4	422	422	440
30.5	464	464	485
30.6	507	507	531
30.7	554	554	580
30.8	602	602	631
30.9	654	654	683
31.0	699	781	699
31.1	736	830	736
31.2	773	880	773
31.3	809	931	809
31.4	845	984	845
31.5	880	1045	880
31.6	916	1113	916
31.7	952	1186	952
31.8	987	1264	987
31.9	1023	1347	1023
32.0	1060	1434	1060
32.1	1096	1525	1096
32.2	1133	1621	1133
32.3	1170	1721	1170
32.4	1208	1825	1208
32.5	1246	1934	1246
33.0	1422	2478	1422


32.0
31.0
30.0
29.0
28.0
27.0
26.0
25.0
Reservoir Elevation, ft
0
100
200
300
400
500
600
700
800
900
1000
Discharge, cfs
Upstream
Downstream

Attachment 8

HEC-HMS Results


32.0
31.5
31.0
30.5
30.0
29.5
29.0
28.5
28.0
27.5
27.0
Stage (FT)
Existing Crest Min: 29.5 ft
Proposed Crest, El 29.0 ft
12:00
15:00
18:00
21:00
00:00
03:00
06:00
09:00
12:00
18Dec2009
19Dec2009
DAM BENCH AND HANDRAIL RUN:F: 100-YR STAGE
EXISTING MILL POND DAM RUN:EXISTING STAGE

8a – Existing Conditions

Project: Rockport Final

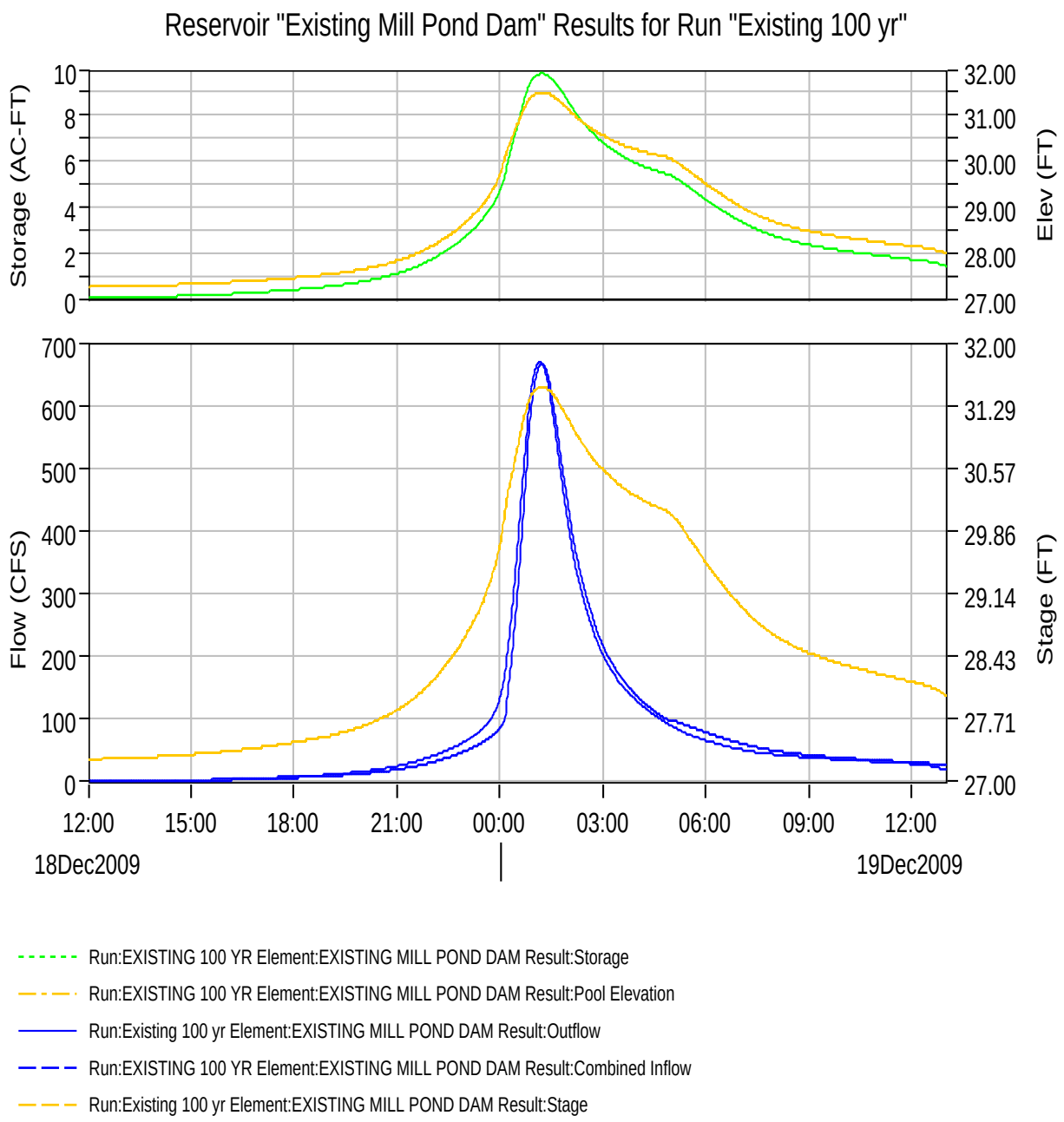
Simulation Run: Existing 100 yr Reservoir: Existing Mill Pond Dam

Start of Run:	18Dec2009, 12:00	Basin Model:	Existing
End of Run:	19Dec2009, 13:00	Meteorologic Model:	100-year, 24-hour storm
Compute Time:	17Feb2010, 09:41:53	Control Specifications:	Control 1

Volume Units: IN

Computed Results

Peak Inflow :	669.3 (CFS)	Date/Time of Peak Inflow :	19Dec2009, 01:10
Peak Outflow :	666.8 (CFS)	Date/Time of Peak Outflow :	19Dec2009, 01:14
Total Inflow :	4.33 (IN)	Peak Storage :	9.8 (AC-FT)
Total Outflow :	4.30 (IN)	Peak Elevation :	31.5 (FT)


Reservoir "Existing Mill Pond Dam" Results for Run "Existing 100 yr"
Storage (AC-FT)
Elev (FT)
Flow (CFS)
Stage (FT)
10
8
6
4
2
0
32.00
31.00
30.00
29.00
28.00
27.00
700
600
500
400
300
200
100
0
32.00
31.29
30.57
29.86
29.14
28.43
27.71
27.00
12:00
15:00
18:00
21:00
00:00
03:00
06:00
09:00
12:00
18Dec2009
19Dec2009
Run:EXISTING 100 YR Element:EXISTING MILL POND DAM Result:Storage
Run:EXISTING 100 YR Element:EXISTING MILL POND DAM Result:Pool Elevation
Run:Existing 100 yr Element:EXISTING MILL POND DAM Result:Outflow
Run:EXISTING 100 YR Element:EXISTING MILL POND DAM Result:Combined Inflow
Run:Existing 100 yr Element:EXISTING MILL POND DAM Result:Stage

8b – Proposed Conditions

Project: Rockport Final

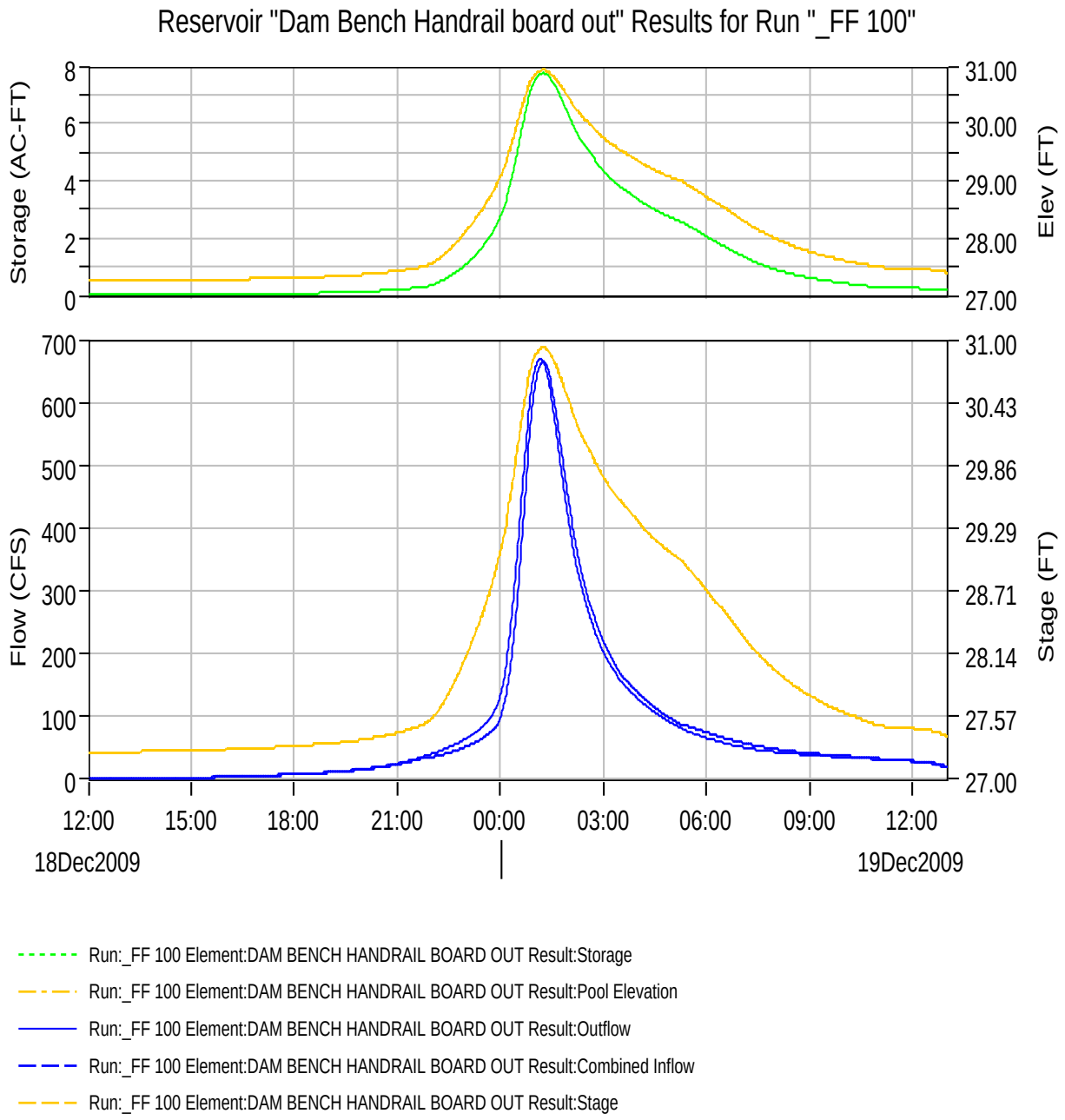
Simulation Run: _FF 100 Reservoir: Dam Bench Handrail board out

Start of Run:	18Dec2009, 12:00	Basin Model:	Mill Pond Dam Final boardout
End of Run:	19Dec2009, 13:00	Meteorologic Model:	100-year, 24-hour storm
Compute Time:	17Feb2010, 09:40:53	Control Specifications:	Control 1

Volume Units: IN

Computed Results

Peak Inflow :	669.3 (CFS)	Date/Time of Peak Inflow :	19Dec2009, 01:10
Peak Outflow :	664.6 (CFS)	Date/Time of Peak Outflow :	19Dec2009, 01:15
Total Inflow :	4.33 (IN)	Peak Storage :	7.8 (AC-FT)
Total Outflow :	4.33 (IN)	Peak Elevation :	30.9 (FT)


Reservoir "Dam Bench Handrail board out" Results for Run "_FF 100"
Storage (AC-FT)
Elev (FT)
Flow (CFS)
Stage (FT)
8
6
4
2
0
31.00
30.00
29.00
28.00
27.00
700
600
500
400
300
200
100
0
31.00
30.43
29.86
29.29
28.71
28.14
27.57
27.00
12:00
15:00
18:00
21:00
00:00
03:00
06:00
09:00
12:00
18Dec2009
19Dec2009
Run:_FF 100 Element:DAM BENCH HANDRAIL BOARD OUT Result:Storage
Run:_FF 100 Element:DAM BENCH HANDRAIL BOARD OUT Result:Pool Elevation
Run:_FF 100 Element:DAM BENCH HANDRAIL BOARD OUT Result:Outflow
Run:_FF 100 Element:DAM BENCH HANDRAIL BOARD OUT Result:Combined Inflow
Run:_FF 100 Element:DAM BENCH HANDRAIL BOARD OUT Result:Stage

Project: Rockport Final

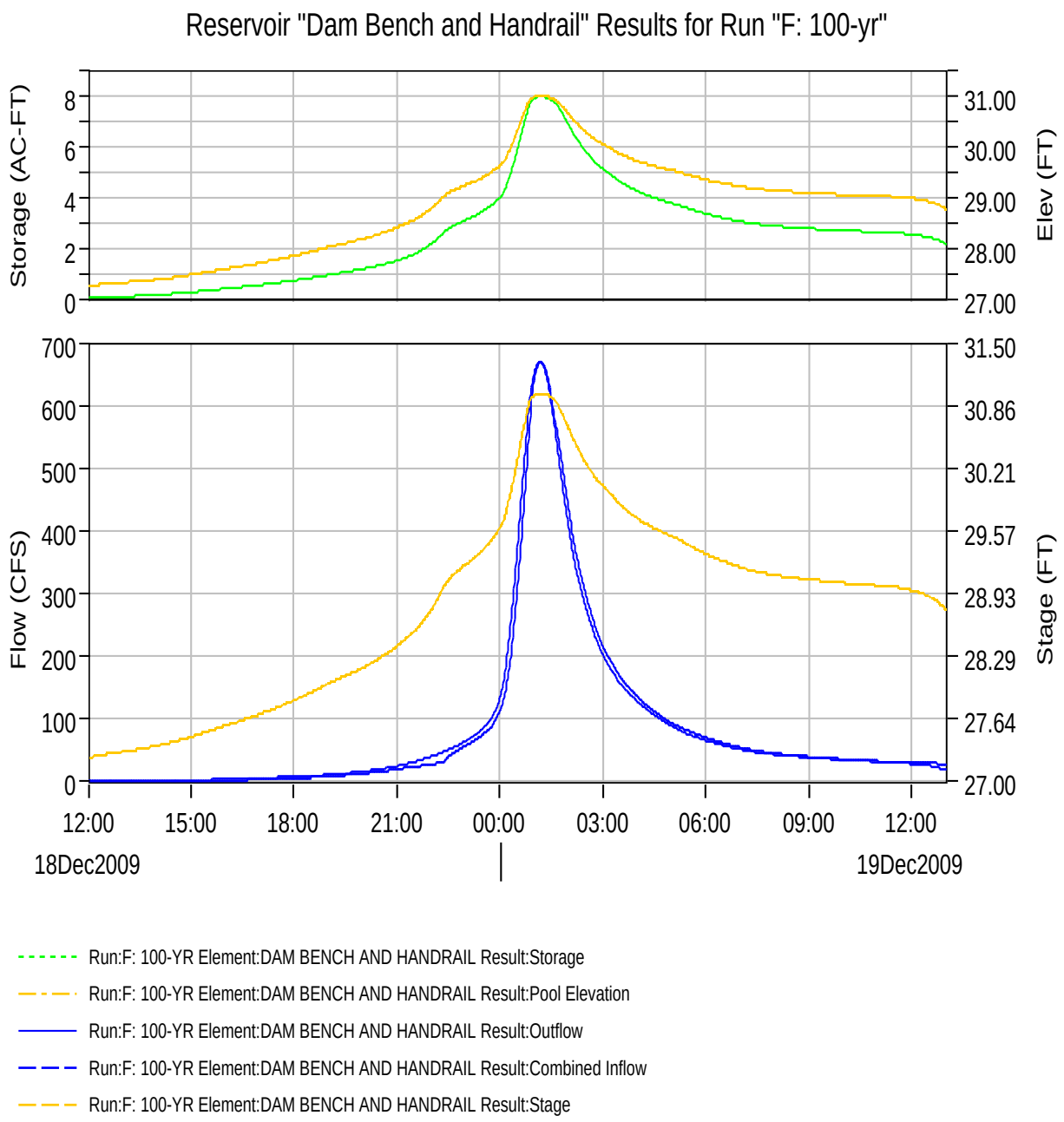
Simulation Run: F: 100-yr Reservoir: Dam Bench and Handrail

Start of Run:	18Dec2009, 12:00	Basin Model:	Mill Pond Dam Final
End of Run:	19Dec2009, 13:00	Meteorologic Model:	100-year, 24-hour storm
Compute Time:	17Feb2010, 09:41:51	Control Specifications:	Control 1

Volume Units: IN

Computed Results

Peak Inflow :	669.3 (CFS)	Date/Time of Peak Inflow :	19Dec2009, 01:10
Peak Outflow :	668.6 (CFS)	Date/Time of Peak Outflow :	19Dec2009, 01:12
Total Inflow :	4.33 (IN)	Peak Storage :	7.9 (AC-FT)
Total Outflow :	4.28 (IN)	Peak Elevation :	31.0 (FT)


Reservoir "Dam Bench and Handrail" Results for Run "F: 100-yr"
Storage (AC-FT)
Elev (FT)
Flow (CFS)
Stage (FT)
0
2
4
6
8
27.00
28.00
29.00
30.00
31.00
100
200
300
400
500
600
700
27.64
28.29
28.93
29.57
30.21
30.86
31.50
12:00
15:00
18:00
21:00
00:00
03:00
06:00
09:00
12:00
18Dec2009
19Dec2009
Run:F: 100-YR Element:DAM BENCH AND HANDRAIL Result:Storage
Run:F: 100-YR Element:DAM BENCH AND HANDRAIL Result:Pool Elevation
Run:F: 100-YR Element:DAM BENCH AND HANDRAIL Result:Outflow
Run:F: 100-YR Element:DAM BENCH AND HANDRAIL Result:Combined Inflow
Run:F: 100-YR Element:DAM BENCH AND HANDRAIL Result:Stage

Appendix F

Technical Memo 3 – Seepage Analyses

Memo Title: Technical Memo 3 – Seepage Analyses

Project Title: Mill Pond Dam – Final Design Report

Project Location: Rockport, Massachusetts

Project #: 07104-1

By: M. Quinn **Date:** 5-4-2010

Checked by: J. DOMINGUEZ **Date:** 5-4-2010

TECHNICAL MEMORANDUM 3

Summary of Seepage Analysis
Mill Pond Dam
Rockport, MA
GEI Project #07104-1

The purpose of this technical memorandum is to summarize the seepage analyses and soil parameters used in seepage analyses for the proposed design of Mill Pond Dam located in Rockport, Massachusetts. All elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Our seepage analyses indicate that the proposed wall design for the Mill Pond Dam satisfies the US Army Corps of Engineers (USACE) recommended maximum exit gradients.

1.0 Evaluation of Cross Section Geometry

We performed seepage analyses at the former sluiceway cross section near the west (left abutment) at Mill Pond Dam. We used Drawing No. S-03 of the Mill Pond Dam Safety Modifications and Fig.1 (attached) to establish the generalized cross section geometry and soil profile for the seepage analyses. The two cross sections we selected represent portions of the dam where the toe of the footing must be shorter to allow for protection of existing features, namely, a dry-laid stone wall and a granite slab façade. In addition to the shorter footing toe, the preservation of these features does not allow for the placement of a graded filter downstream of the footing. In the typical cross section, seepage flows are conveyed through a longer path under the footing and exit into a graded filter that is weighed down by riprap. In our opinion, if gradients in the cross sections we analyzed are acceptable, then the gradients in the typical cross section will be acceptable by inspection.

In our seepage models, we assumed that the wall key will be embedded in glacial till. The pond side of the wall will be backfilled to El. 29 with embankment fill containing at least 20 percent fines. The bottom of the pond is assumed to be at El. 20 and the downstream channel is assumed to be at El. 16.

We evaluated two different seepage pathways. The first model represents a section through the former sluiceway area where the seepage path from the pond side of the dam to the downstream side of the dam is the shortest. The water on the pond side must flow below the key, into the filter sand layer below the footing, and out through a dry-laid stone wall. The second model represents a section through the former sluiceway area that has a granite slab façade. Therefore, the water on the pond side of the dam must flow under the key, into the filter sand layer below the footing and then along the dam axis to bypass the granite slab façade. These geometries are shown in Fig. 1.

To model the shorter flow path, we created a filter section consisting of a nine-inch-thick filter sand layer directly below the footing extending from the downstream face of the key and along the bottom of the footing. We modeled the dry-laid stone wall with a highly pervious layer. To model the longer seepage flow path, we lengthened the filter section to extend 35 feet beyond the downstream face of the key, and capped the filter with a relatively impervious layer to prevent outward flow and force the flow to follow a longer flow path to the downstream side of the dam.

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Woburn, MA 01801
781.721.4000 fax 781. 721.4073

2.0 Soil Parameters

We selected the soil properties for the seepage analyses based on our review of boring logs and typical values found in the Federal Highway Administration (FHWA) Manual HI-97-021 (see Attachment 2). The soil properties used in our analyses are summarized in the table below.

Soil Type	Unit Weight (pcf)	Hydraulic Conductivity (feet/day)	Saturated Vol. Water Content	Residual Vol. Water Content
Filter Sand	120	100	0.44	0.04
Embankment Fill	130	1.5	0.44	0.03
Glacial Till	135	0.8	0.31	0.03
Rip-Rap	NA	10,000	0.44	0.04
Existing Stone Wall	NA	10,000	0.44	0.04

We assumed that concrete had a very low hydraulic conductivity of 2.8×10^{-7} feet/day.

We assumed that the glacial till was anisotropic such that the $k_v/k_h = 0.25$.

Volumetric water content calculations are provided in Attachment 2.

3.0 Seepage Analysis Approach

We performed the seepage analyses for the following conditions:

- Steady state when the pond pool is at the flood level (El. 31).
- Steady state when the pond pool is at the normal pool (El. 27.3).

We used the computer program SEEP/W (GEO-SLOPE International, Ltd.) to compute the phreatic surface and pore pressures within the embankment and foundation soils, and to compute the exit gradients near the downstream toe of the dam.

For both cases, we assumed that steady-state seepage developed within the embankments and foundations, as recommended in EM 1110-2-1913, and Engineer Technical Letter (ETL) 1110-2-569.

3.1 Modeling Approach

We applied no-flow boundary conditions along the bottom.

We applied a constant total head boundary condition equal to the pond water elevation along the pond-side of the dam embankment and ground surface and the pond-side vertical edge of the model.

We applied a constant total head boundary condition along the downstream vertical edge of the model equal to the landside ground surface elevation (El. 16).

We applied a potential seepage face boundary condition to the downstream side of the dam and the downstream ground surface.

We performed both flood case and normal pool case for the shorter and longer seepage paths.

4.0 Seepage Results

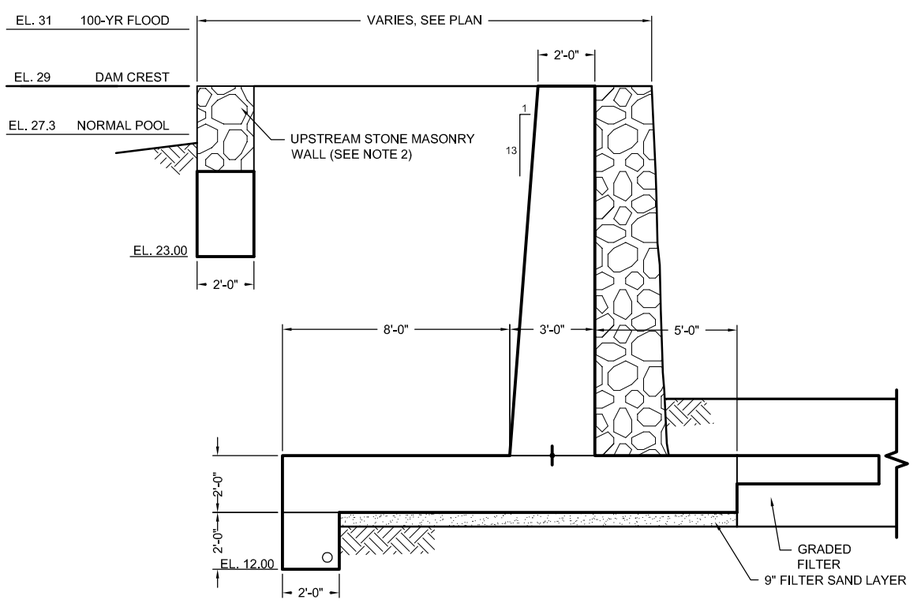
The maximum computed exit gradient for the shorter seepage path was approximately 0.5 for the flood case and approximately 0.3 for the normal pool case. Maximum computed exit gradient for the longer seepage path was approximately 0.3 for the flood case and approximately 0.2 for the normal pool case. The computed exit gradients were at or below the USACE recommended maximum limit of 0.5 for both sections for the 1-percent-annual-chance (100-year) flood. Our results are presented in Attachment 3.

5.0 References

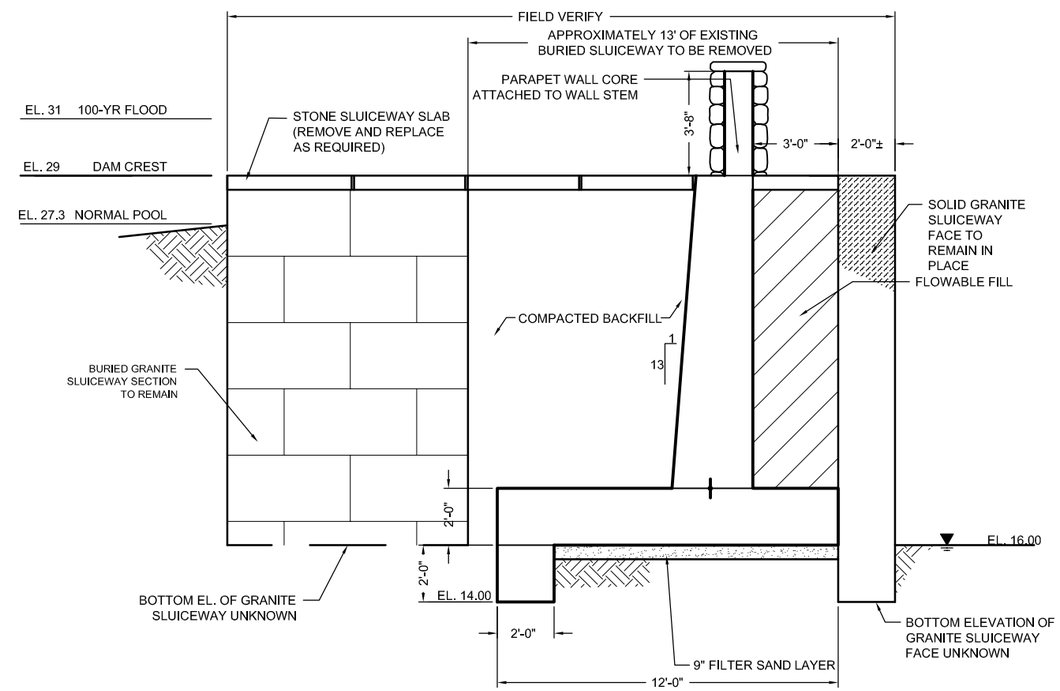
1. Federal Highway Administration (FHWA), Training Course in Geotechnical and Foundation Engineering: Subsurface Investigations – Participants Manual, FHWA Publication HI-97-021, 1997.
2. U.S. Army Corps of Engineers, Design and Construction of Levees, Engineer Manual EM 1110-2-1913, April 30, 2000.
3. U.S. Army Corps of Engineers, ETL 1110-2-569, Design Guidance for Levee Underseepage, May 1, 2005.

Attachment 1

Drawings



TYPICAL SECTION



SECTION AT SLUICEWAY

0 4 8

1/4" = 1'-0"

Mill Pond Dam Safety Modifications Seepage Analyses Rockport, Massachusetts	GEI Consultants	CROSS SECTIONS
Town of Rockport Rockport, Massachusetts	Project 07104-1	March 2010 Fig. 1

M:\DATA_DESIGN\071041 Rockport Mill Pond Dam\Ch253\TM3 - Seepage Analyses\Fig 1 for Seepage Calcs

Attachment 2

Soil Properties

Purpose:

Assign permeability properties to soils.

Hydraulic Conductivity

Typical published values of hydraulic conductivity (k) for various soil types are shown on the next page.

We used The Aberdeen Group (1989) publication to estimate the hydraulic conductivity of the concrete floodwalls. The article states that the permeability of concrete is a function of the permeability of the concrete paste, the gradation of the aggregate, and the relative proportion of the paste to the aggregate. We assumed the floodwall concrete is mature and good-quality and used a hydraulic conductivity of 1 E -10 cm/sec (2.8E-7 ft/day).

The hydraulic conductivities used for the SEEP/W model are shown on page 3.

FHWA HI-97-021, Subsurface Investigation, (1997), page 9-13

TABLE 9-10
TYPICAL PERMEABILITY VALUES FOR SOILS (After Carter and Bentley, 1991)

	10^{-11}	10^{-10}	10^{-9}	10^{-8}	10^{-7}	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}	1						
Coefficient of permeability (log scale)	m/s																	
	10^{-9}	10^{-8}	10^{-7}	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	10^{-1}	1	10	100						
	cm/s																	
Permeability:	Practically impermeable			Very low		Low		Medium			High							
Drainage conditions:	Practically impermeable			Poor				Good										
Typical soil groups:			GC → GM →			SM		SW →		GW →								
			CH	SC	SM-SC			SP →		GP →								
				MH														
				MC-CL														
Soil types:	Homogeneous clays below the zone of weathering			Silts, fine sands, silty sands, glacial till, stratified clays				Clean sands, sand and gravel mixtures				Clean gravels						
				Fissured and weathered clays and clays modified by the effects of vegetation														

Note: The arrow adjacent to group classes indicates that permeability values can be greater than the typical value shown.

Values for Analysis:

Based on the published typical values, we will use the following hydraulic conductivities:

Embankment Fill:	5.3E-04 cm/sec	= 1.5 ft/day
Glacial Till:	2.8E-04 cm/sec	= 0.8 ft/day
Filter Sand:	3.5E-02 cm/sec	= 100 ft/day
Rip Rap:	3.5 cm/sec	= 10,000 ft/day
Proposed Concrete Wall:	1.0E-10 cm/sec	= 2.8E-7 ft/day
Stone Wall:	3.5 cm/sec	= 10,000 ft/day

Volumetric Water Content

To calculate the saturated volumetric water content (θ_{SAT}) of each soil layer, we used estimated unit weight and specific gravity the following equation process, assuming V_T is 1 ft^3 and the soil is 100% saturated:

$$\gamma_T = \frac{W_T}{V_T} = \frac{W_S + W_W}{V_T} = \frac{(1 - V_W)\gamma_s + V_W\gamma_W}{1} = \gamma_s - V_W\gamma_s + V_W\gamma_W \rightarrow V_W = \frac{\gamma_T - \gamma_s}{\gamma_W - \gamma_s}$$

1. $\theta_{SAT} = \frac{V_W}{V_T} = \frac{V_W}{1ft^3}$
2. $\gamma_s = G_s\gamma_w$
3. $\gamma_T = \frac{W_T}{V_T} = \frac{W_S+W_W}{V_S+V_S} = \frac{(1-V_W)\gamma_s+V_W\gamma_W}{1-V_W+V_W} = \gamma_s - V_W\gamma_s + V_W\gamma_W$
4. $\gamma_T = \gamma_s - V_W\gamma_s + V_W\gamma_W$
5. $V_W = \frac{\gamma_T-\gamma_s}{\gamma_W-\gamma_s}$
6. So volumetric water content $\theta_{SAT} = \frac{V_W}{V_T} = \frac{V_W}{1ft^3} = \frac{\gamma_T-\gamma_s}{\gamma_W-\gamma_s}$

We assumed that the residual water content was 10% of the saturated volumetric water content. We will use the following volumetric water contents (ft^3/ft^3):

		θ_{sat}	θ_r
Saturated Volumetric Water Content	Filter Sand	0.44	0.04
	Embankment Sand	0.44	0.04
	Glacial Till	0.35	0.04

Volumetric Water Content Estimate:

Purpose: To estimate volumetric water contents (θ_w) at 100% saturation for seepage analyses.

Calculations:

1. Assumed specific gravity for soil types[1]:

Sand	G_{sand}=	2.65	
Silt	G_{silt}=	2.7	

2. Estimated unit weight of solids:

Water	γ_w=	62.4	pcf
Sand	$\gamma_{s\ sand}$=	165	pcf
Silt	$\gamma_{s\ silt}$=	168	pcf

$$\gamma_s = G_s \gamma_w$$

3. Assumed total unit weights of fill and alluvium soils:

Filter Sand[2] and Fill	$\gamma_{t\ sand}$=	120	pcf
Glacial Till[3]	$\gamma_{t\ till}$=	135	pcf

4. Estimated saturated volumetric water content (θ_{sat}):

Filter Sand and Fill	$\theta_{sat\ sand}$=	0.44	ft^3/ft^3
Glacial Till	$\theta_{sat\ till}$=	0.31	ft^3/ft^3

$$\theta_{sat} = \left(\frac{(\gamma_t - \gamma_s)}{(\gamma_w - \gamma_s)} \right)$$

5. Estimated residual volumetric water content (θ_r)[5]:

Filter Sand and Fill	$\theta_{r\ sand}$=	0.04	ft^3/ft^3
Glacial Till	$\theta_{r\ till}$=	0.03	ft^3/ft^3

Notes:

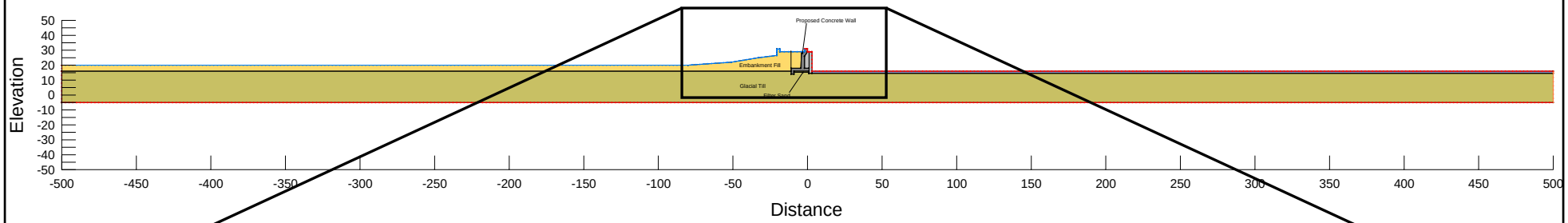
1. We used typical specific gravity values for sand (2.65) and silt (2.7) to estimate dry unit weight of soil.
2. Filter Sand will be placed below the proposed wall footing, and will contain less than 5% fines.
3. We estimated the unit weight for glacial till to be more dense than the filter sand and embankment fill. We assumed the glacial till was 50% sand and 50% silt when calculating the saturated volumetric water content.
4. We estimated the residual water content to be 10% of the saturated water content.

Attachment 3

Seepage Results

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Flood Case
Method: Steady-State
Last Modified: 3/11/2010 3:23:50 PM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
RipRap: K= 10000 ft/days,
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 10000 ft/days



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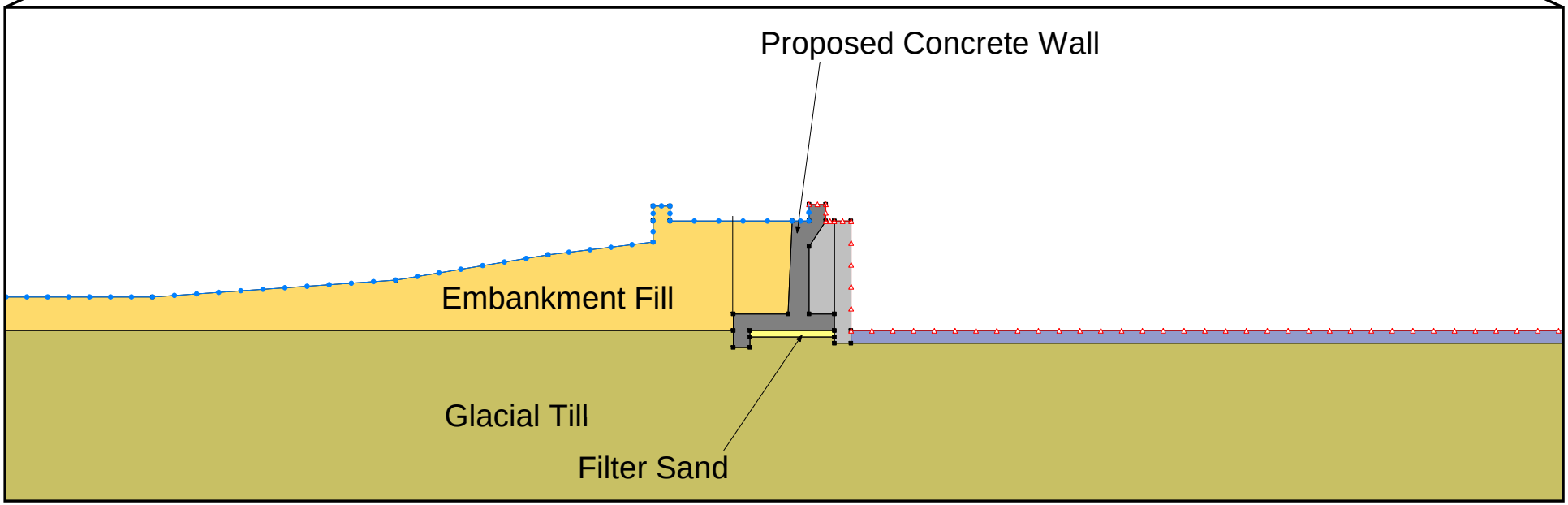
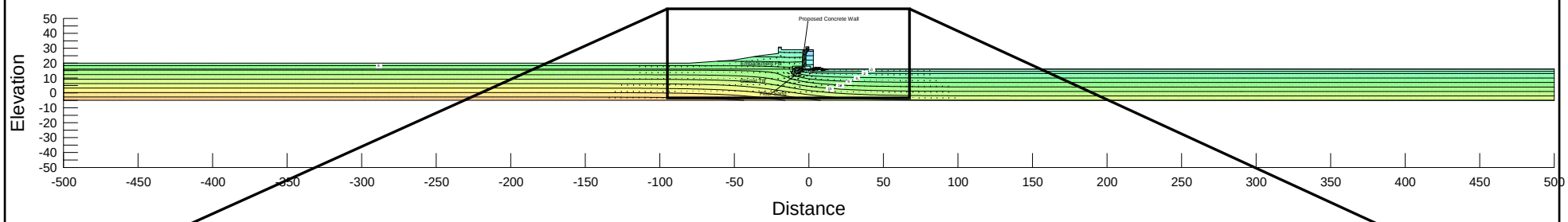


Notes:
1.Elevations reference to NGVD 1929.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W MODEL MILL POND DAM SHORT SEEPAGE PATH
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010 FIG. A-1

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Normal Pool
Method: Steady-State
Last Modified: 3/11/2010 11:50:15 AM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
RipRap: K= 10000 ft/days,
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 10000 ft/days



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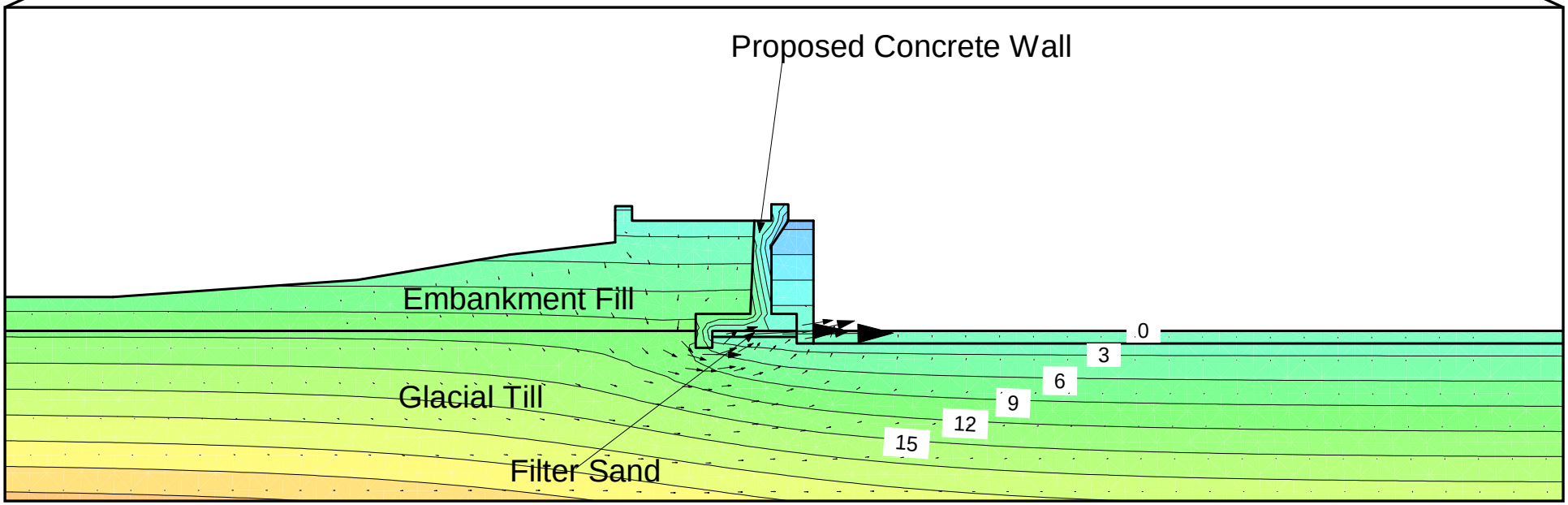


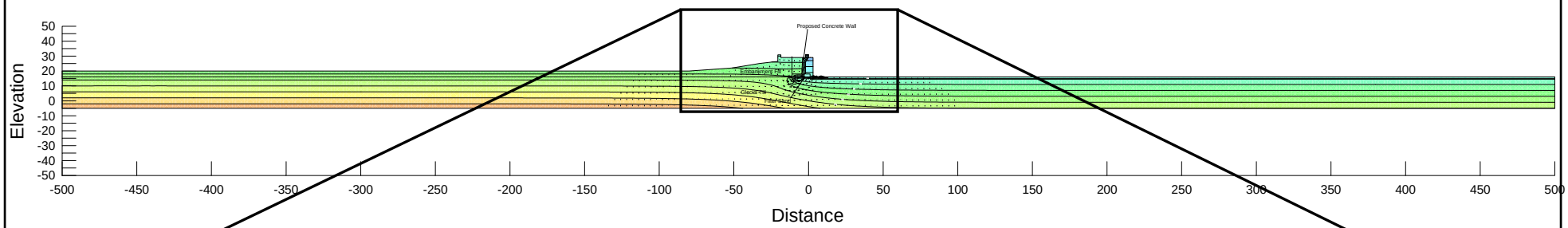
Notes:
1.Elevations referenced to NGVD 1929.
2.Contours represent feet of Pressure Head.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W STEADY-STATE MILL POND DAM SHORT SEEPAGE PATH NORMAL POOL
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010

FIG. A-2

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Flood Case
Method: Steady-State
Last Modified: 3/11/2010 8:38:32 AM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
RipRap: K= 10000 ft/days,
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 10000 ft/days



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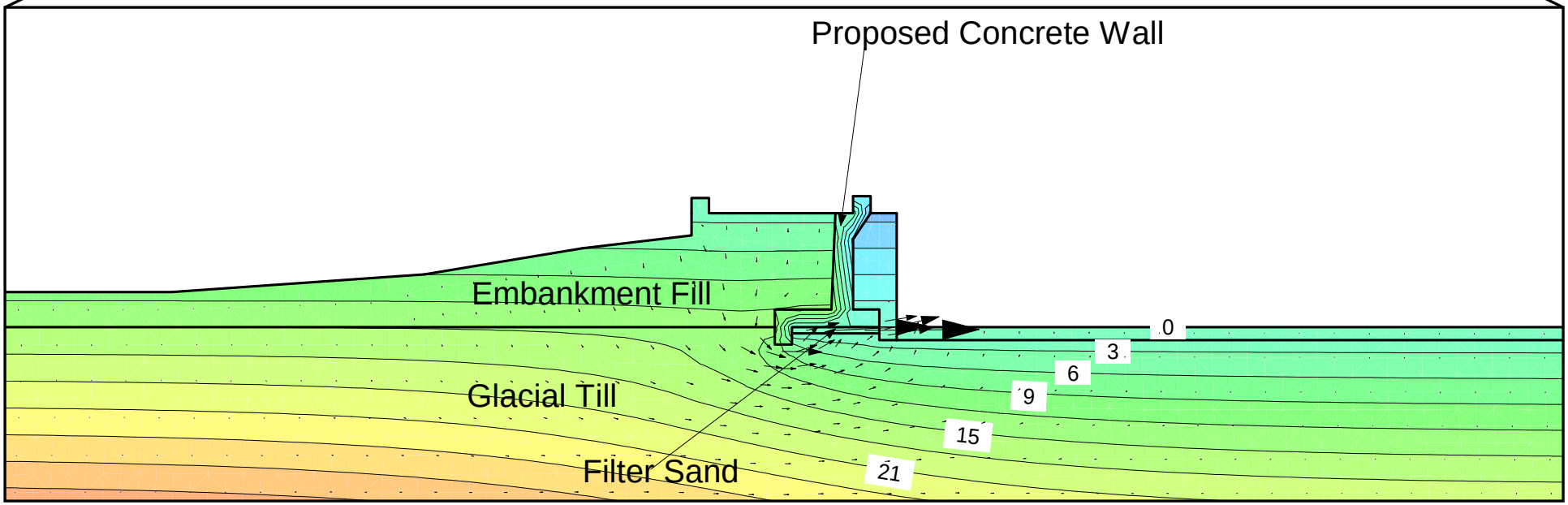
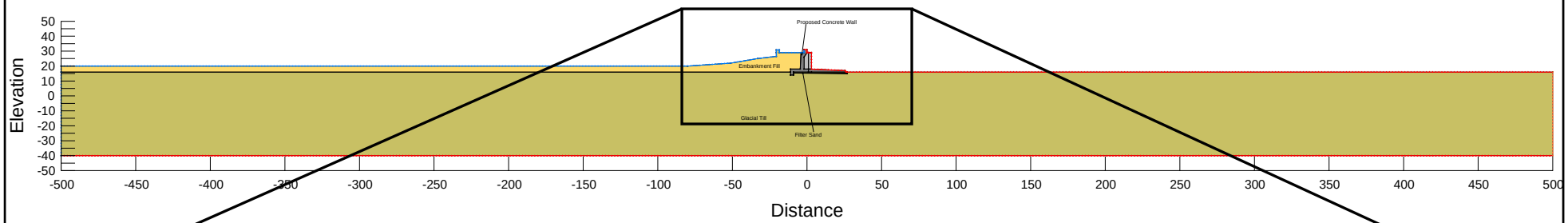


Notes:
1.Elevations referenced to NGVD 1929.
2.Contours represent feet of Pressure Head.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W STEADY-STATE MILL POND DAM SHORT SEEPAGE PATH 100-YEAR FLOOD POOL
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010 FIG. A-3

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Flood Case
Method: Steady-State
Last Modified: 4/29/2010 8:00:41 AM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 2.8e-007 ft/days



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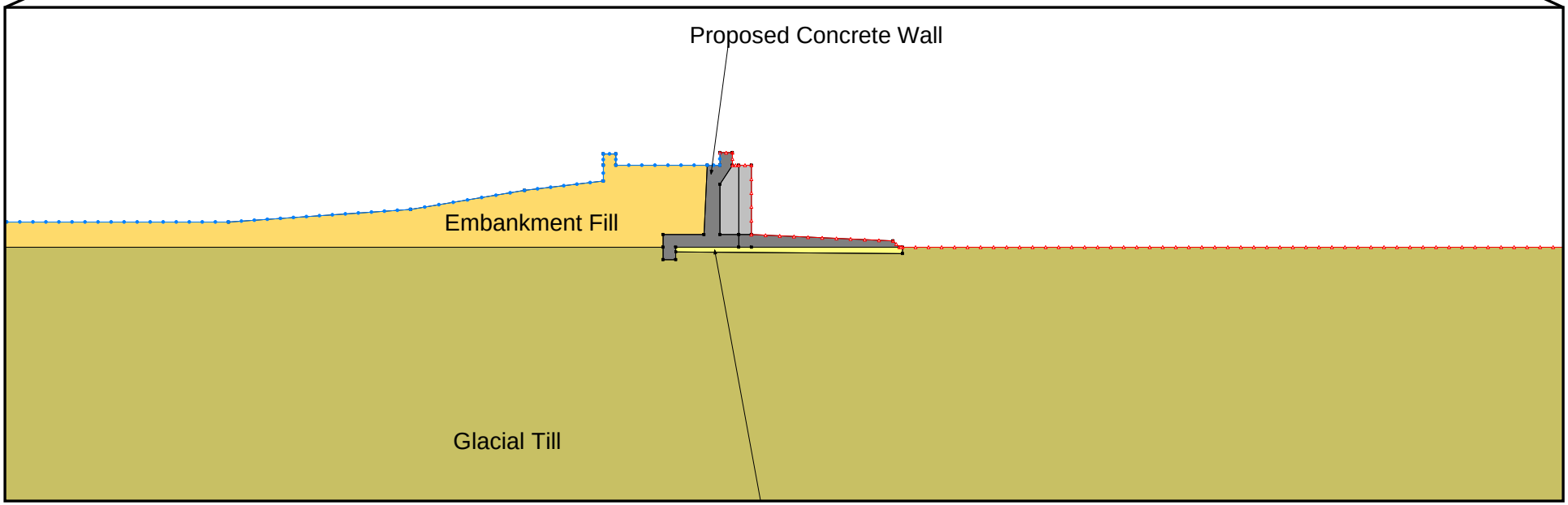
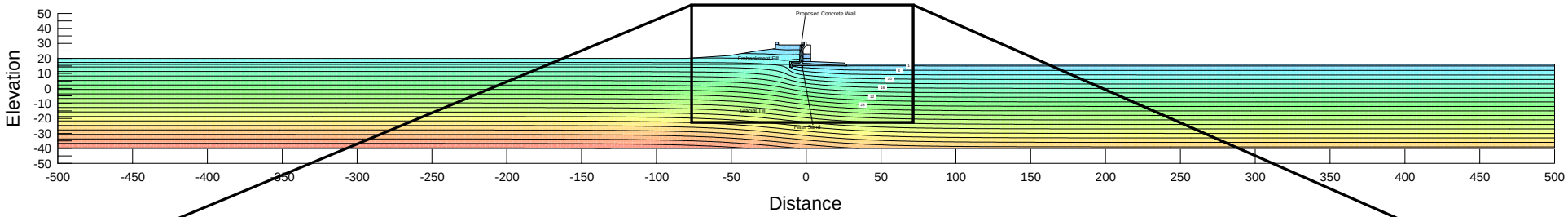


Notes:
1.Elevations referenced to NGVD 1929.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W MODEL MILL POND DAM LONG SEEPAGE PATH
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010 FIG. A-4

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Normal Pool
Method: Steady-State
Last Modified: 3/12/2010 8:37:00 AM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 2.8e-007 ft/days



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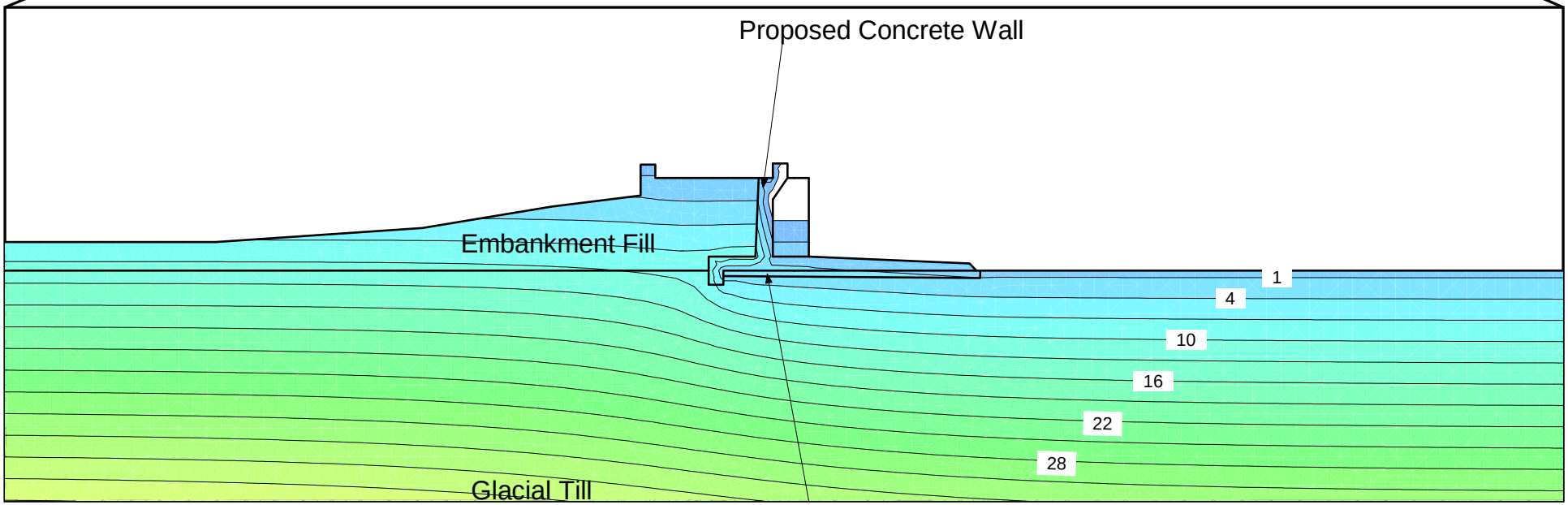


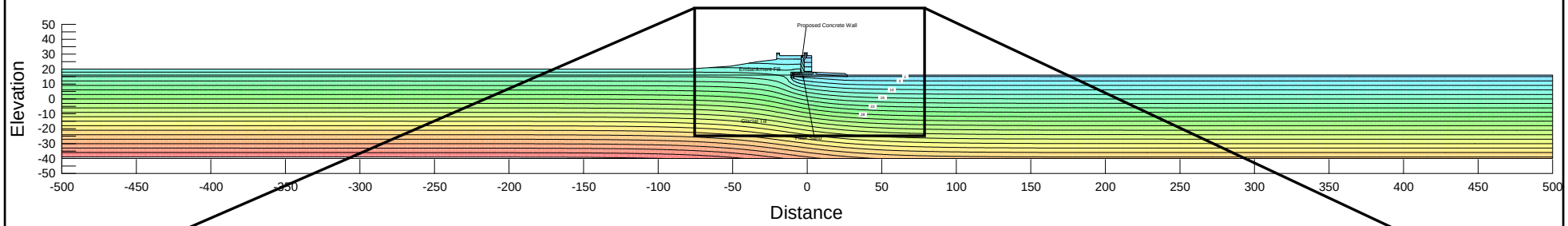
Notes:
1.Elevations referenced to NGVD 1929.
2.Contours represent feet of Pressure Head.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W STEADY-STATE MILL POND DAM LONG SEEPAGE PATH NORMAL POOL
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010

FIG. A-5

Mill Pond Dam - Sluice Gate Cross Section
Steady State Seepage - Flood Case
Method: Steady-State
Last Modified: 3/12/2010 8:37:00 AM

Embankment Fill: K= 1.5 ft/days, K Ratio = 1
Glacial Till: K= 0.8 ft/days, K Ratio = 0.25
Filter Sand: K= 100 ft/days, K Ratio = 1
Proposed Concrete Wall: K= 2.8e-007 ft/days
Stone Wall: K= 2.8e-007 ft/days



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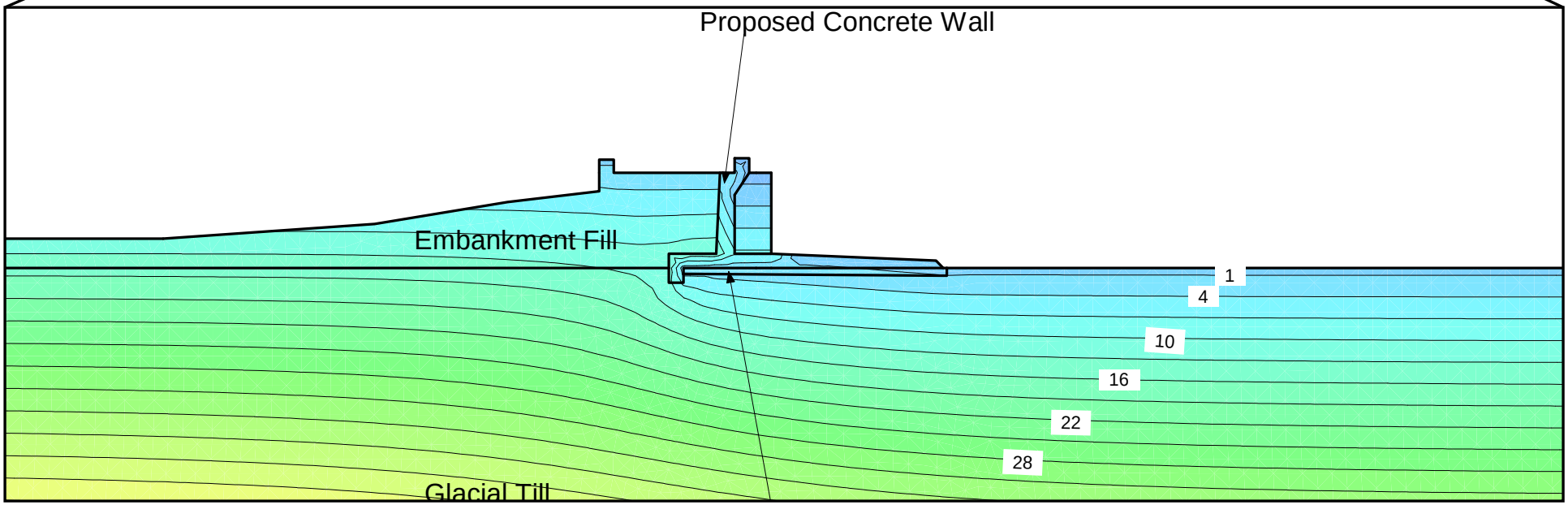

Notes:
1.Elevations referenced to NGVD 1929.
2.Contours represent feet of Pressure Head.

MILL POND DAM REMEDIAL DESIGN ROCKPORT, MASSACHUSETTS	GEI Consultants	SEEP/W STEADY-STATE MILL POND DAM LONG SEEPAGE PATH 100-YEAR FLOOD POOL
TOWN OF ROCKPORT ROCKPORT, MASSACHUSETTS	Project 07104-1	March 2010 FIG. A-6

Appendix G

Technical Memo 4 – Structural Analysis

Memo Title: Technical Memo 4 – Structural Analysis

Project Title: Mill Pond Dam – Final Design Report

Project Location: Rockport, Massachusetts

Project #: 07104-1

By: Max Hurd **Date:** 4/27/10

Checked by: PJ Snyder **Date:** 4/28/10

TECHNICAL MEMORANDUM # 4

Structural Design
Mill Pond Dam
Rockport, MA
GEI Project #07104-1

The purpose of this technical memorandum is to explain the stability analyses and structural design procedures employed in the design of the repair for Mill Pond Dam located in Rockport, Massachusetts.

The existing dam is earth core gravity dam with stone masonry walls on the upstream and downstream faces. The dam repair will replace the earth core with a reinforced concrete cantilevered retaining wall. The cantilevered retaining wall will consist of two standard sections designated by a bend in the wall. The eastern section will be approximately 83 feet long. The western section passes through a historical granite block sluiceway and will be approximately 23 feet long.

The granite block masonry facing will be reinstalled along the downstream face of the new cantilevered wall, and a new stone masonry wall will replace the existing upstream masonry wall to preserve the historic appearance of the dam.

1.0 Geotechnical Evaluation

Two borings were conducted within the footprint of the existing dam in May 2009. The existing ground surface elevation at the locations of the borings was approximately El. 30. The datum used for the project was the National Geodetic Vertical Datum 1929 (NGVD 29).

The finished grade on the downstream side of the dam will range from El. 18 to El. 21 in the eastern section and El. 15 to El. 16 ft in the western section. The elevation of the bottom of the wall footing was selected to provide frost protection. The bottom of footing elevations selected were El. 14 and El. 16 for the east and west wall sections respectively.

In the boring logs, the soils at these depths are typically described as Dense to Very Dense Silt, Fine to Coarse Sand, and Gravel (Glacial Till), with SPT N-Values ranging from 33 to greater than 100. In accordance with the Massachusetts Building Code, 7th Edition (780 CMR 120.R), the foundation soils were classified as Dense Class 7 or 8 soils. From the presumptive allowable bearing capacities in 780 CMR Table 1804.3, an allowable bearing pressure of 4 tons per square foot (8 kips per square foot) was selected.

From the liquefaction susceptibility chart of 780 CMR 1804.6, we concluded that the foundation materials are not susceptible to liquefaction.

2.0 Dam Repair Stability Analyses

The stability of the cantilevered retaining wall was analyzed using the gravity method in general accordance with Engineer Manual 1110-2-2502, Retaining and Flood Walls (U.S. Army Corps of Engineers). We analyzed two sections which represented the maximum sections at both the east and west wall.

400 Unicorn Park Drive
Woburn, MA 01801
781.721.4000 fax 781. 721.4073

Stability criteria were selected as specified in 302 CMR 10.14 (9k), and are summarized below:

Loading Condition	Factor of Safety
Usual	3.0
Unusual	2.0
Extreme	>1.0

Three load cases were analyzed for the cross-section, corresponding to various headwater and external loading conditions:

- Load Case 1 (Usual Condition) - Normal Pool
- Load Case 2 (Unusual Condition) - Design Flood (100 yr)
- Load Case 3 (Extreme Condition) - Normal Pool + Earthquake

We determined that calculations were not required for the winter pool + ice load because ice loads from the pond will be resisted by the upstream masonry wall, and therefore will never be in direct contact with the retaining wall. We also determined that calculations were not required for the Maximum drawdown load case as the normal pool load case is more critical by inspection.

302 CMR 10.14 does not provide any provisions for the percentage of the dam base in compression when considering moment equilibrium. However, since the dam repair design uses a cantilevered retaining wall we determined that overturning criteria for retaining walls from EM 1110-2-2502 (U.S. Army Corps of Engineers) should be used. The criteria are summarized below:

Loading Condition	Base Area in Compression (Overturning)
Usual	100%
Unusual	75%
Earthquake	Resultant within Base

We have the following remarks regarding the design loads and the retaining wall design analyses:

- The true sliding failure plane of the retaining wall (because of the key) passes through the soil and not along the soil-concrete interface. Therefore, the friction angle along the failure plane was chosen to reflect a failure through the soil mass, not along the interface between the concrete and the foundation soils.

- The ground surface on the downstream side of the dam was conservatively assumed to be at El. 18 ft for the eastern section and El. 16 ft for the western section.
- The water level on downstream side of the retaining wall was assumed to act at the ground surface for all load cases as this water level produces the minimum resisting force.
- Driving soil forces were calculated using active soil pressures determined using the Coulomb equations.
- Resisting soil forces were calculated using passive soil pressures determined using the log spiral method.
- For seismic design, we used a seismic acceleration of 0.15g in accordance with 302 CMR 10.14 (9i).

3.0 Dam Repair Structural Analyses

For the structural design of the cantilevered retaining wall, we used Engineer Manual 1110-2-2104, Strength Design of Reinforced Concrete Hydraulic Structures (U.S. Army Corps of Engineers) and ACI 318-05, Building Code Requirements for Structural Concrete. We used the Single Load Factor Method of EM 111-2-2104.

We used the forces and moments calculated as part of the stability analysis for the structural design. For simplicity of construction, we chose not to vary the reinforcing scheme across the length of the dam.

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Attachment 1

Structural Calculations

CALCULATIONS

for

Structural Analysis (Technical Memorandum 4)

Mill Pond Dam
Rockport, Massachusetts

April 29, 2010

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Spillway Cradle Design 4.2.14
Parapet Wall Stability and Structural Design 4.2.19
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Table 1- Stability and Structural Analyses Summary Table
Mill Pond Dam
Rockport, Ma
GEI Project # 07104-1

	Load Case	Sliding Factor of Safety	Base Area in Compression (%)	Stem		Heel		Toe		Reinforcing		
				Moment (K*ft/ft)	Shear (k/ft)	Moment (K*ft/ft)	Shear (k/ft)	Moment (K*ft/ft)	Shear (k/ft)	Stem	Base	Key
Left Side of Dam (STA. 1+25 - 1+50)	1	3.05	100	-40	-11	31	6	17	-10	#9@12" Vertical #7@12" Horizontal	#6@12" E.F. Longitudinal #9@12" E.F. Transverse	#6@6" hoops 4 longitudinal #6
	2	2.19	92	-66	-16	48	11	23	-14			
	3	1.78	87	-42	-10	32	7	16	-10			
Right Side of Dam (STA. 1+50-2+40)	1	>5	100	-67	-16	52	10	54	-17			
	2	3.12	100	**-103**	**-21**	**77**	**15**	**67**	**-21**			
	3	1.24	93	-64	-13	53	11	44	-14			

Load Case Descriptions
-Load Case 1 (Usual Condition) - Normal Pool
-Load Case 2 (Unusual Condition) - Design Flood (100 yr)
-Load Case 3 (Extreme Condition) - Normal Pool + Earthquake

Stability Criteria

Load Case	Minimum Sliding Factor of Safety	Base Area in Compression
Usual	3	100%
Unusual	2	75%
Extreme	>1.0	Resultant within Base

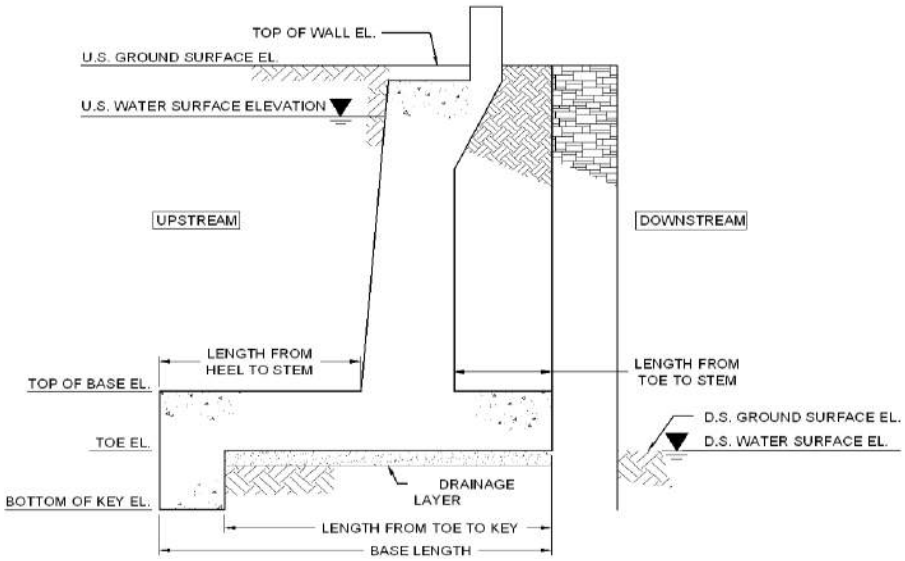
Notes
1. See Attachment 1 for detailed stability calculations
2. See Attachment 2 for detailed structural calculations
3. Bold text denotes controlling load case for structural calculations.
4. Minimum factor of safety against sliding taken from 302 CMR 10.14 (9k) (Massachusetts Dam Safety Regulations)
5. Base area in compression for overturning taken from EM 1110-2-2502 (US Army Corps of Engineers)

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

LOCATION - Left Side of Dam (STA. 1+25 - 1+50)

LOAD CASE 1 - Normal Pool

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

GIVEN

Geometry

Top of Wall Elevation	31.0	ft	
Toe Elevation	16.0	ft	
Bottom of Key Elevation	14.0	ft	
Top of Base Elevation	18.0	ft	
Length from Heel to Stem	6.2	ft	
Length from Toe to Stem	3.0	ft	
Length from Toe to Key	10.00	ft	
Base Length (B)	12.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	12.0	ft^2	$(W \cdot B)$
Base Section Modulus (S)	24.0	ft^3	$(W \cdot B^2) / 6$

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_{fp})	35	degrees
Friction Factor at Failure Plane ($\tan\phi_{fp}$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	27.3	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$	
Active Soil Pressure Coeff., k_a =	0.31		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

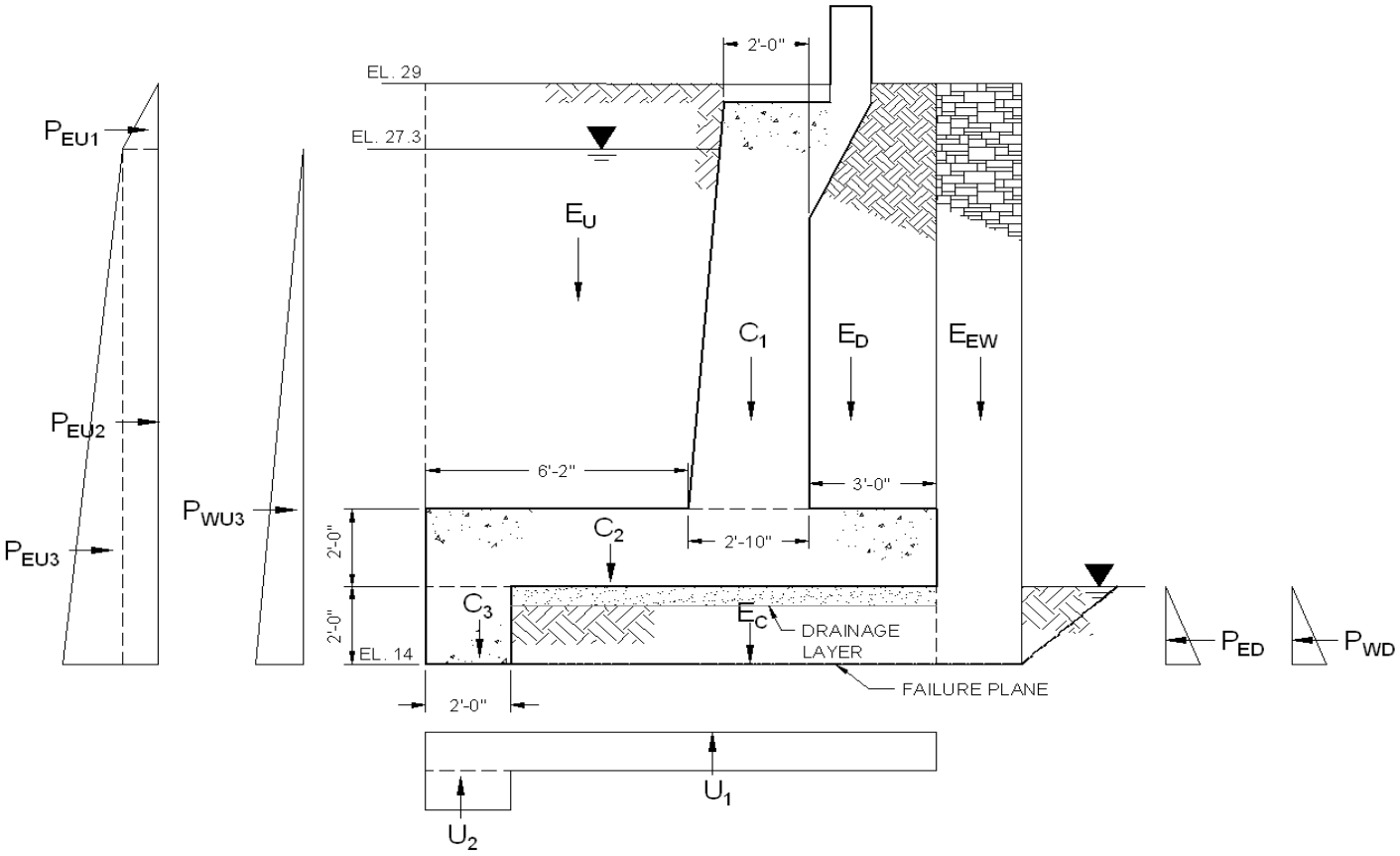
Downstream

Water Surface Elevation	16.0	ft
Ground Surface Elevation	16.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$
Passive Soil Pressure Coeff., k_p =	6.00	

K_p Determined using Log Spiral Charts

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.40	1.00	1.0	0.150	4.68		4.20	19.7	
C_2 Base	2.00	12.00	1.00	1.0	0.150	3.60		6.00	21.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		11.00	6.6	
Soil Weight										
E_C Confined	2.00	10.00	1.00	1.0	0.130	2.60				
E_U Upstream	11.00	6.60	1.00	1.0	0.130	9.44		8.70	82.1	
E_D Flow Fill	11.00	3.00	1.00	1.0	0.130	4.29		1.50	6.4	
E_{ew} Ex. wall	15.00	2.00	1.00	1.0	0.130	3.90 (a)				
Water Weight										
W_U Upstream	0.00	6.60	1.00	1.0	0.0624	0.0		0.00	0.0	
W_D Downstream										
TOTALS						**29.1**			**136**	

(a). Vertical Force from existing wall weight included in sliding analysis only.

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	2.00	12.00	1.00	1.0	0.0624		1.50	6.00		9.0
U_2 Uplift 2	7.00 (b)	2.00	1.00	1.0	0.0624		0.87	11.00		9.6
TOTALS							**2.4**			**19**

(b). = Pressure Head at bottom of key (see below)

HORIZONTAL FORCES

Soil Pressure — **Active / Passive** — Force = H · W · L · SF · Wt. · K — Upstream Ka = 0.31 — Downstream Kp = 6.00

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	1.70	1.70	1.00	0.5	0.130		0.06	11.87		0.7
P_{EU2} Upstream 2	13.30	1.70	1.00	1.0	0.130		0.90	4.65		4.2
P_{EU3} Upstream 3	13.30	13.30	1.00	0.5	0.130		2.39 (c)	2.43		5.8
P_{ED} Downstream	2.00	2.00	1.00	0.5	0.068	0.82		-1.33		1.1
TOTALS						**0.8**	**3.3**		**0**	**12**

(c). Force=(H · W · SF · Wt. - P_{wu3}) · Ka

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU1} Upstream 1	0.00	0.00	1.00	0.5	0.0624		0.00	11.30		0.0
P_{WU2} Upstream 2	0.00	0.00	1.00	1	0.0624		0.00	-2.00		0.0
P_{WU3} Upstream 3	13.30	9.00 (d)	1.00	0.5	0.0624		3.73	2.43		9.1
P_{WD} Downstream	2.00	2.00	1.00	0.5	0.0624	0.12		-1.33		0.2
TOTALS						**0.1**	**3.7**		**0**	**9**

(d). Pressure head at upstream corner of key from seepage analysis

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

SUMMATION

Net Vertical Force, V = 26.7 **k** ↓ **Net Moment About the Toe, M =** 97 **ft-k**

Includes existing wall weight

Net Horizontal Force, H = -6.1 **k** →

STABILITY ANALYSIS

LOAD CASE 1 - Normal Pool

Load Category (Usual, Unusual, or Extreme)	USUAL
Hazard Classification (High, Significant, Low)	SIGNIFICANT
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

Required $F.S._{sliding}$ =	3.0	
Required $F.S._{flotation}$ =	1.3	
Max Fdn Bearing Pressure =	4.0	ksf

100 % of Base in Compression Required

SLIDING ANALYSIS

Active / Passive

	V		H	
Σ Forces =	26.7	k	-6.1	k
	↓		→	

Includes existing wall weight

$F.S._{sliding}$ = 3.05 $(V\tan\phi_{fp} + cA) / H$ OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V		H		M	
Σ Forces =	22.8	k	-6.1	k	**Σ Moments =** 97	ft-k
	↓		→		↺	

Excludes existing wall weight

$F.S._{flotation}$ =	> 5.0		(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	4.0 ft.	**and**	8.0 ft.	Measured from the Toe
Resultant, R at	4.2	**ft. from toe**	(M/V)	Resultant within the kern
Eccentricity, e =	1.8	**ft.**	(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	3.58	**ksf**	(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	0.23	**ksf**	(V/A - Ve/S)	OK - Less than Bearing Capacity
	100%	**of base in compression**		OK - 100% Base in compression

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor:	B_f	1.7	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 b.)*
Unusual/Extreme Load Factor:	E_f	1.0	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 18.0 ft (BASE OF STEM)

Soil Pressure — **At-Rest** — Force = H · W · L · SF · Wt. · K — Upstream Ko = 0.47 — Downstream Ko = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	9.87		0.9
$P_{EU2'}$	Upstream 2	9.30	1.70	1.00	1.0	0.130		0.97	4.65		4.5
$P_{EU3'}$	Upstream 3	9.30	9.30	1.00	0.5	0.068		1.37	3.10		4.3
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
TOTALS							**0.0**	**2.4**		**0**	**10**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	9.30	9.30	1.00	0.5	0.0624		2.70	3.10		8.4
$P_{WD'}$	Downstream	Conservatively Neglect									
TOTALS							**0.0**	**2.7**		**0**	**8**

Net Shear Force, V = -5 k — **Net Moment, M = -18 ft-k**

Factored Shear Force, V_u = -11 k — **Factored Moment, M_u = -40 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) — ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	6.20	1.00	1.0	0.150	1.86		3.10	5.8	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		5.20	3.1	
Soil Weight										
E_U Upstream	11.00	6.20	1.00	1.0	0.130	8.87		3.10	27.5	
Water Weight										
W_U Upstream										
TOTALS						**11.3**			**36**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	2.00	6.20	1.00	1.0	0.0624		0.77	3.10		2.4
U_{2U} Uplift 2	7.00	2.00	1.00	1.0	0.0624		0.87	5.20		4.5
TOTALS							**1.6**			**7**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	0.23	6.20	1.00	1.0	-		1.40	3.10		4.3
B_{2U} Bearing 2	1.73	6.20	1.00	0.5	-		5.37	2.07		11.1
TOTALS							**6.8**			**15**

Net Shear Force, V = 3 k **Net Moment, M = 14 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 6 k **Factored Moment, M_u = 31 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	3.00	1.00	1.0	0.150	0.90		1.50		1.4
Soil Weight										
E_D Flow Fill	11.00	3.00	1.00	1.00	0.130	4.29		1.50		6.4
Water Weight										
W_D Downstream										
TOTALS						**5.2**				**8**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	2.00	3.00	1.00	1.0	0.0624		0.37	1.50	0.6	
U_{2D} Uplift 2										
TOTALS							**0.4**		**1**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	2.74	3.00	1.00	1.0	-		8.22	1.50	12.3	
B_{2D} Bearing 2	0.84	3.00	1.00	0.5	-		1.26	2.00	2.5	
TOTALS							**9.5**		**15**	

Net Shear Force, V = -5 k **Net Moment, M = 8 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -10 k **Factored Moment, M_u = 17 ft-k**

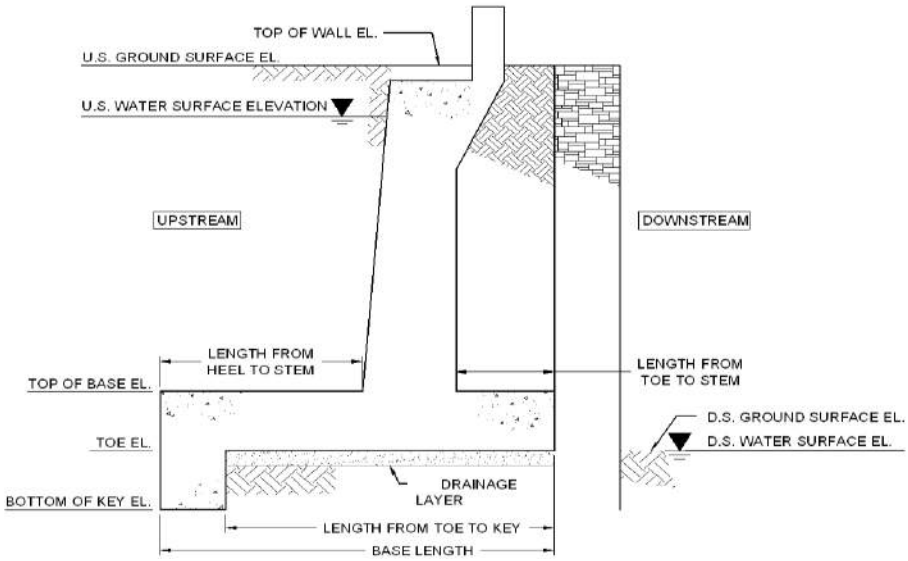
($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

LOCATION - Left Side of Dam (STA. 1+25 - 1+50)

LOAD CASE 2 - 100 year flood

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

GIVEN

Geometry

Top of Wall Elevation	31.0	ft	
Toe Elevation	16.0	ft	
Bottom of Key Elevation	14.0	ft	
Top of Base Elevation	18.0	ft	
Length from Heel to Stem	6.2	ft	
Length from Toe to Stem	3.0	ft	
Length from Toe to Key	10.00	ft	
Base Length (B)	12.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	12.0	ft^2	$(W \cdot B)$
Base Section Modulus (S)	24.0	ft^3	$(W \cdot B^2) / 6$

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_{fp})	35	degrees
Friction Factor at Failure Plane ($\tan\phi_{fp}$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	31.0	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$	
Active Soil Pressure Coeff., k_a =	0.31		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

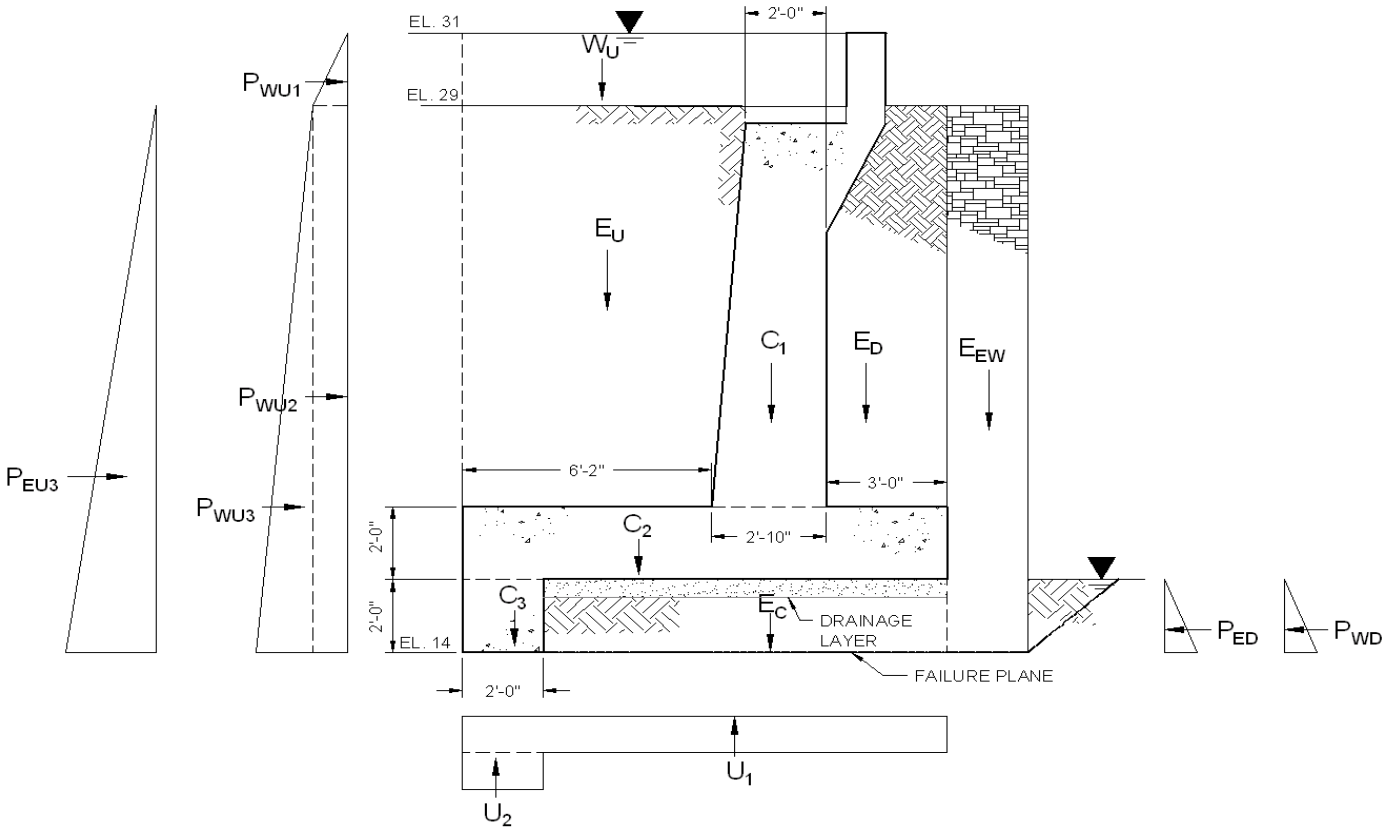
Downstream

Water Surface Elevation	16.0	ft
Ground Surface Elevation	16.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$
Passive Soil Pressure Coeff., k_p =	6.00	

K_p Determined using Log Spiral Charts

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport	
		Project: Mill Pond Dam Repairs	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants

Left Side of Dam
100 Year Flood

Client: Town of Rockport
Project: Mill Pond Dam Repairs
Subject: Stability Analysis
Prepared By: M. Hurd — Rev. Date: 4/21/10
Checked By: J. Christensen — Date: 04/23/10

TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.40	1.00	1.0	0.150	4.68		4.20	19.7	
C_2 Base	2.00	12.00	1.00	1.0	0.150	3.60		6.00	21.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		11.00	6.6	
Soil Weight										
E_C Confined	2.00	10.00	1.00	1.0	0.130	2.60				
E_U Upstream	11.00	6.60	1.00	1.0	0.130	9.44		8.70	82.1	
E_D Flow Fill	11.00	3.00	1.00	1.0	0.130	4.29		1.50	6.4	
E_{ew} Ex wall	15.00	2.00	1.00	1.0	0.130	3.90 (a)				
Water Weight										
W_U Upstream	2.00	9.00	1.00	1.0	0.0624	1.1		7.50	8.4	
W_D Downstream										
TOTALS						**30.2**			**145**	

(a). Vertical Force from existing wall weight included in sliding analysis only.

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	2.00	12.00	1.00	1.0	0.0624		1.50	6.00		9.0
U_2 Uplift 2	10.00 (b)	2.00	1.00	1.0	0.0624		1.25	11.00		13.7
TOTALS							**2.7**			**23**

(b). = Pressure Head at bottom of key (see below)

HORIZONTAL FORCES

Soil Pressure

Active / Passive Force = H · W · L · SF · Wt. · K Upstream Ka = 0.31 Downstream Kp = 6.00

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	0.00	0.00	1.00	0.5	0.130		0.00	15.00		0.0
P_{EU2} Upstream 2	15.00	0.00	1.00	1.0	0.130		0.00	5.50		0.0
P_{EU3} Upstream 3	15.00	15.00	1.00	0.5	0.130		3.06 (c)	3.00		9.2
P_{ED} Downstream	2.00	2.00	1.00	0.5	0.068	0.82		-1.33		1.1
TOTALS						**0.8**	**3.1**		**0**	**10**

(c). Force=(H · W · SF · Wt. - P_{wu3}) · Ka

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU1} Upstream 1	2.00	2.00	1.00	0.5	0.0624		0.12	15.67		2.0
P_{WU2} Upstream 2	15.00	2.00	1.00	1	0.0624		1.87	5.50		10.3
P_{WU3} Upstream 3	15.00	10.00 (d)	1.00	0.5	0.0624		4.68	3.00		14.0
P_{WD} Downstream	2.00	2.00	1.00	0.5	0.0624	0.12		-1.33		0.2
TOTALS						**0.1**	**6.7**		**0**	**26**

(d). Pressure head at upstream corner of key from seepage analysis

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

SUMMATION

Net Vertical Force, V = **27.5** **k** ↓ **Net Moment About the Toe, M =** **85** **ft-k**

Net Horizontal Force, H = **-8.8** **k** →

STABILITY ANALYSIS

LOAD CASE 2 - 100 year flood

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

		Required $F.S._{sliding}$ =	2.0	
Load Category (Usual, Unusual, or Extreme)	UNUSUAL	Required $F.S._{flotation}$ =	1.2	
Hazard Classification (High, Significant, Low)	SIGNIFICANT	Max Fdn Bearing Pressure =	4.6	ksf
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY	75 % of Base in Compression Required		

SLIDING ANALYSIS

Active / Passive

	V	H
Σ Forces =	**27.5** k ↓	**-8.8** k →

Includes existing wall weight

$F.S._{sliding}$ = **2.19** $(V\tan\phi_{fp} + cA) / H$ OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V	H	M
Σ Forces =	**23.6** k ↓	**-8.8** k →	**Σ Moments =** **85** ft-k ↺

Excludes existing wall weight

$F.S._{flotation}$ =	**> 5.0**	(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	**4.0** ft. **and** **8.0** ft.		Measured from the Toe
Resultant, R at	**3.6** ft. from toe	(M/V)	Resultant DOWNSTREAM of kern
Eccentricity, e =	**2.4** ft.	(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	**4.30** ksf	(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	**-0.37** ksf	(V/A - Ve/S)	OK - Less than Bearing Capacity
	92% of base in compression		<< Tension in Heel - Crack Develops >>

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor:	B_f	1.7	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 b.)*
Unusual/Extreme Load Factor:	E_f	1.00	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 18.0 ft (BASE OF STEM)

Soil Pressure **At-Rest** Force = H · W · L · SF · Wt. · K Upstream Ko = 0.47 Downstream Ko = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1										
$P_{EU2'}$	Upstream 2										
$P_{EU3'}$	Upstream 3	11.00	11.00	1.00	0.5	0.068		1.92	3.67		7.0
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
TOTALS							**0.0**	**1.9**		**0**	**7**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	13.00	13.00	1.00	0.5	0.0624		5.27	4.33		22.8
$P_{WD'}$	Downstream	Conservatively Neglect									
TOTALS							**0.0**	**5.3**		**0**	**23**

Net Shear Force, V = -7 k

Net Moment, M = -30 ft-k

Factored Shear Force, V_u = -16 k

($V_u = V \cdot B_f \cdot E_f \cdot H_f$)

Factored Moment, M_u = -66 ft-k

($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	6.20	1.00	1.0	0.150	1.86		3.10	5.8	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		5.20	3.1	
Soil Weight										
E_U Upstream	11.00	6.20	1.00	1.0	0.130	8.87		3.10	27.5	
Water Weight										
W_U Upstream	2.00	6.20	1.00	1.0	0.0624	0.77		3.10	2.4	
TOTALS						**12.1**			**39**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	2.00	6.20	1.00	1.0	0.0624		0.77	3.10		2.4
U_{2U} Uplift 2	10.00	2.00	1.00	1.0	0.0624		1.25	5.20		6.5
TOTALS							**2.0**			**9**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	-0.37	6.20	1.00	1.0	-		-2.31	3.10		-7.2
B_{2U} Bearing 2	2.42	6.20	1.00	0.5	-		7.49	2.07		15.5
TOTALS							**5.2**			**8**

Net Shear Force, V = 5 k **Net Moment, M = 22 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 11 k **Factored Moment, M_u = 48 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	3.00	1.00	1.0	0.150	0.90		1.50		1.4
Soil Weight										
E_D Flow Fill	11.00	3.00	1.00	1.00	0.130	4.29		1.50		6.4
Water Weight										
W_D Downstream										
TOTALS						**5.2**				**8**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	2.00	3.00	1.00	1.0	0.0624		0.37	1.50	0.6	
U_{2D} Uplift 2										
TOTALS							**0.4**		**1**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	3.13	3.00	1.00	1.0	-		9.40	1.50	14.1	
B_{2D} Bearing 2	1.17	3.00	1.00	0.5	-		1.75	2.00	3.5	
TOTALS							**11.2**		**18**	

Net Shear Force, V = -6 k **Net Moment, M = 10 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -14 k **Factored Moment, M_u = 23 ft-k**

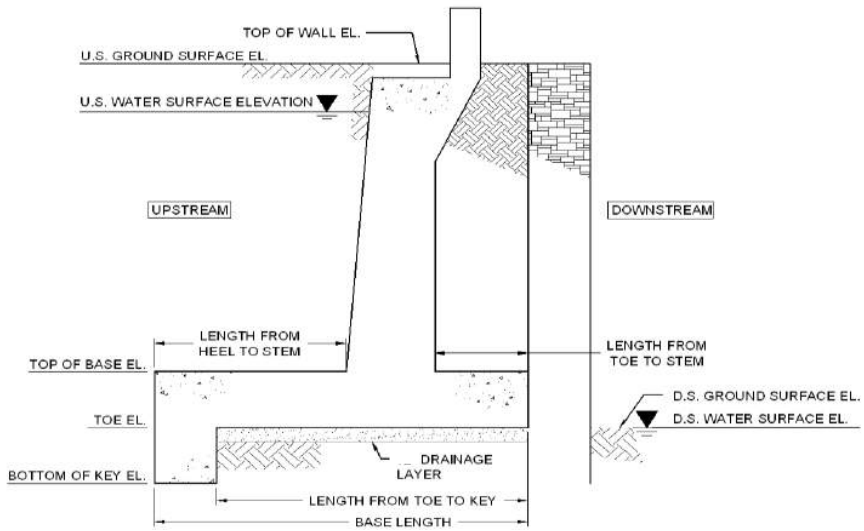
($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam Eathquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Repairs	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOCATION - Left Side of Dam (STA. 1+25 - 1+50)

LOAD CASE 3 - Normal Pool + Earthquake

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

GIVEN

Geometry

Top of Wall Elevation	31.0	ft	
Toe Elevation	16.0	ft	
Bottom of Key Elevation	14.0	ft	
Top of Base Elevation	18.0	ft	
Length from Heel to Stem	6.2	ft	
Length from Toe to Stem	3.0	ft	
Length from Toe to Key	10.00	ft	
Base Length (B)	12.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	12.0	ft^2	$(W \cdot B)$
Base Section Modulus (S)	24.0	ft^3	$(W \cdot B^2) / 6$

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_{fp})	35	degrees
Friction Factor at Failure Plane ($\tan\phi_{fp}$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	27.3	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$	
Active Soil Pressure Coeff., k_a=	0.31		
Eq. Soil Pressure Coeff. Inc., k_{ae}	0.12		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

Downstream

Water Surface Elevation	16.0	ft
Ground Surface Elevation	16.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	$(1 - \sin\phi) \cdot (1 + \sin\beta)$
Passive Soil Pressure Coeff., k_p =	6.00	

k_p assumed as k_o for seismic event

GEI Consultants	**Left Side of Dam Eathquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Repairs	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
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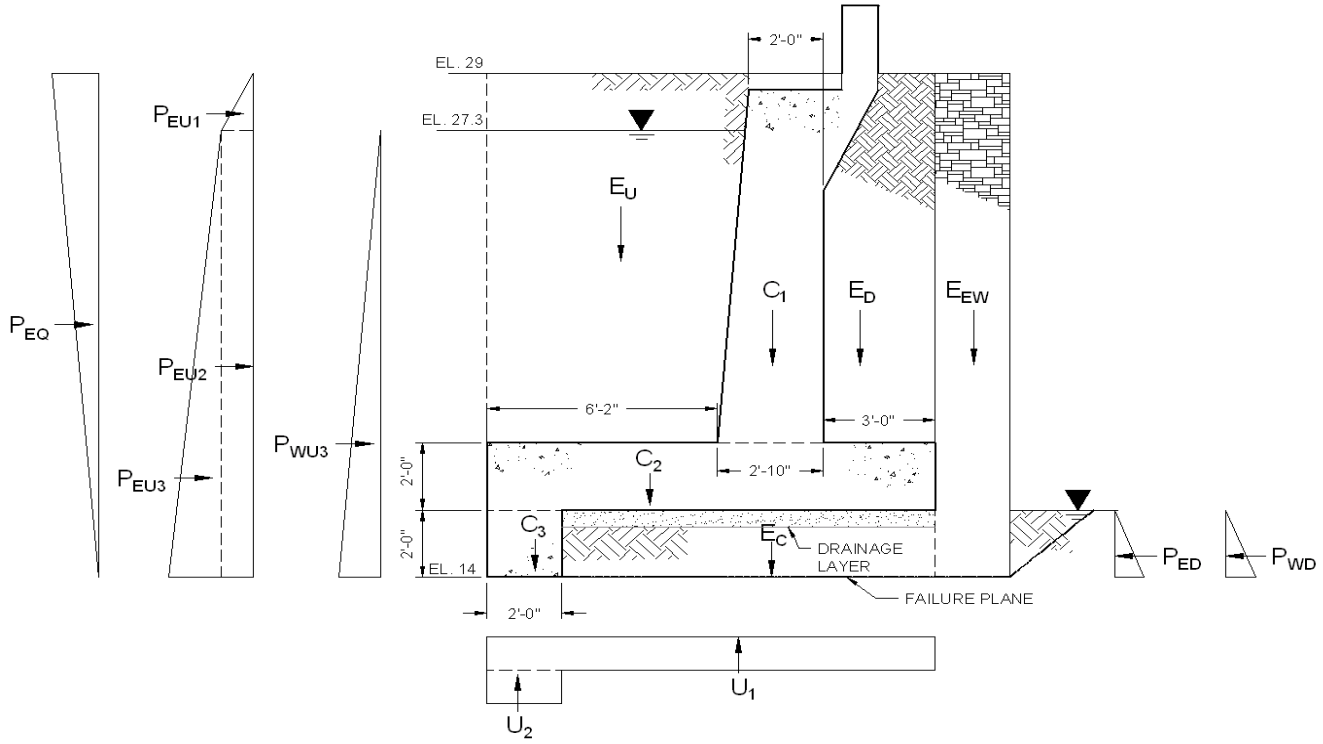
LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants	**Left Side of Dam Eathquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Repairs	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
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TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.00	1.00	1.0	0.150	3.90		4.00	15.6	
C_2 Base	2.00	12.00	1.00	1.0	0.150	3.60		6.00	21.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		11.00	6.6	
Soil Weight										
E_C Confined	2.00	10.00	1.00	1.0	0.130	2.60				
E_U Upstream	11.00	7.00	1.00	1.0	0.130	10.01		8.50	85.1	
E_D Flow Fill	11.00	3.00	1.00	1.0	0.130	4.29		1.50	6.4	
E_{ew} Ex wall	15.00	2.00	1.00	1.0	0.130	3.90 (a)				
Water Weight										
W_U Upstream	0.00	6.20	1.00	1.0	0.0624	0.0		0.00	0.0	
W_D Downstream										
TOTALS						**28.9**			**135**	

(a). Vertical Force from existing wall weight included in sliding analysis only.

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	2.00	12.00	1.00	1.0	0.0624		1.50	6.00		9.0
U_2 Uplift 2	7.00 (b)	2.00	1.00	1.0	0.0624		0.87	11.00		9.6
TOTALS							**2.4**			**19**

(b). = Pressure Head at bottom of key (see below)

HORIZONTAL FORCES

Soil Pressure

At-Rest Force = H · W · L · SF · Wt. · K Upstream Ko = 0.47 Downstream Ko = 0.47

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	11.87		1.0
P_{EU2} Upstream 2	13.30	1.70	1.00	1.0	0.130		1.38	4.65		6.4
P_{EU3} Upstream 3	13.30	13.30	1.00	0.5	0.130		3.65 (c)	2.43		8.9
P_{ED} Downstream	2.00	2.00	1.00	0.5	0.068	0.06		-1.33		0.1
TOTALS						**0.1**	**5.1**		**0**	**16**

(c). Force=(H · W · SF · Wt. - P_{wu3}) · Ka

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU1} Upstream 1	0.00	0.00	1.00	0.5	0.0624		0.00	11.30		0.0
P_{WU2} Upstream 2	0.00	0.00	1.00	1	0.0624		0.00	-2.00		0.0
P_{WU3} Upstream 3	13.30	9.00 (d)	1.00	0.5	0.0624		3.73	2.43		9.1
P_{WD} Downstream	2.00	2.00	1.00	0.5	0.0624	0.12		-1.33		0.2
TOTALS						**0.1**	**3.7**		**0**	**9**

(d). Pressure head at upstream corner of key from seepage analysis

Earthquake Loading

K_{ae}= 0.12

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{eq} Dyn. Soil Eq.	15.0	15.0	1	0.5	0.130		1.76 (e)	9.00 (f)		15.8
TOTALS						**0.0**	**1.8**		**0**	**16**

(e.) Soil Force = H · W · L · SF · Wt. · K_{ae}

(f.) Moment arm acts at .6 · H from the base of the dam.

GEI Consultants	**Left Side of Dam** **Eathquake**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

SUMMATION

Net Vertical Force, V = 26.5 **k** ↓ **Net Moment About the Toe, M =** 75 **ft-k**

Net Horizontal Force, H = -10.4 **k** →

STABILITY ANALYSIS

LOAD CASE 3 - Normal Pool + Earthquake

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

Load Category (Usual, Unusual, or Extreme)	EXTREME	Required $F.S._{sliding}$ =	1.0	
Hazard Classification (High, Significant, Low)	SIGNIFICANT	Required $F.S._{flotation}$ =	1.1	
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY	Max Fdn Bearing Pressure =	6.0	ksf
		Resultant Location Required Within Base		

SLIDING ANALYSIS

At-Rest

	V		H	
Σ Forces =	26.5	k	-10.4	k
	↓		→	

Includes existing wall weight

$F.S._{sliding}$ = 1.78 ($V\tan\phi_{fp} + cA$) / H — OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V		H			M	
Σ Forces =	22.6	k	-10.4	k	**Σ Moments =**	75	ft-k
	↓		→			↺	

Excludes existing wall weight

$F.S._{flotation}$ =	> 5.0			(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	4.0	ft.	**and** 8.0	ft.	Measured from the Toe
Resultant, R at	3.3	ft. from toe		(M/V)	Resultant DOWNSTREAM of kern
Eccentricity, e =	2.7	ft.		(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	4.41	ksf		(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	-0.64	ksf		(V/A - Ve/S)	OK - Less than Bearing Capacity
	87%	of base in compression			<< Tension in Heel - Crack Develops >>

GEI Consultants	**Left Side of Dam Eathquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Repairs	
		Subject: Stability Analysis	
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STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor (Dead, Live, and Earthquake):	B_f	1.5	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 d)*
Unusual/Extreme Load Factor:	E_f	0.75	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 18.0 ft (BASE OF STEM)

Soil Pressure **At-Rest** Force = H · W · L · SF · Wt. · K Upstream Ko = 0.47 Downstream Ko = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	9.87		0.9
$P_{EU2'}$	Upstream 2	9.30	1.70	1.00	1.0	0.130		0.97	4.65		4.5
$P_{EU3'}$	Upstream 3	9.30	9.30	1.00	0.5	0.068		1.37	3.10		4.3
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
TOTALS							**0.0**	**2.4**		**0**	**10**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	9.30	9.30	1.00	0.5	0.0624		2.70	3.10		8.4
$P_{WD'}$	Downstream	Conservatively Neglect									
TOTALS							**0.0**	**2.7**		**0**	**8**

Earthquake Loading K_{ae}= 0.12

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{eq}	Dyn. Soil Eq.	11.0	9.5	1	1	0.130		1.63	6.60		10.8
TOTALS							**0.0**	**1.6**		**0**	**11**

Net Shear Force, V = -7 k **Net Moment, M = -29 ft-k**

Factored Shear Force, V_u = -10 k **Factored Moment, M_u = -42 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **Eathquake**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	6.20	1.00	1.0	0.150	1.86		3.10	5.8	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		5.20	3.1	
Soil Weight										
E_U Upstream	11.00	6.20	1.00	1.0	0.130	8.87		3.10	27.5	
Water Weight										
W_U Upstream										
TOTALS						**11.3**			**36**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	2.00	6.20	1.00	1.0	0.0624		0.77	3.10		2.4
U_{2U} Uplift 2	7.00	2.00	1.00	1.0	0.0624		0.87	5.20		4.5
TOTALS							**1.6**			**7**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	0.00	6.20	1.00	1.0	-		0.00	3.10		0.0
B_{2U} Bearing 2	1.97	4.69	1.00	0.5	-		4.62	1.56		7.2
TOTALS							**4.6**			**7**

Net Shear Force, V = 5 k **Net Moment, M = 22 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 7 k **Factored Moment, M_u = 32 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Left Side of Dam** **Eathquake**	Client: Town of Rockport Project: Mill Pond Dam Repairs Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	3.00	1.00	1.0	0.150	0.90		1.50		1.4
Soil Weight										
E_D Flow Fill	11.00	3.00	1.00	1.00	0.130	4.29		1.50		6.4
Water Weight										
W_D Downstream										
TOTALS						**5.2**				**8**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	2.00	3.00	1.00	1.0	0.0624		0.37	1.50	0.6	
U_{2D} Uplift 2										
TOTALS							**0.4**		**1**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	3.15	3.00	1.00	1.0	-		9.44	1.50	14.2	
B_{2D} Bearing 2	1.26	3.00	1.00	0.5	-		1.89	2.00	3.8	
TOTALS							**11.3**		**18**	

Net Shear Force, V = -7 k **Net Moment, M = 11 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -10 k **Factored Moment, M_u = 16 ft-k**

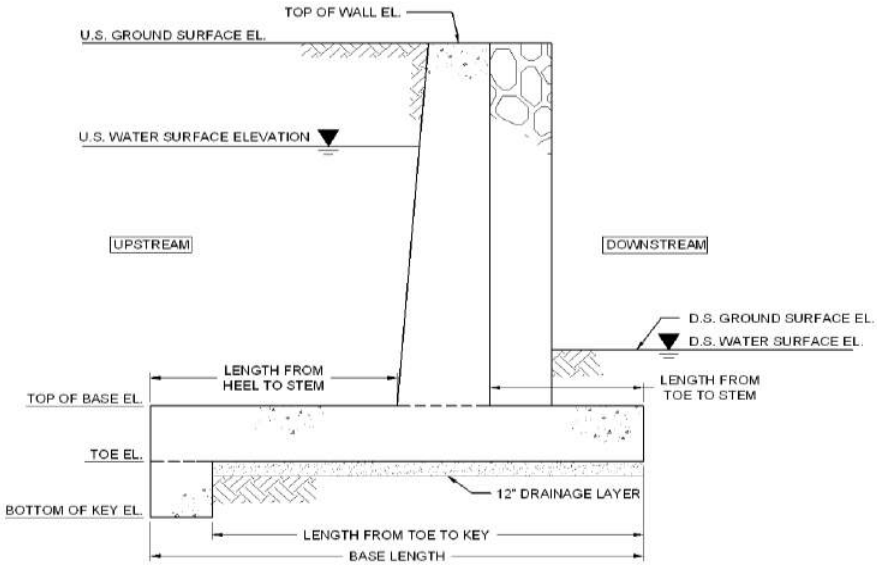
($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

LOCATION - Right Side of Dam (STA 1+50 - 2+35)

LOAD CASE 1 - Normal Pool

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

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Geometry

Top of Wall Elevation	29.0	ft	
Toe Elevation	14.0	ft	
Bottom of Key Elevation	12.0	ft	
Top of Base Elevation	16.0	ft	
Length from Heel to Stem	8.0	ft	
Length from Toe to Stem	5.0	ft	
Length from Toe to Key	14.00	ft	
Base Length (B)	16.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	16.0	ft^2	(W · B)
Base Section Modulus (S)	42.7	ft^3	(W · B^2) / 6

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_f)	35	degrees
Friction Factor at Failure Plane ($\tan\phi_f$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	27.3	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)	
Active Soil Pressure Coeff., k_a =	0.31		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

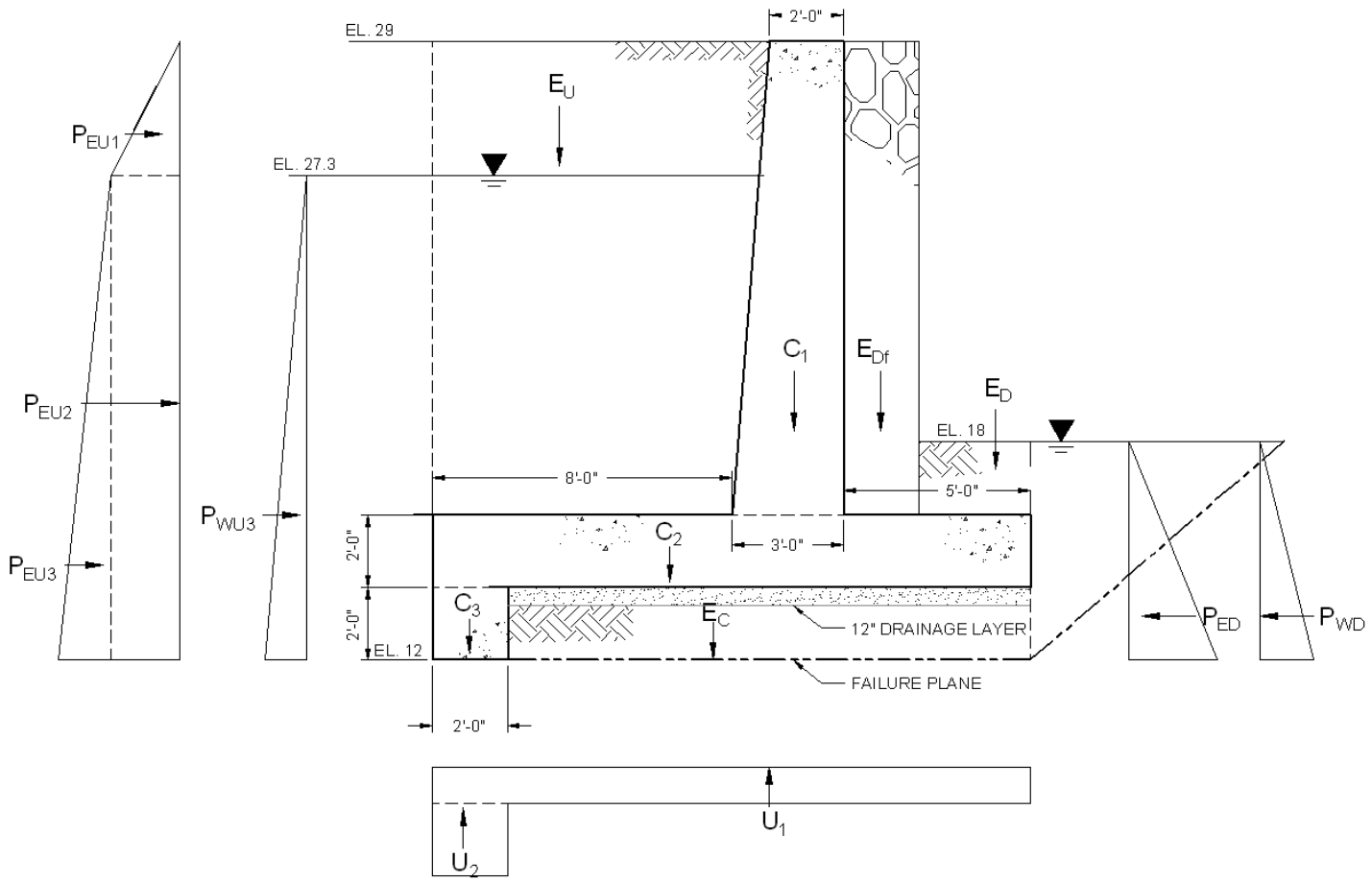
Downstream

Water Surface Elevation	18.0	ft
Ground Surface Elevation	18.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)
Passive Soil Pressure Coeff., k_p =	6.00	

K_p Determined using Log Spiral Charts

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants

Right Side of Dam
Normal Pool

Client: Town of Rockport
Project: Mill Pond Dam Safety Modifications
Subject: Stability Analysis
Prepared By: M. Hurd — Rev. Date: 4/21/10
Checked By: J. Christensen — Date: 04/23/10

TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.50	1.00	1.0	0.150	4.88		6.25	30.5	
C_2 Base	2.00	16.00	1.00	1.0	0.150	4.80		8.00	38.4	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		15.00	9.0	
Soil Weight										
E_C Confined	2.00	14.00	1.00	1.0	0.130	3.64				
E_U Upstream	13.00	8.50	1.00	1.0	0.130	14.37		11.75	168.8	
E_D Downstream	2.00	3.00	1.00	1.0	0.130	0.78		1.50	1.2	
E_{Df} Stone Façade	13.00	2.00	1.00	1.0	0.130	3.38		4.00	13.5	
Water Weight										
W_U Upstream										
W_D Downstream										
TOTALS						**32.4**			**261**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	6.00	16.00	1.00	1.0	0.0624		5.99	8.00		47.9
U_2 Uplift 2	9.30	2.00	1.00	1.0	0.0624		1.16	15.00		17.4
TOTALS							**7.2**			**65**

HORIZONTAL FORCES

Soil Pressure

Active / Passive Force = H · W · L · SF · Wt. · K Upstream Ka = 0.31 Downstream Kp = 6.00

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	1.70	1.70	1.00	0.5	0.130		0.06	13.87		0.8
P_{EU2} Upstream 2	15.30	1.70	1.00	1.0	0.130		1.04	5.65		5.9
P_{EU3} Upstream 3	15.30	15.30	1.00	0.5	0.068		2.43	3.10		7.5
P_{ED} Downstream	6.00	6.00	1.00	0.5	0.068	7.30		0.00	0.0	0.0
TOTALS						**7.3**	**3.5**		**0**	**14**

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU} Upstream	15.30	15.30	1.00	0.5	0.0624		7.30	3.10		22.6
P_{WD} Downstream	6.00	6.00	1.00	0.5	0.0624	1.12		0.00	0.0	
TOTALS						**1.1**	**7.3**		**0**	**23**

SUMMATION

Net Vertical Force, V = **25.3** **k** ↓

Net Moment About the Toe, M = **159** **ft-k**

Net Horizontal Force, H = **-2.4** **k** →

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd	Rev. Date: 4/21/10 Date: 04/23/10 Checked By: J. Christensen

STABILITY ANALYSIS

LOAD CASE 1 - Normal Pool

Load Category (Usual, Unusual, or Extreme)	USUAL
Hazard Classification (High, Significant, Low)	SIGNIFICANT
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

Required $F.S._{sliding}$ =	3.0	
Required $F.S._{flotation}$ =	1.3	
Max Fdn Bearing Pressure =	4.0	ksf

100 % of Base in Compression Required

SLIDING ANALYSIS

Active / Passive

	V		H	
Σ Forces =	**25.3**	**k**	**-2.4**	**k**
	↓		→	

$F.S._{sliding}$ = **> 5.0** (Vtanϕ_r + cA) / H — OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V		H		M	
Σ Forces =	**25.3**	**k**	**-2.4**	**k**	**Σ Moments =** **159**	**ft-k**
	↓		→		↺	

$F.S._{flotation}$ =	**4.54**		(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	**5.3** **ft.**	**and**	**10.7** **ft.**	Measured from the Toe
Resultant, R at	**6.3**	**ft. from toe**	(M/V)	Resultant within the kern
Eccentricity, e =	**1.7**	**ft.**	(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	**2.59**	**ksf**	(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	**0.57**	**ksf**	(V/A - Ve/S)	OK - Less than Bearing Capacity
	100%	**of base in compression**		OK - 100% Base in compression

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

STRENGTH DESIGN

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Load Factors:			
Basic Load Factor:	B_f	1.7	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 b.)*
Unusual/Extreme Load Factor:	E_f	1.0	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 16.0 ft (BASE OF STEM)

Soil Pressure — **At-Rest** — Force = H · W · L · SF · Wt. · K — Upstream Ko = 0.47 — Downstream Ko = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	11.87		1.0
$P_{EU2'}$	Upstream 2	11.30	1.70	1.00	1.0	0.130		1.17	5.65		6.6
$P_{EU3'}$	Upstream 3	11.30	11.30	1.00	0.5	0.068		2.03	3.77		7.6
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
	TOTALS						**0.0**	**3.3**		**0**	**15**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	11.30	11.30	1.00	0.5	0.0624		3.98	3.77		15.0
$P_{WD'}$	Downstream	Conservatively Neglect									
	TOTALS						**0.0**	**4.0**		**0**	**15**

Net Shear Force, V = -7 k — **Net Moment, M = -30 ft-k**

Factored Shear Force, V_u = -16 k — **Factored Moment, M_u = -67 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) — ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	8.00	1.00	1.0	0.150	2.40		4.00	9.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		7.00	4.2	
Soil Weight										
E_U Upstream	13.00	8.50	1.00	1.0	0.130	14.37		4.25	61.1	
Water Weight										
W_U Upstream										
TOTALS						**17.4**			**75**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	6.00	8.50	1.00	1.0	0.0624		3.18	4.25		13.5
U_{2U} Uplift 2	9.30	2.00	1.00	1.0	0.0624		1.16	7.50		8.7
TOTALS							**4.3**			**22**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	0.57	8.00	1.00	1.0	-		4.55	4.00		18.2
B_{2U} Bearing 2	1.01	8.00	1.00	0.5	-		4.04	2.67		10.8
TOTALS							**8.6**			**29**

Net Shear Force, V = 4 k **Net Moment, M = 24 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 10 k **Factored Moment, M_u = 52 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **Normal Pool**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	5.00	1.00	1.0	0.150	1.50		2.50		3.8
Soil Weight										
E_D Downstream	2.00	3.00	1.00	1.00	0.130	0.78		3.50		2.7
E_{Df} Stone Façade	13.00	2.00	1.00	1.00	0.130	3.38		1.00		3.4
Water Weight										
W_D Downstream										
TOTALS						**5.7**				**10**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	6.00	5.00	1.00	1.0	0.0624		1.87	2.50	4.7	
U_{2D} Uplift 2										
TOTALS							**1.9**		**5**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	1.96	5.00	1.00	1.0	-		9.80	2.50	24.5	
B_{2D} Bearing 2	0.63	5.00	1.00	0.5	-		1.58	3.33	5.3	
TOTALS							**11.4**		**30**	

Net Shear Force, V = -8 k **Net Moment, M = 25 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -17 k **Factored Moment, M_u = 54 ft-k**

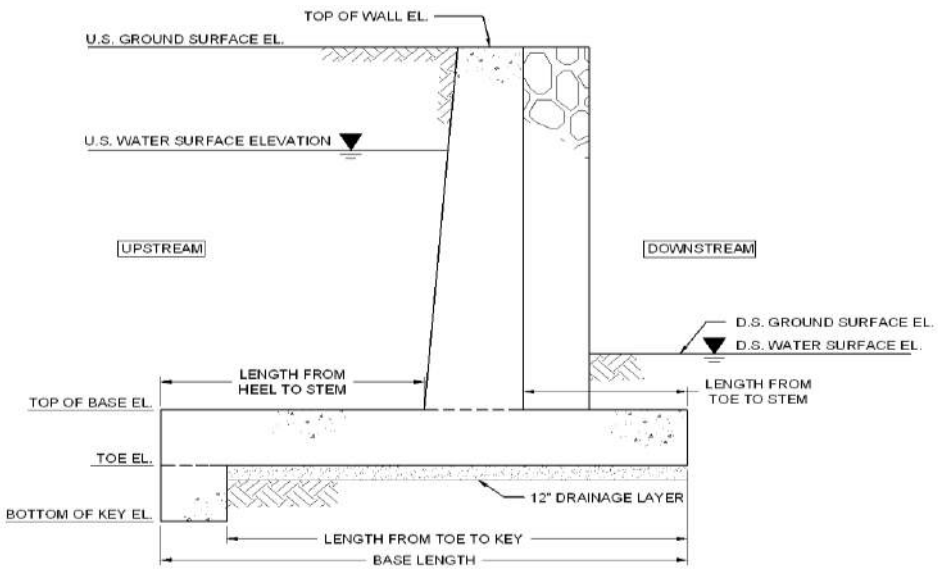
($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd Checked By: J. Christensen	Rev. Date: 4/21/10 Date: 04/23/10

LOCATION - Right Side of Dam (STA 1+50 - 2+35)

LOAD CASE 2 - 100 year flood

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

GIVEN

Geometry

Top of Wall Elevation	29.0	ft	
Toe Elevation	14.0	ft	
Bottom of Key Elevation	12.0	ft	
Top of Base Elevation	16.0	ft	
Length from Heel to Stem	8.0	ft	
Length from Toe to Stem	5.0	ft	
Length from Toe to Key	14.00	ft	
Base Length (B)	16.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	16.0	ft^2	(W · B)
Base Section Modulus (S)	42.7	ft^3	(W · B^2) / 6

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_f)	35	degrees
Friction Factor at Failure Plane ($\tan\phi_f$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	31.0	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)	
Active Soil Pressure Coeff., k_a =	0.31		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

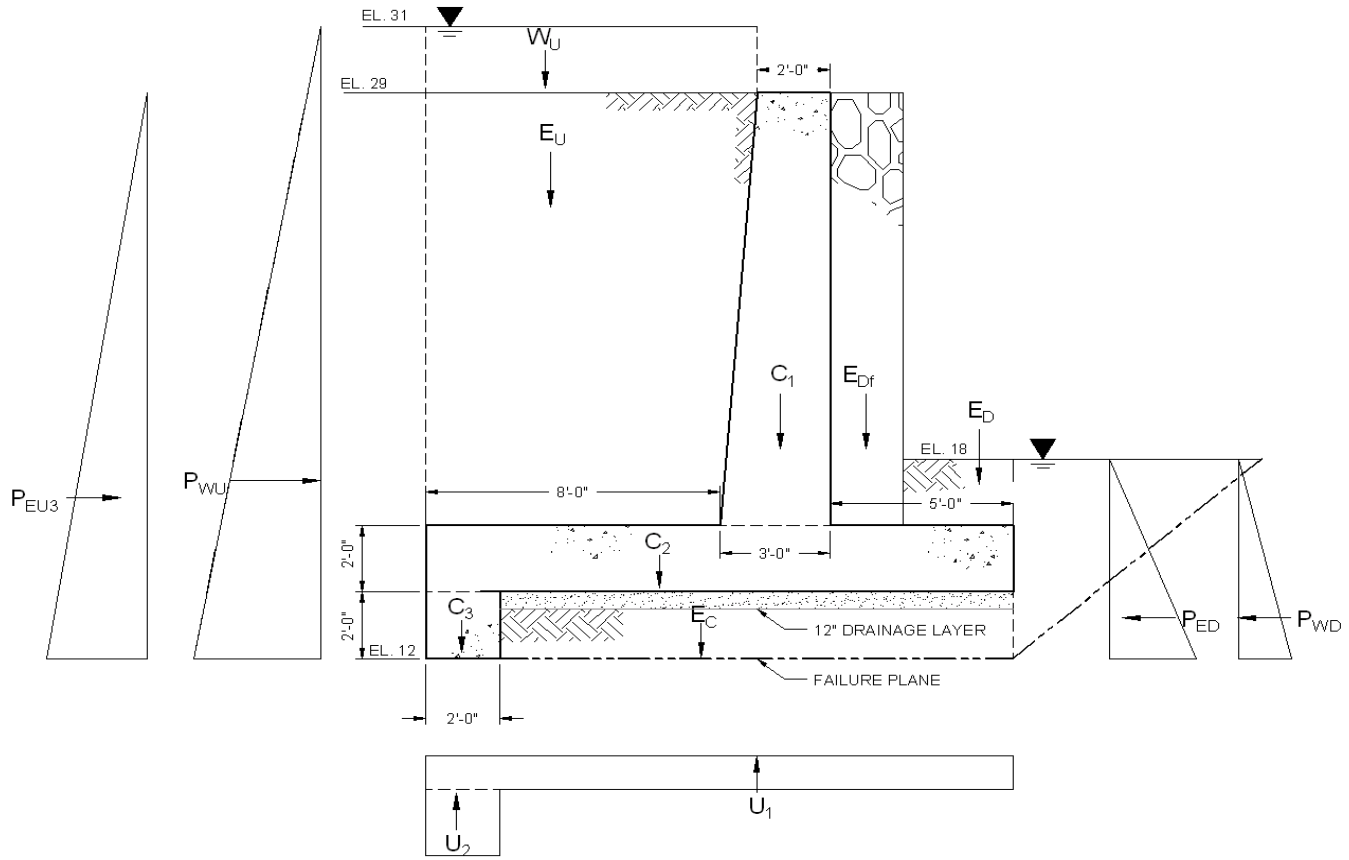
Downstream

Water Surface Elevation	18.0	ft
Ground Surface Elevation	18.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)
Passive Soil Pressure Coeff., k_p =	6.00	

K_p Determined using Log Spiral Charts

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.50	1.00	1.0	0.150	4.88		6.25	30.5	
C_2 Base	2.00	16.00	1.00	1.0	0.150	4.80		8.00	38.4	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		15.00	9.0	
Soil Weight										
E_C Confined	2.00	14.00	1.00	1.0	0.130	3.64				
E_U Upstream	13.00	8.50	1.00	1.0	0.130	14.37		11.75	168.8	
E_D Downstream	2.00	3.00	1.00	1.0	0.130	0.78		1.50	1.2	
E_{Df} Stone Façade	13.00	2.00	1.00	1.0	0.130	3.38		4.00	13.5	
Water Weight										
W_U Upstream	2.00	8.00	1.00	1.0	0.0624	1.0		12.00	12.0	
W_D Downstream										
TOTALS						**33.4**			**273**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	6.00	16.00	1.00	1.0	0.0624		5.99	8.00		47.9
U_2 Uplift 2	13.00	2.00	1.00	1.0	0.0624		1.62	15.00		24.3
TOTALS							**7.6**			**72**

HORIZONTAL FORCES

Soil Pressure — **Active / Passive** — Force = H · W · L · SF · Wt. · K — Upstream Ka = 0.31 — Downstream Kp = 6.00

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	0.00	0.00	1.00	0.5	0.130		0.00	17.00		0.0
P_{EU2} Upstream 2	17.00	0.00	1.00	1.0	0.130		0.00	6.50		0.0
P_{EU3} Upstream 3	17.00	17.00	1.00	0.5	0.068		3.00	3.67		11.0
P_{ED} Downstream	6.00	6.00	1.00	0.5	0.068	7.34		0.00	0.0	0.0
TOTALS						**7.3**	**3.0**		**0**	**11**

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU} Upstream	19.00	19.00	1.00	0.5	0.0624		11.26	4.33		48.8
P_{WD} Downstream	6.00	6.00	1.00	0.5	0.0624	1.12				
TOTALS						**1.1**	**11.3**		**0**	**49**

SUMMATION

Net Vertical Force, V = 25.8 k ↓

Net Moment About the Toe, M = 141 ft-k

Net Horizontal Force, H = -5.8 k →

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

STABILITY ANALYSIS

LOAD CASE 2 - 100 year flood

Load Category (Usual, Unusual, or Extreme)	UNUSUAL
Hazard Classification (High, Significant, Low)	SIGNIFICANT
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

Required $F.S._{sliding}$ =	2.0	
Required $F.S._{flotation}$ =	1.2	
Max Fdn Bearing Pressure =	4.6	ksf
75 % of Base in Compression Required		

SLIDING ANALYSIS

Active / Passive

	V		H	
Σ Forces =	**25.8**	**k** ↓	**-5.8**	**k** →

$F.S._{sliding}$ = **3.12** (Vtanϕ_r + cA) / H — OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V		H		M	
Σ Forces =	**25.8**	**k** ↓	**-5.8**	**k** →	**Σ Moments =** **141**	**ft-k** ↺

$F.S._{flotation}$ =	**4.39**	(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	**5.3** **ft.** **and**	**10.7** **ft.**	Measured from the Toe
Resultant, R at	**5.5** **ft. from toe**	(M/V)	Resultant within the kern
Eccentricity, e =	**2.5** **ft.**	(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	**3.15** **ksf**	(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	**0.08** **ksf**	(V/A - Ve/S)	OK - Less than Bearing Capacity
	100% **of base in compression**		OK - 100% Base in compression

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor:	B_f	1.7	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 b.)*
Unusual/Extreme Load Factor:	E_f	1.00	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 16.0 ft (BASE OF STEM)

Soil Pressure — **At-Rest** — Force = H · W · L · SF · Wt. · K — Upstream Ka = 0.47 — Downstream Kp = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1										
$P_{EU2'}$	Upstream 2										
$P_{EU3'}$	Upstream 3	13.00	13.00	1.00	0.5	0.068		2.69	4.33		11.6
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
	TOTALS						**0.0**	**2.7**		**0**	**12**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	15.00	15.00	1.00	0.5	0.0624		7.02	5.00		35.1
$P_{WD'}$	Downstream	Conservatively Neglect									
	TOTALS						**0.0**	**7.0**		**0**	**35**

Net Shear Force, V = -10 k — **Net Moment, M = -47 ft-k**

Factored Shear Force, V_u = -21 k — **Factored Moment, M_u = -103 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) — ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	8.00	1.00	1.0	0.150	2.40		4.00	9.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		7.00	4.2	
Soil Weight										
E_U Upstream	13.00	8.50	1.00	1.0	0.130	14.37		4.25	61.1	
Water Weight										
W_U Upstream	2.00	8.50	1.00	1.0	0.0624	1.06		4.25	4.5	
TOTALS						**18.4**			**79**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	6.00	8.50	1.00	1.0	0.0624		3.18	4.25		13.5
U_{2U} Uplift 2	13.00	2.00	1.00	1.0	0.0624		1.62	7.50		12.2
TOTALS							**4.8**			**26**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	0.08	8.00	1.00	1.0	-		0.66	4.00		2.6
B_{2U} Bearing 2	1.53	8.00	1.00	0.5	-		6.13	2.67		16.3
TOTALS							**6.8**			**19**

Net Shear Force, V = 7 k **Net Moment, M = 35 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 15 k **Factored Moment, M_u = 77 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam** **100 Year Flood**	Client: Town of Rockport Project: Mill Pond Dam Safety Modifications Subject: Stability Analysis Prepared By: M. Hurd — Rev. Date: 4/21/10 Checked By: J. Christensen — Date: 04/23/10	

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	5.00	1.00	1.0	0.150	1.50		2.50		3.8
Soil Weight										
E_D Downstream	2.00	3.00	1.00	1.00	0.130	0.78		3.50		2.7
E_{Df} Stone Façade	13.00	2.00	1.00	1.00	0.130	3.38		1.00		3.4
Water Weight										
W_D Downstream										
TOTALS						**5.7**				**10**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	6.00	5.00	1.00	1.0	0.0624		1.87	2.50	4.7	
U_{2D} Uplift 2										
TOTALS							**1.9**		**5**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	2.19	5.00	1.00	1.0	-		10.94	2.50	27.4	
B_{2D} Bearing 2	0.96	5.00	1.00	0.5	-		2.39	3.33	8.0	
TOTALS							**13.3**		**35**	

Net Shear Force, V = -10 k **Net Moment, M = 30 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -21 k **Factored Moment, M_u = 67 ft-k**

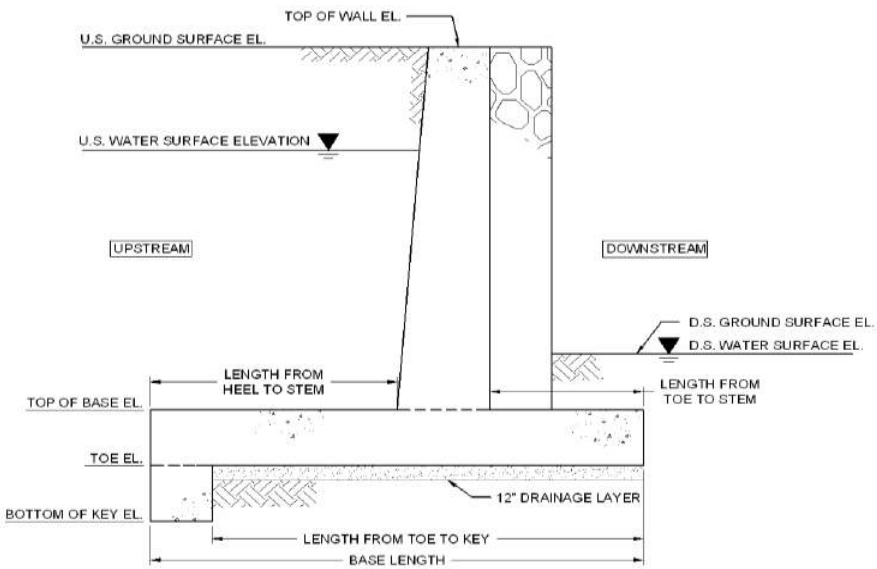
($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOCATION - Right Side of Dam (STA 1+50 - 2+35)

LOAD CASE 3 - Normal Pool+ Earthquake

SIGN CONVENTION & NOTATIONS



V = sum of vertical forces, including uplift

H = sum of horizontal forces

N = total force normal to sliding failure plane

T = total force parallel to sliding failure plane

M = sum of moments about the toe

R = sum of vertical reactions at base = V

e = eccentricity of R with respect to base center line

C = distance to the CG from the toe

α = slope of analyzed failure plane

GIVEN

Geometry

Top of Wall Elevation	29.0	ft	
Toe Elevation	14.0	ft	
Bottom of Key Elevation	12.0	ft	
Top of Base Elevation	16.0	ft	
Length from Heel to Stem	8.0	ft	
Length from Toe to Stem	5.0	ft	
Length from Toe to Key	14.00	ft	
Base Length (B)	16.00	ft	
Base Width (W)	1.00	ft	
Base Area (A)	16.0	ft^2	(W · B)
Base Section Modulus (S)	42.7	ft^3	(W · B^2) / 6

Geotechnical

Ultimate Fdn Bearing Capacity (q_{ult})	12	ksf
Friction Angle at Failure Plane (ϕ_f)	35	degrees
Friction Factor at Failure Plane ($\tan\phi_f$)	0.70	
Slope of Failure Plane (α)	0.0	degrees
Unit Weight of Soil (γ)	0.130	kcf
Bouyant Unit Weight of Soil (γ_b)	0.068	kcf
Cohesion (c)	0	ksf
Soil Friction Angle (ϕ)	32	degrees

Upstream

Water Surface Elevation	27.3	ft	
Ground Surface Elevation	29.0	ft	
Ground Surface Slope (β)	0	degrees	
Wall Friction Angle (δ)	0	degrees	
Back of Wall Angle (θ)	0	degrees	
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)	
Active Soil Pressure Coeff., k_a =	0.31		
Eq. Soil Pressure Coeff. Inc., k_{ae}	0.12		

$$k_a = \frac{\cos(\phi - \theta)^2}{\cos(\theta)^2 \cdot \cos(\theta + \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \beta)}{\cos(\theta + \delta) \cdot \cos(\theta - \beta)}}\right)^2}$$

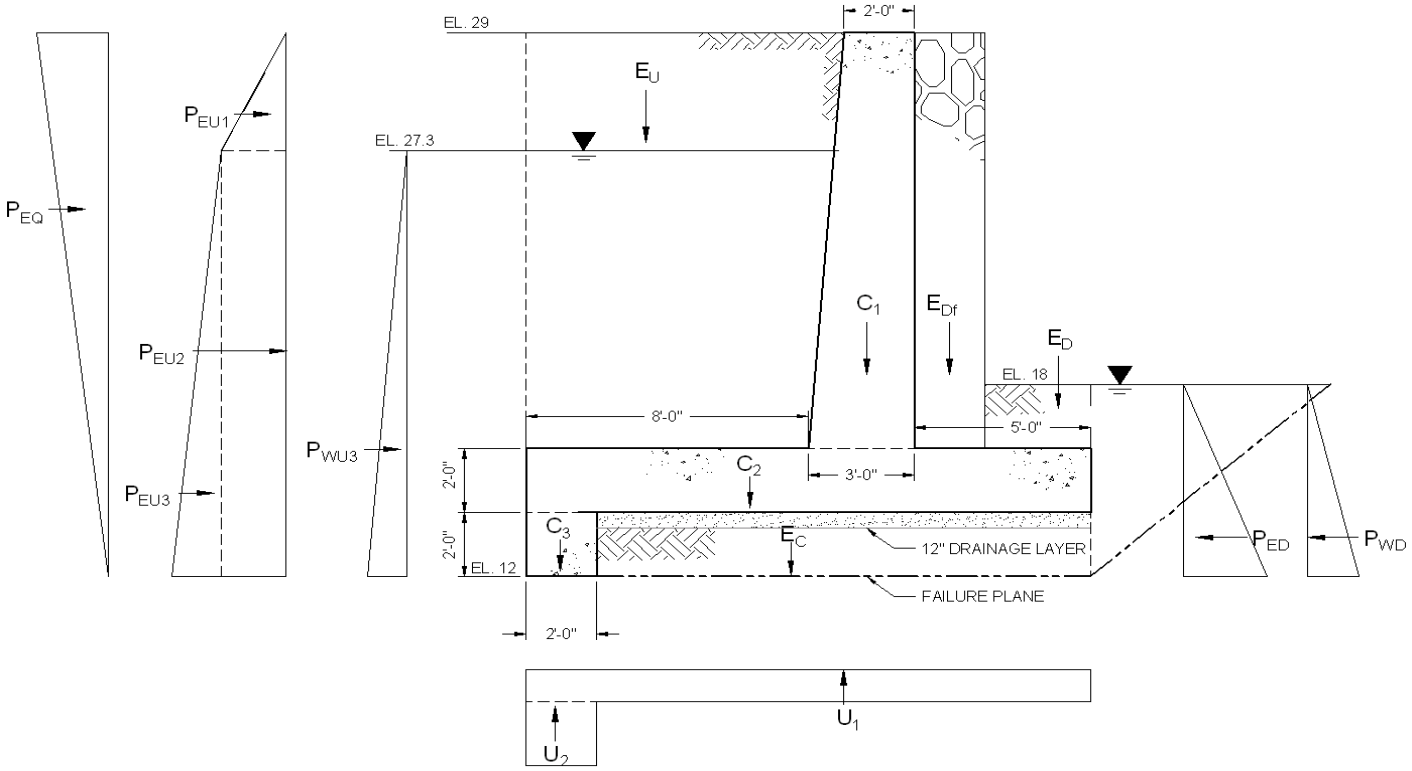
Downstream

Water Surface Elevation	18.0	ft
Ground Surface Elevation	18.0	ft
Ground Surface Slope (β)	0	degrees
Wall Friction Angle (δ)	0	degrees
Back of Wall Angle (θ)	0	degrees
At-Rest Soil Pressure Coeff., k_o =	0.47	(1 - sinϕ) · (1 + sinβ)
Passive Soil Pressure Coeff., k_p =	6.00	

K_p assumed as at rest soil pressure for Seismic Event

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

LOAD DIAGRAMS FOR STABILITY ANALYSES



GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

TABULATION OF FORCES

VERTICAL FORCES

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_1 Stem	13.00	2.00	1.00	1.0	0.150	3.90		6.00	23.4	
C_2 Base	2.00	16.00	1.00	1.0	0.150	4.80		8.00	38.4	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		15.00	9.0	
Soil Weight										
E_C Confined	2.00	14.00	1.00	1.0	0.130	3.64				
E_U Upstream	13.00	8.00	1.00	1.0	0.130	13.52		12.00	162.2	
E_D Downstream	2.00	3.00	1.00	1.0	0.130	0.78		1.50	1.2	
E_{Df} Stone Façade	13.00	2.00	1.00	1.0	0.130	3.38		4.00	13.5	
Water Weight										
W_U Upstream										
W_D Downstream										
TOTALS						**30.6**			**248**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_1 Uplift 1	6.00	16.00	1.00	1.0	0.0624		5.99	8.00		47.9
U_2 Uplift 2	9.30	2.00	1.00	1.0	0.0624		1.16	15.00		17.4
TOTALS							**7.2**			**65**

HORIZONTAL FORCES

Soil Pressure

At-Rest Force = H · W · L · SF · Wt. · K Upstream Ko = 0.47 Downstream Ko = 0.47

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{EU1} Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	13.87		1.2
P_{EU2} Upstream 2	15.30	1.70	1.00	1.0	0.130		1.59	5.65		9.0
P_{EU3} Upstream 3	15.30	15.30	1.00	0.5	0.068		3.72	3.10		11.5
P_{ED} Downstream	6.00	6.00	1.00	0.5	0.068	0.58		0.00	0.0	0.0
TOTALS						**0.6**	**5.4**		**0**	**22**

Water Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{WU} Upstream	15.30	15.30	1.00	0.5	0.0624		7.30	3.10		22.6
P_{WD} Downstream	6.00	6.00	1.00	0.5	0.0624	1.12				
TOTALS						**1.1**	**7.3**		**0**	**23**

Earthquake Loading

K_{ae}= 0.12

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{eq} Dyn. Soil Eq.	17.0	17.0	1	0.5	0.130		2.25 (d)	10.20 (e)		23.0
TOTALS						**0.0**	**2.3**		**0**	**23**

(d.) Soil Force = H · W · L · SF · Wt. · K_{ae}

(e.) Moment arm acts at .6 · H from the base of the dam.

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

SUMMATION

Net Vertical Force, V = 23.5 **k** ↓ **Net Moment About the Toe, M =** 115 **ft-k**

Net Horizontal Force, H = -13.3 **k** →

STABILITY ANALYSIS

LOAD CASE 3 - Normal Pool+ Earthquake

CRITERIA - DCR 302 CMR 10.00 & USACE EM 1110-2-2100

Load Category (Usual, Unusual, or Extreme)	EXTREME	Required $F.S._{sliding}$ =	1.0	
Hazard Classification (High, Significant, Low)	SIGNIFICANT	Required $F.S._{flotation}$ =	1.1	
Site Info (Well Defined, Ordinary, or Limited)	ORDINARY	Max Fdn Bearing Pressure =	6.0	ksf
		Resultant Location Required Within Base		

SLIDING ANALYSIS

At-Rest

	V		H	
Σ Forces =	23.5	k	-13.3	k
	↓		→	

$F.S._{sliding}$ = 1.24 $(V\tan\phi_i + cA) / H$ OK - Meets Sliding Criteria

RESULTANT, FLOATATION, & BEARING ANALYSIS

	V		H		M	
Σ Forces =	23.5	k	-13.3	k	**Σ Moments =** 115	ft-k
	↓		→		↺	

$F.S._{flotation}$ =	4.28			(V↓ / V↑)	OK - Meets Floatation Criteria
Kern between	5.3	ft.	**and** 10.7	ft.	Measured from the Toe
Resultant, R at	4.9	ft. from toe		(M/V)	Resultant DOWNSTREAM of kern
Eccentricity, e =	3.1	ft.		(B/2 - M/V)	Right of Base Centerline
Toe Pressure =	3.17	ksf		(V/A + Ve/S)	OK - Less than Bearing Capacity
Heel Pressure =	-0.24	ksf		(V/A - Ve/S)	OK - Less than Bearing Capacity
	93%	of base in compression			<< Tension in Heel - Crack Develops >>

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor (Dead, Live, and Earthquake):	B_f	1.5	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 d)*
Unusual/Extreme Load Factor:	E_f	0.75	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 16.0 ft (BASE OF STEM)

Soil Pressure **At-Rest** Force = H · W · L · SF · Wt. · K Upstream Ko = 0.47 Downstream Ko = 0.47

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{EU1'}$	Upstream 1	1.70	1.70	1.00	0.5	0.130		0.09	11.87		1.0
$P_{EU2'}$	Upstream 2	11.30	1.70	1.00	1.0	0.130		1.17	5.65		6.6
$P_{EU3'}$	Upstream 3	11.30	11.30	1.00	0.5	0.068		2.03	3.77		7.6
$P_{ED'}$	Downstream	Neglect (per EM 1110-2-2502, Section 3-8)									
	TOTALS						**0.0**	**3.3**		**0**	**15**

Water Pressure

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
$P_{WU'}$	Upstream	11.30	11.30	1.00	0.5	0.0624		3.98	3.77		15.0
$P_{WD'}$	Downstream	Conservatively Neglect									
	TOTALS						**0.0**	**4.0**		**0**	**15**

Earthquake Loading K_{ae}= 0.12

Load		Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft³)	Horizontal Forces (k) ← +	Horizontal Forces (k) → -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
P_{eq}	Dyn. Soil Eq.	13.0	8.5	1	1	0.130		1.72	7.80		13.4
	TOTALS						**0.0**	**1.7**		**0**	**13**

Net Shear Force, V = -9 k **Net Moment, M = -44 ft-k**

Factored Shear Force, V_u = -13 k **Factored Moment, M_u = -64 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

FORCES ON BASE

- UPSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2U} Base	2.00	8.00	1.00	1.0	0.150	2.40		4.00	9.6	
C_3 Key 1	2.00	2.00	1.00	1.0	0.150	0.60		7.00	4.2	
Soil Weight										
E_U Upstream	13.00	8.00	1.00	1.0	0.130	13.52		4.00	54.1	
Water Weight										
W_U Upstream										
TOTALS						**16.5**			**68**	

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1U} Uplift 1	6.00	8.00	1.00	1.0	0.0624		3.00	4.00		12.0
U_{2U} Uplift 2	9.30	2.00	1.00	1.0	0.0624		1.16	7.00		8.1
TOTALS							**4.2**			**20**

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1U} Bearing 1	0.00	8.00	1.00	1.0	-		0.00	4.00		0.0
B_{2U} Bearing 2	1.47	6.88	1.00	0.5	-		5.05	2.29		11.6
TOTALS							**5.0**			**12**

Net Shear Force, V = 7 k **Net Moment, M = 36 ft-k**

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 11 k **Factored Moment, M_u = 53 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants	**Right Side of Dam Earthquake**	Client: Town of Rockport	
		Project: Mill Pond Dam Safety Modifications	
		Subject: Stability Analysis	
		Prepared By: M. Hurd	Rev. Date: 4/21/10
		Checked By: J. Christensen	Date: 04/23/10

FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

Dead Loads

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
Concrete Weight										
C_{2D} Base	2.00	5.00	1.00	1.0	0.150	1.50		2.50		3.8
Soil Weight										
E_D Downstream	2.00	3.00	1.00	1.00	0.130	0.78		3.50		2.7
E_{Df} Stone Façade	13.00	2.00	1.00	1.00	0.130	3.38		1.00		3.4
Water Weight										
W_D Downstream										
TOTALS						**5.7**				**10**

Uplift Pressure

Load	Height (ft)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
U_{1D} Uplift 1	6.00	5.00	1.00	1.0	0.0624		1.87	2.50	4.7	
U_{2D} Uplift 2										
TOTALS							**1.9**		**5**	

Bearing Pressure

Load	Height (k/ft^2)	Width (ft)	Length (ft)	Shape Factor	Weight (k/ft^3)	Vertical Forces (k) ↓ +	Vertical Forces (k) ↑ -	Arm (ft)	Moments (ft-k) ↺ +	Moments (ft-k) ↻ -
B_{1D} Bearing 1	2.11	5.00	1.00	1.0	-		10.53	2.50	26.3	
B_{2D} Bearing 2	1.07	5.00	1.00	0.5	-		2.66	3.33	8.9	
TOTALS							**13.2**		**35**	

Net Shear Force, V = -9 k **Net Moment, M = 30 ft-k**

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -14 k **Factored Moment, M_u = 44 ft-k**

($V_u = V \cdot B_f \cdot E_f \cdot H_f$) ($M_u = M \cdot B_f \cdot E_f \cdot H_f$)

GEI Consultants

Client		Project 07104-1	Page 1
		By M. Hurd	Date 1-4-10
Subject Rockport Mill Pond Dam		Checked PJ Snyder	Date 4/28/10
		Approved	Date

Determine Passive coefficient for downstream side of the dam: using Log Spiral charts

Source: NAVFAC DM 7.02 - Foundations and Earth Structures Pg. 7.2-67

Use following Parameters

Angle of internal friction (ϕ) = 32°

Backfill Slope Angle (β) = 0°

Concrete/soil friction angle D/S side (δ) = -19°
From NAVFAC 7.2-63 (Formed concrete on silty sand-gravel mixture)

$$\frac{\delta}{\phi} = \frac{19}{32} = -.593 \approx -.6$$

Reduction factor R from table on 7.2-67 for $\frac{\delta}{\phi} = -.6$ and $\phi = 32°$

R = .81

Kp from 7.2-67 for $\delta/\phi = -1$ Kp ≈ 7.5

Kp = .81 (7.5)

Kp = 6.075

Use Kp = 6

REDUCTION FACTOR (R) OF K_P FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

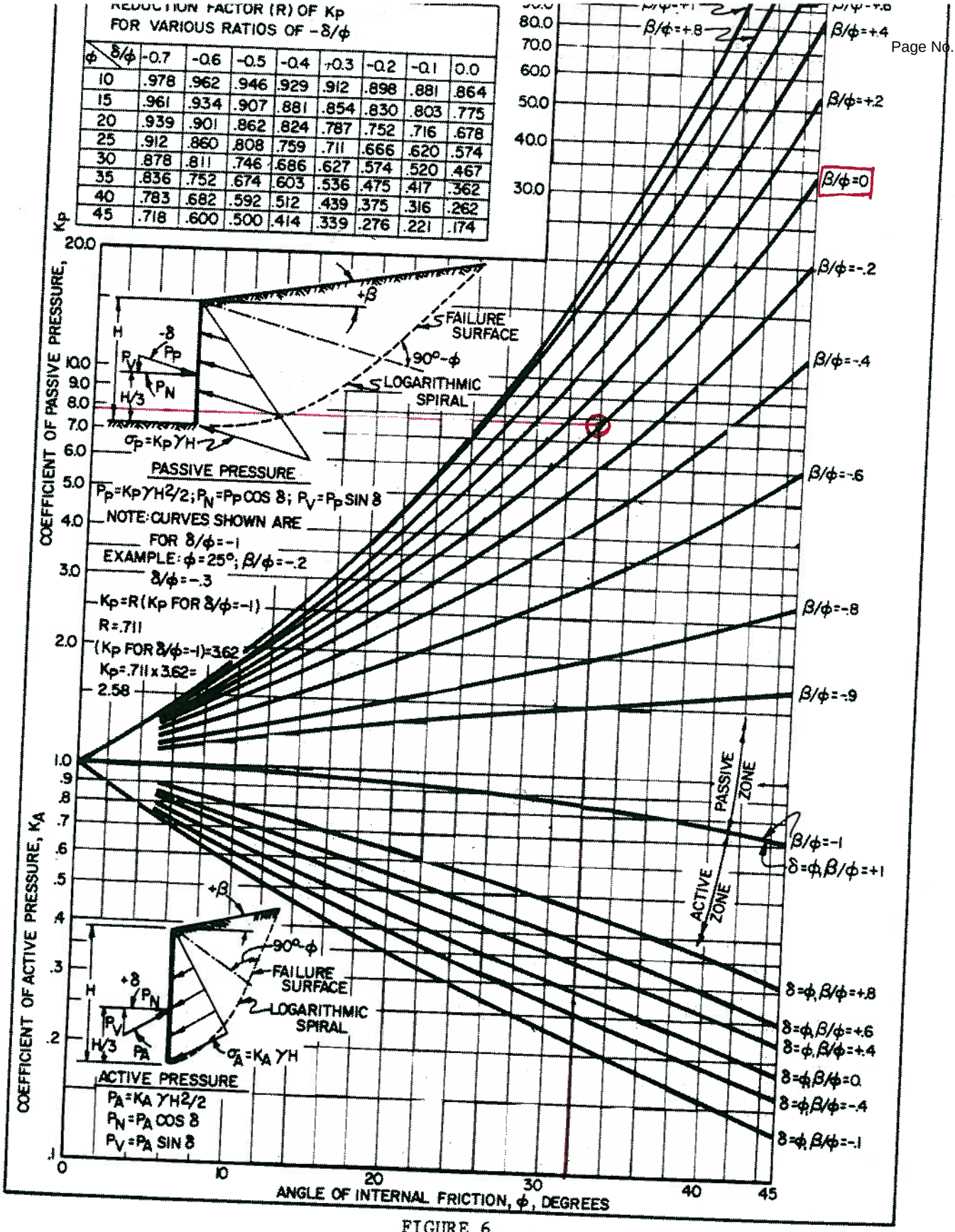
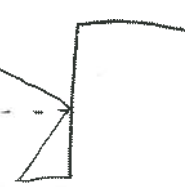


FIGURE 6
Active and Passive Coefficients with Wall Friction
(Sloping Backfill)
7.2-67

PASSIVE FOR SLOPING PASSIVE SURFACE



GEI Consultants

Client	Project 07104-1	Page 1	
	By M. Hurd	Date 1-4-10	
Subject Rockport Mill Pond	Checked J. CHRISTENSEN	Date 4.21.10	
	Approved	Date	

Purpose: Determine earthquake acceleration coefficient for new Dam structure, using ASCE 7-05, 780 CMR 7th Edition, and NEHRP 2003 Commentary using short period spectral response acceleration.

Find S_s for Rockport, MA using Table 1604.10 (780 CMR)

$S_s = .33$

Find site class using 9.4.1.2.1 (780 CMR Ch. 16)

Site Class = D (stiff soil)

$S_{ms} = F_a \times S_s$ (ASCE 7-05 Eq. 11.4-1)

$F_a = 1.5$ for Class D $.33 \leq S_s \leq .5$ (780 CMR Table 9.4.1.2.4A)

$S_{ms} = .33 \times 1.5$

$S_{ms} = .495$

$S_{DS} = 2/3\ S_{ms}$ (ASCE 7-05 Eq. 11.4-3)

$S_{DS} = 2/3 \times .495$

$S_{DS} = .33$

For earth retaining structures use Peak ground acceleration (Pg. 156 NEHRP 2003 Commentary)

$PGA = .4\ S_{DS}$ in this case $K_H = S_{DS} = .33$

$PGA = .132$

Convert PGA from Strength design method to factored load design Method

$PGA_{factored} = .7(PGA)$

$PGA_{factored} = .7(.132) = .0924$

Use PGA = .15 from 302 CMR 10.14 (9.i)

GEI Consultants

Client		Project 07104-1	Page 2
		By M. Hurd	Date 1-4-10
Subject Rockport Mill Pond Dam		Checked	Date
		Approved	Date

Determine the dynamic active component of lateral thrust.

$\Delta K_{AE} = 3/4\ K_h$ C7.5-3a 2003 NEHRP Commentary

$K_h = PGA_{Factored}$

$\Delta K_{AE} = 3/4\,(.15)$

$\Delta K_{AE} = .12$

According to NEHRP Commentary (2003) pg 156 The dynamic thrust resultant should be taken to act .6H above the base of the wall.

BUILDING SEISMIC SAFETY COUNCIL

of the National Institute of Building Sciences

Program on Improved Seismic Safety Provisions

2003 Edition

RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS AND OTHER STRUCTURES (FEMA 450)

Part 2: Commentary

walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

Yielding walls. At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Monobe-Okabe (M-O) seismic coefficient analysis (Monobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1 - k_v) K_{AE} \tag{C7.5-1}$$

where: P_{AE} is the total (static + dynamic) lateral thrust, γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \tag{C7.5-2a}$$

or

$$K_{AE} = K_A + \Delta K_{AE} \tag{C7.5-2b}$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \tag{C7.5-2c}$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} = (3/4) K_h \tag{C7.5-3a}$$

$$\Delta P_{AE} = (1/2)\gamma H^2 (3/4) k_h = (3/8) k_h \gamma H^2 \tag{C7.5-3b}$$

where k_h is horizontal ground acceleration divided by gravitational acceleration. It is recommended that k_h be taken equal to the site peak ground acceleration that is consistent with design earthquake ground motions as determined in *Provisions* Sec. 7.5.2 (that is, $k_h = S_{DS}/2.5$). Eq. C7.5-3a and C7.5-3b generally are referred to as the simplified M-O formulation.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and Whitman (1970) recommended that the resultant dynamic thrust act at $0.6H$ above the base of the wall (that is, inverted trapezoidal pressure distribution).

Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. There are a number of empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by

TABLE 1604.10 GROUND SNOW LOADS; BASIC WIND SPEEDS; EARTHQUAKE DESIGN FACTORS

(For R-3 of three stories or less one- and two-family stand alone buildings, see 780 CMR 53.00 for snow and wind loads)

City/Town	Ground Snow Load p_g, psf	Basic Wind Speed V, MPH	Earthquake Design Factors	
			Ss	S_1
Rochester	45	110	0.23	0.059
Rockland	45	110	0.26	0.064
Rockport	45	110	0.33	0.073
Rowe	65	100	0.22	0.069
Rowley	55	110	0.34	0.075
Royalston	65	100	0.25	0.070
Russell	65	100	0.23	0.066
Rutland	55	100	0.24	0.068
Salem	45	110	0.31	0.071
Salisbury	55	110	0.35	0.077
Sandisfield	65	90	0.23	0.066
Sandwich	35	120	0.22	0.058
Saugus	45	110	0.30	0.070
Savoy	65	90	0.22	0.068
Scituate	45	110	0.27	0.065
Seekonk	55	110	0.24	0.062
Sharon	55	100	0.25	0.065
Sheffield	65	90	0.23	0.066
Shelburne	65	100	0.23	0.068
Sherborn	55	100	0.26	0.066
Shirley	65	100	0.28	0.072
Shrewsbury	55	100	0.25	0.067
Shutesbury	65	100	0.23	0.068
Somerset	55	110	0.23	0.060
Somerville	45	105	0.28	0.069
South Hadley	55	100	0.23	0.066
Southampton	55	100	0.23	0.066
Southborough	55	100	0.26	0.067
Southbridge	55	100	0.23	0.064
Southwick	55	100	0.23	0.065
Spencer	55	100	0.23	0.066
Springfield	55	100	0.23	0.065
Sterling	55	100	0.26	0.069
Stockbridge	65	90	0.22	0.066
Stoneham	45	105	0.30	0.071
Stoughton	55	100	0.26	0.065
Stow	55	100	0.27	0.069
Sturbridge	55	100	0.23	0.065
Sudbury	55	100	0.27	0.069
Sunderland	65	100	0.23	0.068
Sutton	55	100	0.24	0.065
Swampscott	45	110	0.30	0.070
Swansea	55	110	0.24	0.061
Taunton	55	110	0.24	0.062
Templeton	65	100	0.25	0.070
Tewksbury	55	100	0.31	0.073
Tisbury	35	120	0.18	0.052
Tolland	65	100	0.23	0.066
Topsfield	45	110	0.33	0.074
Townsend	65	100	0.28	0.072
Truro	35	120	0.22	0.057

$\overline{N}_{ch}$ is **EQUATION 9.4.1.2-3**

$$\overline{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

whereby $= \sum_{i=1}^{m} d_i = d_s$. (Use only d_i and N_i for cohesionless soils.)

d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m)

$\overline{s}_u$ is the undrained shear strength in psf (kPa), not to exceed 5000 psf (240 kPa), ASTM D2166-91 or D2850-87.

$\overline{s}_u$ is **EQUATION 9.4.1.2-4**

$$\overline{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{di}{s_{ui}}}$$

whereby $\sum_{i=1}^{k} d_i = d_c$.

d_c is the total thickness (100 - d_s) of cohesive soil layers in the top 100 ft (30 m)

Pl is the plasticity index, ASTM D4318-93

w is the moisture content in percent, ASTM D2216-92

Figures 9.4.1.1(a) *through* 9.4.1.1(j)

These figures are not included. Refer to Table 1604.10 of the Massachusetts State Building Code for all references to these figures.

9.4.1.2.4 Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration Parameters. The maximum considered earthquake spectral response acceleration for short periods (S_{MS}) and at 1-sec (S_{M1}), adjusted for site class effects, shall be determined by Equations 9.4.1.2.4-1 and 9.4.1.2.4-2, respectively.

EQUATION 9.4.1.2.4-1

$$S_{MI} = F_a S_s$$

EQUATION 9.4.1.2.4-2

$$S_{m1} = F_v S_1$$

where

S_1 = the tabulated maximum considered earthquake spectral response acceleration at a period of 1-sec as determined in accordance with Section 9.4.1

S_s = the tabulated maximum considered earthquake spectral response acceleration at short periods as determined in accordance with Section 9.4.1

where site coefficients F_a and F_v are defined in Tables 9.4.1.2.4a and b, respectively.

TABLE 9.4.1.2.4a VALUES OF F_a AS A FUNCTION OF SITE CLASS AND SHORT PERIOD MAXIMUM CONSIDERED EARTHQUAKE SPECTRAL ACCELERATION

Site Class	*Tabulated Maximum Considered Earthquake Spectral Response Acceleration at Short Periods*					
	$S_S \le 0.26$	$0.27 \le S_S \le 0.29$	$0.30 \le S_S \le 0.32$	$0.33 \le S_S \le 0.35$	$0.36 \le S_S \le 0.38$	$S_S \ge 0.39$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.2	1.2	1.2	1.2
D	1.6	1.6	1.55	1.5	1.5	1.5
E	2.5	2.4	2.3	2.2	2.1	2.0
F	Note a	Note a	Note a	Note a	Note a	Note a

Note a: Site-specific geotechnical investigation and dynamic site response analyses shall be performed except that for structures with periods of vibration equal to or less than 0.5-seconds, values of F_a for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquefaction in Step 3 of 9.4.1.2.2.

ASCE STANDARD ASCE/SEI 7-05

Including Supplement No. 1

American Society of Civil Engineers

Minimum Design Loads for Buildings and Other Structures

This document uses both the International System of Units (SI) and customary units.

Published by the American Society of Civil Engineers

δ_{xe} = deflection of Level x at the center of the mass at and above Level x determined by an elastic analysis, Section 12.8-6

δ_{xm} = modal deflection of Level x at the center of the mass at and above Level x as determined by Section 19.3.2

$\bar{\delta}_x, \bar{\delta}_{x1}$ = deflection of Level x at the center of the mass at and above Level x, Eqs. 19.2-13 and 19.3-3 (in. or mm)

θ = stability coefficient for P-delta effects as determined in Section 12.8.7

ρ = a redundancy factor based on the extent of structural redundancy present in a building as defined in Section 12.3.4

ρ_s = spiral reinforcement ratio for precast, prestressed piles in Sections 14.2.7.1.6 and 14.2.7.2.6

λ = time effect factor

Ω_0 = overstrength factor as defined in Tables 12.2-1, 5.4-1, and 15.3-1

11.4 SEISMIC GROUND MOTION VALUES

11.4.1 Mapped Acceleration Parameters. The parameters S_S and S_1 shall be determined from the 0.2 and 1.0 s spectral response accelerations shown on Figs. 22-1 through 22-14, respectively. Where S_1, is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A and is only required to comply with Section 11.7.

11.4.2 Site Class. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E, or F in accordance with Chapter 20. Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.

11.4.3 Site Coefficients and Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameters. The MCE spectral response acceleration for short periods (S_{MS}) and at 1 s (S_{M1}), adjusted for Site Class effects, shall be determined by Eqs. 11.4-1 and 11.4-2, respectively.

$$S_{MS} = F_a S_s \tag{11.4-1}$$

$$S_{M1} = F_v S_1 \tag{11.4-2}$$

where

S_s = the mapped MCE spectral response acceleration at short periods as determined in accordance with Section 11.4.1, and

S_1 = the mapped MCE spectral response acceleration at a period of 1 s as determined in accordance with Section 11.4.1

where site coefficients F_a and F_v are defined in Tables 11.4-1 and 11.4-2, respectively. Where the simplified design procedure

TABLE 11.4-1 SITE COEFFICIENT, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

NOTE: Use straight-line interpolation for intermediate values of S_S.

TABLE 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

NOTE: Use straight-line interpolation for intermediate values of S_1.

of Section 12.14 is used, the value of F_a shall be determined in accordance with Section 12.14.8.1, and the values for F_v, S_{MS}, and S_{M1} need not be determined.

11.4.4 Design Spectral Acceleration Parameters. Design earthquake spectral response acceleration parameter at short period, S_{DS}, and at 1 s period, S_{D1}, shall be determined from Eqs. 11.4-3 and 11.4-4, respectively. Where the alternate simplified design procedure of Section 12.14 is used, the value of S_{DS} shall be determined in accordance with Section 12.14.8.1, and the value for S_{D1} need not be determined.

$$S_{DS} = \frac{2}{3} S_{MS} \tag{11.4-3}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{11.4-4}$$

11.4.5 Design Response Spectrum. Where a design response spectrum is required by this standard and site-specific ground motion procedures are not used, the design response spectrum curve shall be developed as indicated in Fig. 11.4-1 and as follows:

1. For periods less than T_0, the design spectral response acceleration, S_a, shall be taken as given by Eq. 11.4-5:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \tag{11.4-5}$$

2. For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration, S_a, shall be taken equal to S_{DS}.

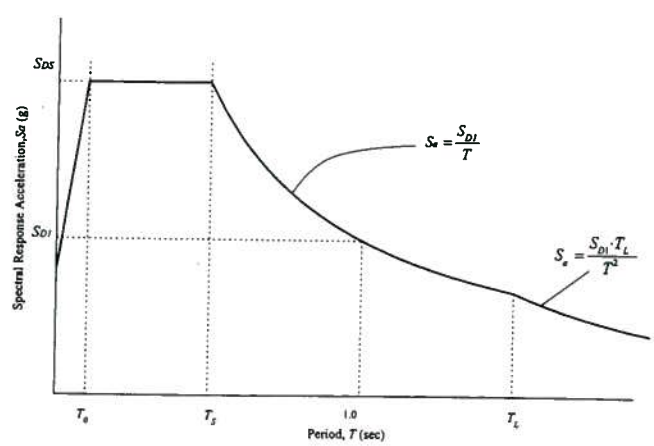


FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM

10.14: continued

SLOPE STABILITY ANALYSIS
MINIMUM FACTORS OF SAFETY

Loading Condition	Minimum Factor of Safety Analyzed	Slope to be
End of construction condition	1.3	upstream and downstream
Sudden drawdown from maximum pool	>1.1*	upstream
Sudden drawdown from spillway crest or top of gates	1.2	upstream
Steady seepage with maximum storage pool	1.5	upstream and downstream
Steady seepage with surcharge pool	1.4	downstream
Earthquake (for steady seepage conditions with seismic loading using seismic coefficient method)	>1.0	upstream and downstream

(d) Foundation bearing capacity and sliding base analysis shall be considered for all dams and are required for high hazard dams. Where bearing capacity or sliding base analysis is required, documentation of assumptions, computations, and safety factors shall be included in the final design report.

(e) Resistance of appurtenant structures against flotation uplift shall be provided for all dams. If the structures are anchored by dead weight alone, the buoyant weight shall be used for analysis. If the structures are anchored to soil or rock, the minimum factor of safety for that portion of the resistance provided by soil or rock anchorage shall be 2.0 unless the design engineer provides a thoroughly documented basis for using a lower safety factor.

(f) For concrete, masonry, or other similar dams of relatively narrow cross section, resistance against overturning and sliding under maximum design loading conditions shall be considered; overturning and sliding stability computations shall be required for significant and high hazard dams.

(g) The anticipated reservoir and tailwater drawdown conditions shall be considered in all stability computations and shall be included in the design documents provided in the final design report.

(h) The slopes shall be protected against erosion by wave action, and the crest and downstream slope shall be protected against erosion due to wind and rain. Riprap and other erosion protection shall be provided over the full range in stage between the lowest drawdown elevation and at least two feet above maximum water storage elevation. Exemptions for specific site conditions, slowly rising reservoirs, such as waste storage facilities, may be approved in writing by the Commissioner upon written request by the Applicant.

(i) All significant and high hazard potential dams shall be designed to withstand seismic accelerations of the following intensities: Zone 1 = 0.025 g., Zone 2 = 0.05 g., Zone 3 = 0.15 g. Zones refer to "Geologic Hazard Maps".

(j) Loading Combinations. The following conditions and requirements are suitable in general for gravity dams of intermediate size. Loads which are not indicated such as wave action, or any unusual loadings should be considered where applicable.

*The factor of safety shall not be less than 1.5 when drawdown rate and pore water pressures developed from flow nets are used in the stability analyses and where rapid drawdown is a normal operating condition as with pumped storage reservoir.

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STRENGTH DESIGN

GIVEN			
Load Factors:			
Basic Load Factor (Dead, Live, and Earthquake):	B_f	1.7	*(EM 1110-2-2104, Strength Design for Reinforced-Concrete Hydraulic Structures, 1992, Section 3-3 d)*
Unusual/Extreme Load Factor:	E_f	0.75	
Hydraulic Load Factor:	H_f	1.3	
Strength Reduction Factors:			
Tension-Controlled Sections:	Φ_t	0.90	*(ACI 318-08, Section 9.3.2)*
Shear:	Φ_v	0.75	
Material Properties:			
28-Day Compressive Strength of Concrete:	f'_c	4000	psi
Ultimate Compressive Strain of Concrete:	ε_u	0.003	*(EM 1110-2-2104, Section 4.1.a)*
Modulus of Elasticity of Steel:	E_s	29E+6	psi
Yield Strength of Reiforcing Steel:	f_y	60	ksi

FORCES ON STEM

AT ELEVATION 16.0 ft (BASE OF STEM)

Factored Shear Force, V_u = -21 k **Factored Moment, M_u = -103 ft-k**

DESIGN STEM REINFORCEMENT

Stem Dimensions and Reinforcement:

Length of Wall for Analysis:	b	12	in	
Minimum Thickness of Stem:	t_{min}	24	in	*(at top of stem)*
Maximum Thickness of Stem:	t_{max}	36	in	*(at base of stem)*
Clear Cover on Vertical Bars:	clr.	4	in	
Bar Size of Tension Reinforcement:	No.	9	bars	
Diameter of Tension Bar:	d_b	1.128	in	
Spacing of Tension Bars:	s	12	in	
Area of Tension Reinforcment per Length of Wall:	A_s	1.00	in^2	
Minimum Depth to Centroid of Tension Reinforcement:	d_{min}	19.44	in	*($d_{min} = t_{min}$ - clr. - $d_b/2$)*
Maximum Depth to Centroid of Tension Reinforcement:	d_{max}	31.44	in	*($d_{max} = t_{max}$ - clr. - $d_b/2$)*

Check Shear Strength:

From ACI 318-08, Section 11.2.1.1:

$V_c := 2bd \cdot \sqrt{f'_c}$ $V_c =$ 48 k

$\Phi V_n := \Phi_v * V_c$ $\Phi V_n =$ 36 k > $V_u =$ 21 k

OK - No shear reinforcement required

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Check Flexural Strength:

Depth to Neutral Axis:

$c := (A_s \cdot f_y) / (0.85 \cdot f'_c \cdot \beta_1 \cdot b)$ $\quad c = 1.73$ in

Moment Capacity: (at base of stem)

$M_n := A_s \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))$ $\quad M_n = 154$ ft·k

$\Phi M_n := \Phi_t \cdot M_n$ $\quad \Phi M_n = 138$ ft·k $\quad > \quad M_u = 103$ ft·k

OK - Adequate Flexural Strength

Use #9 bars at 12" o.c., upstream face of stem.

Check Ductility:

From ACI 318-08, Section 10.3.5:

Strain in Steel:

$\varepsilon_s := \varepsilon_u \cdot (d_{min} - c) / c$ $\quad \varepsilon_s = 0.031$

$\varepsilon_y := f_y / E_s$ $\quad \varepsilon_y = 0.002$

$\varepsilon_s > \varepsilon_y$

OK - Behavior is ductle

Check the Reinforcement Ratio:

From ACI 318-08, Section 10.2.7.3:

Actual Reinforcement Ratios:

$\rho_{(min)} := A_s / bd_{max}$ $\quad \rho_{(min)} = 0.003$

$\rho_{(max)} := A_s / bd_{min}$ $\quad \rho_{(max)} = 0.004$

Balanced Reinforcement Ratio:

$\rho_b := 0.85 \cdot \beta_1 \cdot (f'_c / f_y) \cdot (87 / (87 + f_y))$ $\quad$ where $\beta_1 = 0.85$

(EM 1110-2-2104, Ex. C-2) $\quad \rho_b = 0.029$

Minimum Tension Reinforcement:

Determine Minimum A_s Required for Flexure:

$A_{s\,(min)} := M_u / [\Phi_t \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))]$ $\quad A_{s\,(min)} = 0.75$ in^2

From ACI 318-08, Section 10.5.3: $\quad 1.33 \cdot A_{s\,(min)} = 0.99$ in^2

$A_s > 1.33 \cdot A_{s\,(min)}$ $\quad \therefore$ need not apply 10.5.1

From ACI 318-08, Section 10.5.1:

Minimum Reinforcement Ratio:

$\rho_{min} := \max \left| \begin{matrix} 3 \cdot \sqrt{f'_c} / f_y \\ 200 / f_y \end{matrix} \right.$ $\quad \rho_{min} = 0.003$

Actual Reinforcement Ratio: $\quad \rho_{(min)} = 0.003$ $\quad$ **Requirements of 10.5.1 waived by 10.5.3**

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Maximum Tension Reinforcement:

From EM 1110-2-2104, Section 3-5:

Recommended Limit: $\rho \leq 0.25\ \rho_b$

Maximum Without "Special Study": $\rho \leq 0.375\ \rho_b$

Actual Reinforcement Ratio: $\rho_{(max)} = 0.15\ \rho_b$ **OK - Less than recommended limit**

Temperature & Shrinkage Reinforcement:

From EM 1110-2-2104, Section 2-8:

Total for two faces: $A_{st} := 0.0028 \cdot A_g$

$A_{st} = 1.21\ in^2$

Per face: $A_{st} / 2 = 0.60\ in^2$ **Use #7 bars at 12" o.c. (0.60in^2)**

Development Length for Wall Reinforcement into Footing:

Development Length for Standard Hooks in Tension

From ACI 318-08, Section 12.5:

For uncoated bars: $\Psi_e := 1.0$

For normal weight concrete: $\lambda := 1.0$

Basic Development Length: $l_{dh'} := 0.02 \cdot \Psi_e \cdot \lambda \cdot (f_y / \sqrt{f'_c}) \cdot d_b$

$l_{dh'} = 21.4\ in$

Reduce Required Development Length

From ACI 318-08, Section 12.5.3:

For bars smaller the No. 11 with side cover
greater than 2.5", and end cover greater than 2":

$l_{dh} := 0.7 \cdot l_{dh'}$ $l_{dh} = 15.0\ in$ **Provide minimum 15 in. development length**

Splice Length for Wall Reinforcement Above Footing:

Basic Development Length

From ACI 318-08, Section 12.2:

Conservatively assume no transverse reinforcement: $K_{tr} = 0$

Minimum Cover and Spacing of Bars:

Cover (to center of bar): $c' := clr. + 0.5d_b$ $c' = 4.56$ in.

Half Center-to-Center Spacing: $s' := s / 2$ $s' = 6.00$ in.

$c_b := \min (c' , s')$ $c_b = 4.56$ in.

Limit $(c_b + K_{tr}) / d_b < 2.5$: $(c_b + K_{tr}) / d_b = 2.5$

Reinforcement Location Factor:
For vertical reinforcemtent: $\Psi_t := 1.0$

Reinforcement Coating Factor:
For uncoated reinforcement: $\Psi_e := 1.0$

$\Psi_t \cdot \Psi_e := \min (\Psi_t \cdot \Psi_e , 1.7)$ $\Psi_t \cdot \Psi_e = 1.0$

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Bar Size Factor:

$$\psi_s := \begin{cases} 0.8 & \text{if No. 6 bars or smaller} \\ 1.0 & \text{otherwise} \end{cases} \qquad \psi_s = 1.0$$

Lightweight Concrete Factor:

For normal weight concrete: $\lambda := 1.0$

Basic Development Length:

$$L_d := \max\left[12in, \left(\frac{3}{40}\cdot\frac{f_y}{\sqrt{f'_c\cdot psi}}\cdot\frac{\psi_t\cdot\psi_e\cdot\psi_s\cdot\lambda}{\min\left(\frac{c_b+K_{tr}}{d_b}, 2.5\right)}\right)\cdot d_b\right] \qquad L_d = 32.1 \text{ in.}$$

Lap Splice Length

From ACI 318-08, Section 12.15:

Use Class B Splice: $L_{splice} := 1.3 \cdot L_d$ $\qquad L_{splice} = 41.7$ in.

Provide 42 in. minimum splice length

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FORCES ON BASE

- UPSTREAM SIDE OF STEM -

<< TOP FACE IN TENSION >>

Factored Shear Force, V_u = 15 **k** **Factored Moment, M_u =** 77 **ft-k**

DESIGN BASE REINFORCEMENT

- UPSTREAM SIDE OF STEM -

Base Dimensions and Reinforcement:

Length of Base for Analysis:	b	12	in	
Thickness of Base:	t	24	in	
Clear Cover on Vertical Bars:	clr.	4	in	
Bar Size of Tension Reinforcement:	No.	9	bars	
Diameter of Tension Bar:	d_b	1.128	in	
Spacing of Tension Bars:	s	12	in	
Area of Tension Reinforcment per Length of Wall:	A_s	1.00	in^2	
Depth to Centroid of Tension Reinforcement:	d	19.44	in	*(d = t - clr. - d_b/2)*

Check Shear Strength:

From ACI 318-08, Section 11.2.1.1:

$V_c := 2bd \cdot \sqrt{f'_c}$ $V_c =$ 30 k

$\Phi V_n := \Phi_v * V_c$ $\Phi V_n =$ 22 k > $V_u =$ 15 k **OK**

Check Flexural Strength:

Depth to Neutral Axis:

$c := (A_s \cdot f_y) / (0.85 \cdot f'_c \cdot \beta_1 \cdot b)$ $c =$ 1.73 in

Moment Capacity:

$M_n := A_s \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))$ $M_n =$ 93 ft·k

$\Phi M_n := \Phi_t \cdot M_n$ $\Phi M_n =$ 84 ft·k > $M_u =$ 77 ft·k **OK**

Use #9 bars at 12" o.c., top face of footing, upstream of stem.

Check Ductility:

From ACI 318-08, Section 10.3.5:

Strain in Steel:

$\varepsilon_s := \varepsilon_u \cdot (d - c) / c$ $\varepsilon_s =$ 0.031

$\varepsilon_y := f_y / E_s$ $\varepsilon_y =$ 0.002

ε_s > ε_y

OK - Behavior is ductle

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Check the Reinforcement Ratio:

From ACI 318-08, Section 10.2.7.3:

Actual Reinforcement Ratio:

$\rho := A_s / bd$ $\rho = 0.004$

Balanced Reinforcement Ratio:

$\rho_b := 0.85 \cdot \beta_1 \cdot (f'_c / f_y) \cdot (87 / (87 + f_y))$ where $\beta_1 = 0.85$

(EM 1110-2-2104, Ex. C-2) $\rho_b = 0.029$

Minimum Tension Reinforcement:

Determine Minimum A_s Required for Flexure:

$A_{s\,(min)} := M_u / [\Phi_t \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))]$ $A_{s\,(min)} = 0.91$ in^2

From ACI 318-08, Section 10.5.3: $1.33 \cdot A_{s\,(min)} = 1.22$ in^2

$A_s < 1.33 \cdot A_{s\,(min)}$ ∴ must meet 10.5.1

From ACI 318-08, Section 10.5.1:

Minimum Reinforcement Ratio:

$\rho_{min} := \max \left| \begin{matrix} 3 \cdot \sqrt{f'_c} / f_y \\ 200 / f_y \end{matrix} \right.$ $\rho_{min} = 0.003$

Actual Reinforcement Ratio: $\rho = 0.004$ **OK - Exceeds minimum reinforcemet required**

Maximum Tension Reinforcement:

From EM 1110-2-2104, Section 3-5:

Recommended Limit: $\rho \leq 0.25\ \rho_b$

Maximum Without "Special Study": $\rho \leq 0.375\ \rho_b$

Actual Reinforcement Ratio: $\rho = 0.15\ \rho_b$ **OK - Less than recommended limit**

Temperature & Shrinkage Reinforcement:

From EM 1110-2-2104, Section 2-8:

Total for two faces: $A_{st} := 0.0028 \cdot A_g$

$A_{st} = 0.81$ in^2

Per face: $A_{st} / 2 = 0.40$ in^2 **Use #6 bars at 12" o.c. (0.44 in^2)**

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Splice Length for Footing Reinforcement:

Basic Development Length

From ACI 318-08, Section 12.2:

Conservatively assume no transverse reinforcement: $K_{tr} = 0$

Minimum Cover and Spacing of Bars:

Cover (to center of bar): $c' := clr. + 0.5d_b$ $c' = 4.56$ in.

Half Center-to-Center Spacing: $s' := s / 2$ $s' = 6.00$ in.

$c_b := \min(c', s')$ $c_b = 4.56$ in.

Limit $(c_b + K_{tr}) / d_b < 2.5$: $(c_b + K_{tr}) / d_b = 2.5$

Reinforcement Location Factor:

For vertical reinforcemtent: $\Psi_t := 1.0$

$\Psi_t \cdot \Psi_e := \min(\Psi_t \cdot \Psi_e, 1.7)$

Reinforcement Coating Factor:

For uncoated reinforcement: $\Psi_e := 1.0$

$\Psi_t \cdot \Psi_e = 1.0$

Bar Size Factor:

$$\Psi_s := \begin{cases} 0.8 & \text{if No. 6 bars or smaller} \\ 1.0 & \text{otherwise} \end{cases}$$

$\Psi_s = 1.0$

Lightweight Concrete Factor:

For normal weight concrete: $\lambda := 1.0$

Basic Development Length:

$$L_d := \max\left[12in, \left(\frac{3}{40} \cdot \frac{f_y}{\sqrt{f'_c \cdot psi}} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s \cdot \lambda}{\min\left(\frac{c_b + K_{tr}}{d_b}, 2.5\right)}\right) \cdot d_b\right]$$

$L_d = 32.1$ in.

Lap Splice Length

From ACI 318-08, Section 12.15:

Use Class B Splice: $L_{splice} := 1.3 \cdot L_d$ $L_{splice} = 41.7$ in.

Provide 42 in. minimum splice length

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FORCES ON BASE

- DOWNSTREAM SIDE OF STEM -

<< BOTTOM FACE IN TENSION >>

Factored Shear Force, V_u = -21 k **Factored Moment, M_u = 67 ft-k**

DESIGN BASE REINFORCEMENT

- DOWNSTREAM SIDE OF STEM -

Base Dimensions and Reinforcement:

Length of Wall for Analysis:	b	12	in	
Thickness of Stem:	t	24	in	
Clear Cover on Vertical Bars:	clr.	4	in	
Bar Size of Tension Reinforcement:	No.	9	bars	
Diameter of Tension Bar:	d_b	1.13	in	
Spacing of Tension Bars:	s	12	in	
Area of Tension Reinforcment per Length of Wall:	A_s	1.00	in^2	
Depth to Centroid of Tension Reinforcement:	d	19.44	in	*(d = t - clr. - d_b/2)*

Check Shear Strength:

From ACI 318-08, Section 11.2.1.1:

$V_c := 2bd \cdot \sqrt{f'_c}$ V_c = 30 k

$\Phi V_n := \Phi_v * V_c$ ΦV_n = 22 k > V_u = 21 k **OK**

Check Flexural Strength:

Depth to Neutral Axis:

$c := (A_s \cdot f_y) / (0.85 \cdot f'_c \cdot \beta_1 \cdot b)$ c = 1.73 in

Moment Capacity:

$M_n := A_s \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))$ M_n = 94 ft·k

$\Phi M_n := \Phi_t \cdot M_n$ ΦM_n = 84 ft·k > M_u = 67 ft·k **OK**

Use #9 bars at 12" o.c., bottom face of footing, downstream of stem.

Check Ductility:

From ACI 318-08, Section 10.3.5:

Strain in Steel:

$\varepsilon_s := \varepsilon_u \cdot (d - c) / c$ ε_s = 0.031

$\varepsilon_y := f_y / E_s$ ε_y = 0.002

ε_s > ε_y

OK - Behavior is ductle

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Check the Reinforcement Ratio:

From ACI 318-08, Section 10.2.7.3:

Actual Reinforcement Ratio:

$\rho := A_s / bd$ $\rho = 0.004$

Balanced Reinforcement Ratio:

$\rho_b := 0.85 \cdot \beta_1 \cdot (f'_c / f_y) \cdot (87 / (87 + f_y))$ where $\beta_1 = 0.85$

(EM 1110-2-2104, Ex. C-2) $\rho_b = 0.029$

Minimum Tension Reinforcement:

Determine Minimum A_s Required for Flexure:

$A_{s\,(min)} := M_u / [\Phi_t \cdot f_y \cdot (d - (\beta_1 \cdot c / 2))]$ $A_{s\,(min)} = 0.80$ in^2

From ACI 318-08, Section 10.5.3: $1.33 \cdot A_{s\,(min)} = 1.06$ in^2

$A_s > 1.33 \cdot A_{s\,(min)}$ ∴ need not apply 10.5.1

From ACI 318-08, Section 10.5.1:

Minimum Reinforcement Ratio:

$\rho_{min} := \max \left| \begin{matrix} 3 \cdot \sqrt{f'_c} / f_y \\ 200 / f_y \end{matrix} \right.$ $\rho_{min} = 0.003$

Actual Reinforcement Ratio: $\rho = 0.004$ **Requirements of 10.5.1 waived by 10.5.3**

Maximum Tension Reinforcement:

From EM 1110-2-2104, Section 3-5:

Recommended Limit: $\rho \le 0.25\ \rho_b$

Maximum Without "Special Study": $\rho \le 0.375\ \rho_b$

Actual Reinforcement Ratio: $\rho = 0.15\ \rho_b$ **OK - Less than recommended limit**

Temperature & Shrinkage Reinforcement:

From EM 1110-2-2104, Section 2-8:

Total for two faces: $A_{st} := 0.0028 \cdot A_g$

$A_{st} = 0.81$ in^2

Per face: $A_{st} / 2 = 0.40$ in^2 **Use #6 bars at 12" o.c. (0.44 in^2)**

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Splice Length for Footing Reinforcement:

Basic Development Length

From ACI 318-08, Section 12.2:

Conservatively assume no transverse reinforcement: $K_{tr} = 0$

Minimum Cover and Spacing of Bars:

Cover (to center of bar): $c' := clr. + 0.5d_b$ $c' = 4.56$ in.

Half Center-to-Center Spacing: $s' := s / 2$ $s' = 6.00$ in.

$c_b := \min(c', s')$ $c_b = 4.56$ in.

Limit $(c_b + K_{tr}) / d_b < 2.5$: $(c_b + K_{tr}) / d_b = 2.5$

Reinforcement Location Factor:

For vertical reinforcemtent: $\Psi_t := 1.0$

$\Psi_t \cdot \Psi_e := \min(\Psi_t \cdot \Psi_e, 1.7)$

Reinforcement Coating Factor:

For uncoated reinforcement: $\Psi_e := 1.0$

$\Psi_t \cdot \Psi_e = 1.0$

Bar Size Factor:

$$\Psi_s := \begin{cases} 0.8 & \text{if No. 6 bars or smaller} \\ 1.0 & \text{otherwise} \end{cases}$$

$\Psi_s = 0.8$

Lightweight Concrete Factor:

For normal weight concrete: $\lambda := 1.0$

Basic Development Length:

$$L_d := \max\left[12in, \left(\frac{3}{40} \cdot \frac{f_y}{\sqrt{f'_c \cdot psi}} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s \cdot \lambda}{\min\left(\frac{c_b + K_{tr}}{d_b}, 2.5\right)}\right) \cdot d_b\right]$$

$L_d =$ **26** **in.**

Lap Splice Length

From ACI 318-08, Section 12.15:

Use Class B Splice: $L_{splice} := 1.3 \cdot L_d$ $L_{splice} = 33.3$ in.

Provide 34 in. minimum splice length

GEI Consultants

	Project 071041	Page 1	
Client TOWN OF ROCKPORT	By E. TURGEON	Date 2/1/10	
Subject MILL POND DAM - CONCRETE STRUCTURES	Checked M. Hurd	Date 4/14/10	
	Approved	Date	

PURPOSE - DESIGN REINFORCING DETAILS FOR VARIOUS COMPONANTS OF DAM CONCRETE COMPONANTS.

REFERENCES -
1. ATTACHED GEI PRELIMINARY DRAWINGS (4)
2. STABILITY/STRENGTH CALCS BY E. TURGEON DATED 01/07/10
3. MASSACHUSETTS BUILDING CODE 780 CMR
4. ACI 318-08
5. EM 1110-2-2104

① CHECK SHEAR FRICTION AT CONST JOINT BETWEEN STEMS AND FOOTING FOR CANTILEVER T-WALL SECTIONS.

FROM ACI 318-08, 11.6 - $V_n = A_{vf} f_y \mu$

AND $\phi V_n > V_u$ WHERE $\phi_v = 0.75$

FROM REF. 2 -

MAX V_u @ BASE OF STEM

TYPICAL SECTION RIGHT SIDE OF DAM - MCH 21 ~~16~~ K PER FOOT OF WALL

TYPICAL SECTION LEFT SIDE OF DAM - 16 ~~12~~ K PER FOOT OF WALL

FROM ACI 318-08, 11.6.4.3 - $\mu = 0.6\lambda$ FOR CONC. PLACED AGAINST HARDENED SURFACE NOT ROUGHENED

$\lambda = 1.0$ FOR NORMAL WT. CONC.

$f_y \leq 60$ ksi

So, $\phi V_n = (0.75)(A_{vf})(60\text{ ksi})(0.6) = 27 A_{vf}$

@ RIGHT SIDE OF DAM $27 A_{vf} > \cancel{16}^{21}$ K $\Rightarrow A_{vf} > \cancel{0.59}^{0.77}$ in²

@ LEFT SIDE OF DAM $27 A_{vf} > \cancel{12}^{16}$ K $\Rightarrow A_{vf} > \cancel{0.44}^{0.59}$ in²

CHECK AGAINST A_s PROVIDE FOR FLEXURAL STRENGTH... (REF. 2)

@ RIGHT SIDE OF DAM $\Rightarrow$ No. 8 @ 12" - $A_s = 0.79$ in²/FT WALL

@ LEFT SIDE OF DAM $\Rightarrow$ ~~No. 6 @ 9"~~ No. 8 @ 12" - A_s = ~~0.59~~ .79 in²/FT WALL NCH

∴ CARRYING TENSION REINFORCEMENT ACROSS CONST. JOINT WILL PROVIDE ADEQUATE SHEAR FRICTION CAPACITY.

GEI Consultants

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	By E Turgeon	Date 2/1/10	
Subject Mill Pond Dam – Concrete Structures	Checked M. Hurd	Date 4/14/10	
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CHECK MAX V_n PER ACI 11.6.5 –

$V_n < 0.2 f'_c A_c$, $800 A_c$ WHERE $f'_c = 4000$ psi

$(0.2 f'_c = 800)$

$A_c = 3' \times 1' = 3 FT^2$

$V_n < 0.2(4^k/in^2)(3 FT^2 \cdot \frac{(12)^2 in^2}{1 FT^2}) = 345^k$

$345^k >> V_n = (0.59 in^2, 0.44 in^2)(60 ksi)(0.6) = 21^k, 16^k$ OK

② DOWNSTREAM HANDRAIL SUPPORTS

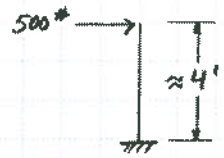
LOADS FROM REF. 3, SECTION 1607.7

$= 50$ lb/ft OR 200 lb. AT TOP

MAX SUPPORTED SPAN ≈ 10'

50 lb/ft $\times 10' = 500$ lb. > 200 lb.

DESIGN SUPPORTS FOR 500 lb. FORCE AT TOP HANDRAIL



$M = 500^\# \times 4' = 2000^{\#\text{-}FT}$ OR 2.0^{KFT}

$V = 500^\#$ OR 0.5^k

$M_u = 1.6 \times 2.0^{k\text{-}FT} = 3.2$ k-FT

$V_u = 1.6 \times 0.5^k = 0.8^k$

REINF. CONC. CAPACITY –

FLEXURE – TRY 2-No. 4 BARS

$c = A_s f_y / 0.85 f'_c \beta_1 b = 0.4 in^2 (60^k/in^2) / 0.85(4^k/in^2)(0.85)(12")$

$= 0.69"$

$\phi M_n = 0.9 A_s f_y (d - \frac{\beta_1 c}{2}) = 0.9(0.4 in^2)(60^k/in^2)(8.5 in - \frac{0.85(0.69 in)}{2})$

$= 177$ k-in

$= 14.8$ k-FT > $M_u = 3.2$ k-FT OK

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CHECK MIN. STEEL PER ACI 10.5.1

$$\rho_{min} = \max \begin{cases} 3\sqrt{f'_c}/f_y = 3\sqrt{4000}/60{,}000 = 0.0032 \\ 200/f_y = 200/60{,}000 = 0.0033 \leftarrow \end{cases}$$

$$\rho_{actual} = A_s/bd = 0.4\ in^2/(12\ in)(8.5\ in) = 0.0039 > \rho_{min}$$

OK

SHEAR –

$$\phi V_c = 0.75(2)(b)(d)\sqrt{f'_c} = 0.75(2)(12in)(8.5in)(\sqrt{4000})$$

$$= 9.7^k >> 0.8^k \quad \text{OK}$$

ASSUME CONSTRUCTION JOINT AT TOP OF CORBEL / BASE OF 12" POST

↳ CHECK SHEAR FRICTION

$$\phi V_n = 0.75\, A_{vf}(f_y)(\mu)$$

$$= 0.75(A_{vf})(60^k/in^2)(0.6) > 0.8^k$$

$$A_{vf} > 0.03\ in^2 \qquad 1\ \#4\ BAR = 0.2\ in^2$$

DETERMINE MIN. DEVELOPMENT LENGTH –

FROM ACI 12.2.3

$$\ell_d = \left(\frac{3}{40}\frac{f_y}{\lambda\sqrt{f'_c}}\frac{\psi_t\psi_e\psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b \quad \text{WHERE } \psi_t = 1.0 \quad \psi_s = 0.8$$

$$\psi_e = 1.0 \quad \lambda = 1$$

$$= \left[\frac{3(60{,}000\ lb/in^2)(1)(1)(0.8)}{40(1)(\sqrt{4000}\ lb/in^2)\left(\frac{3in+0}{0.5in}\right)}\right](0.5") \quad c_b = 3" \quad K_{tr} = 0$$

$$= 4.75" < \text{MIN. ALLOWED} = 12" \quad \text{USE } \cancel{12"}$$

1'-7"

MCH

IF HOOKED – ACI 12.5

$$\ell_{dh} = 0.7\left[(0.02)(\psi_e)(f_y)/\lambda\sqrt{f'_c}\right](d_b)$$

$$= (0.7)\left[(0.02)(1)(60{,}000\ lb/in^2)/(1)\sqrt{4000}\ lb/in^2\right](0.5") = \cancel{6.6"}$$

9"

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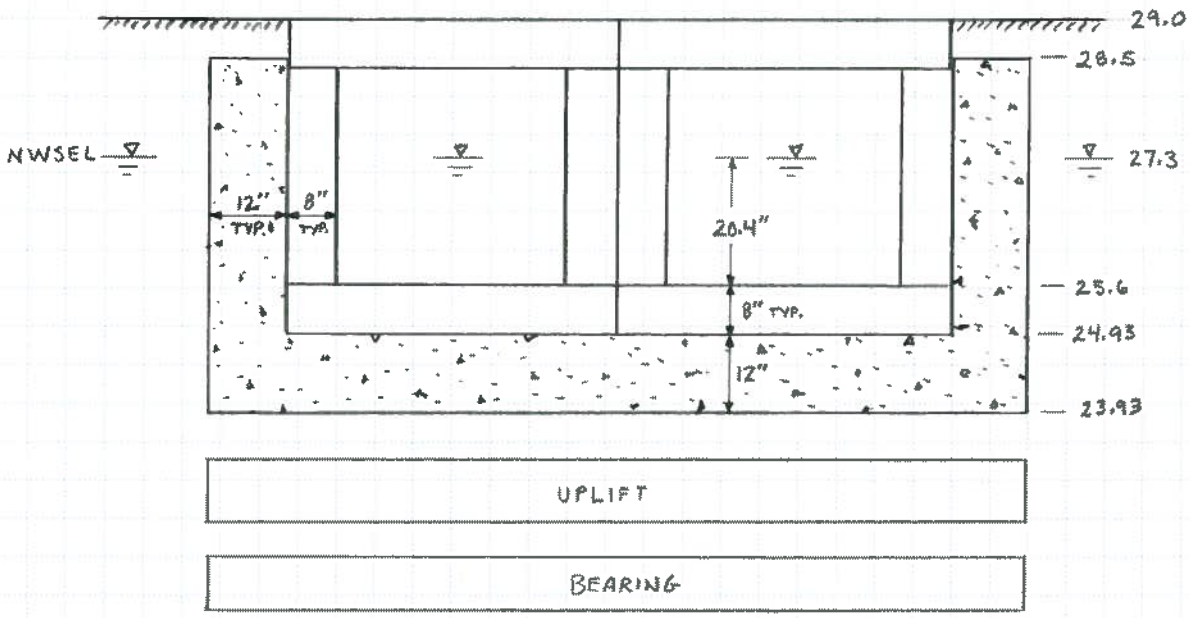
③ UPSTREAM HANDRAIL SUPPORTS

SAME HEIGHT AND SMALLER SPAN WHEN COMPARED TO DOWNSTREAM HANDRAIL

∴ USE SAME REINFORCING DESIGN FOR ALL HANDRAILS

④ SPILLWAY CRADLE



CASE 1 - NORMAL POOL

- CALCULATE VERTICAL FORCES / WEIGHTS:

GRANITE SLABS - $[(2.73' \times 4) + (4' \times 4)]\left(\frac{8''}{12}\right)(175\ LB/CF) = 3.14\ K/FT$

WATER - $(2 \times 2.67' \times 1.7')(62.4\ LB/CF) = 0.57\ K/FT$

CONCRETE - $[(2 \times 3.57') + 10.67'](1')(150\ LB/CF) = 2.67\ K/FT$

$6.4\ K/FT \downarrow$

UPLIFT = $-(3.37' \times 10.67')(62.4\ LB/CF) = -2.2\ K/FT \uparrow$

NET VERTICAL = $6.4\ K/FT - 2.2\ K/FT = 4.2\ K/FT \downarrow$

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- CHECK BEARING PRESSURE $= \frac{V}{A} = \frac{4.2^{K}/FT}{10.67'} = 0.4$ KSF OK

- CHECK FLOTATION -

$$\frac{\Sigma \downarrow}{\Sigma \uparrow} = \frac{6.4^{K}/FT}{2.2^{K}/FT} = 2.9 \quad \text{OK}$$

MCH

- FIND NET ~~FORCE~~ PRESSURE ON BASE SLAB -

WATER + GRANITE $\approx (3.14^{K} + 0.57^{K})/8' = 0.46\ ^{K}/FT^{2} \downarrow$

CONCRETE $= 0.15\ ^{K}/FT^{2} \downarrow$

BEARING + UPLIFT $= \left(\frac{2.2^{K}}{10.67'}\right) + 0.4\ ^{K}/FT^{2} = 0.6\ ^{K}/FT^{2} \uparrow$

NET $= 0.46\ ^{K}/FT^{2} + 0.15\ ^{K}/FT^{2} - 0.6\ ^{K}/FT^{2} \approx 0$

- CALCULATE UNBALANCED SOIL LOADS ON SIDE WALLS (ASSUMING GRANITE SLABS DON'T ACT AS BRACE)

LATERAL WATER FORCES CANCEL

WORST CASE DRY SOIL $\qquad K_o = 0.47$ FOR $\phi = 32°$

$$P = \tfrac{1}{2}\gamma K_o H^2 = \tfrac{1}{2}(.13^{K}/CF)(0.47)(29'-24.93')^2$$
$$= 0.51^{K}$$

$$M = 0.51^{K} \times \frac{(29'-24.93')}{3} = 0.7\ FT\text{-}K$$

$$M_u = 1.7 \times 1.3 \times 0.7\ FT\text{-}K = 1.5\ FT\text{-}K$$

$$V_u = 1.7 \times 1.3 \times 0.51^{K} = 1.1^{K}$$

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CASE 2 - 100 YR. FLOOD - EL. 32.0'

- CALCULATE VERTICAL FORCES

GRANITE SLABS = 3.14 K/FT

WATER = $\left(\left[(32'-29') \times 10.67'\right] + \left[2 \times 2.67' \times 2.73'\right]\right)(62.4\ LB/CF) = 2.9\ K/FT$

CONCRETE = 2.67 K/FT

TOTAL = 8.7 K/FT ↓

UPLIFT = $-(8.07' \times 10.67')(62.4\ LB/CF) = -5.4\ K/FT$ ↑

NET VERTICAL = 3.3 K ↓

- BEARING PRESSURE = $\frac{V}{A} = \frac{3.3\ K/FT}{10.67'} = 0.3$ KSF OK

- FLOTATION = $\frac{8.7^K}{5.4^K} = 1.6$ OK

- NET ~~FORCE~~ PRESSURE MCH ON BASE SLAB

WATER + GRANITE $\approx \left(\frac{3.14^K}{8'}\right) + \left[(32-29)(.0624)\right] + \left[\frac{(2)(2.67')(2.73')(.0624)}{8'}\right]$

$\approx 0.69\ K/FT^2$ ↓

CONCRETE = $0.15\ K/FT^2$ ↓

BEARING + UPLIFT = $\left(\frac{5.4^K}{10.67'}\right) + 0.3\ K/FT = 0.8\ K/FT^2$ ↑

NET = $0.69\ K/FT^2 + 0.15\ K/FT^2 - 0.8\ K/FT^2 = 0.04\ K/FT^2$ ↓

≈ 0

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SIZE REINFORCING –

SIDE WALLS –

$M_u = 1.5$ FT-K

MIN. STEEL: $A_{s\,REQ.} = 1.33 \times M_u \Big/ \left(\phi f_y \left(d - \frac{\beta_1 c}{2}\right)\right)$

$= 1.33\,(1.5\text{ FT-K})(12) \Big/ (0.9)(60\text{ ksi})\left(10'' - \frac{.85(.5)}{2}\right)$

$= 0.05\text{ in}^2 <$ TEMP. & SHRINKAGE

TEMP. & SHRINKAGE:

$A_s > (12'')(12'')(0.0028)/2$ FACES

$> 0.2\text{ in}^2$ PER FACE

FOR No. 4's @ 12" – $A_s = 0.2\text{ in}^2$

$C = A_s f_y / 0.85 f'_c \beta_1 b = 0.20\text{ in}^2 (60\text{ k/in}^2) \Big/ 0.85\,(4\text{ k/in}^2)(0.85)(12'')$

$= 0.35''$

$\phi M_n = 0.9 A_s f_y \left(d - \frac{\beta_1 c}{2}\right) = 0.9(0.20\text{ in}^2)(60\text{ k/in}^2)\left(9.75'' - \frac{0.85(.35'')}{2}\right)$

$= 104\text{ IN-K} \cdot \frac{1\text{ FT}}{12\text{ IN}}$

$= 8.6\text{ FT-K} > 1.5\text{ FT-K} = M_u$

OK

CHECK SHEAR FRICTION ACROSS CONST. JOINTS –

$V_u = 1.1^K$

$\phi V_n = \phi A_{vf} f_y \mu = 0.75\,(0.20\text{ in}^2)(60\text{ ksi})(0.6)$

$= 5.4^K > 1.1^K$ OK

DEVELOPMENT LENGTH –

12.2.3 – (SEE PAGE 3) $\ell_d < 12''$ ∴ USE MIN. REQ'D ~~12"~~ 1'-7"

12.5 – HOOKED (SEE PAGE 3) $\ell_{dh} =$ ~~7"~~ 9" MCH

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ESSENTIALLY NO LOAD ON BASE SLAB –

∴ USE MIN. TEMPERATURE & SHRINKAGE STEEL

PER USACE 2-8.

$$\rho > 0.0020 < \frac{A_s}{bh} = \frac{A_s}{(12'')(12'')}$$

$$A_s > 0.40\ in^2 / 2 \text{ FACES}$$

$$A_s > 0.20\ in^2 \text{ EACH FACE}$$

USE No. 4 BARS AT 12" o.c. ($A_s = 0.2\ in^2$)

REVISIT WALLS : $\phi M_n >> M_u$

PER ACI 10.5.3, MIN. FLEXURAL STEEL REQUIREMENT DOES NOT NEED TO BE MET IF A_s provided > 1.33 A_s required

$$A_s \text{ required} = 0.04\ in^2$$

$$1.33\ A_{s\ req'd.} = 1.33 \times 0.04\ in^2 = 0.05\ in^2$$

$$\text{No. 4 BARS @ 12"} \Rightarrow A_s = 0.2\ in^2$$

∴ No. 4 BARS @ 12" o.c. OK

SHEAR FRICTION OK BY INSPECTION

l_d = ~~12"~~ 17" MCH

l_{dh} = ~~7"~~ 9"

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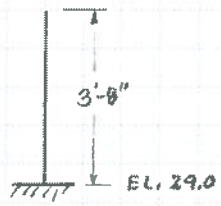
(5) PARAPET WALL EXTENDING OFF RETAINING WALL –



POSSIBLE LOADS:

(1) FLOOD (WATER TO EL. 32.0)

$$P = \tfrac{1}{2}\gamma H^2 = \tfrac{1}{2}(0.0624\ \text{k/cf})(3')^2$$

$$= 0.28\ \text{k/ft} \quad @ \quad H/3 = 1'$$

$$M = 0.28^k \times 1' = 0.28\ \text{ft-k}$$

$$M_u = 1.7 \times 1.3 \times \cancel{0.75}\ 1.0 \times 0.28\ \text{ft-k} = \cancel{0.446}\ .62\ \text{ft-k} \leftarrow$$ MCH

(2) TREAT AS HANDRAIL –

$$P = 0.05\ \text{k/ft} \quad @ \quad 3'\text{-}8''$$

$$M = 0.05^k \times 3'\text{-}8'' = 0.18\ \text{ft-k}$$

$$M_u = 1.6 \times 0.18\ \text{ft-k} = 0.29\ \text{ft-k}$$

BY INSPECTION, MIN. TEMP & SHRINKAGE STEEL

$$\frac{0.0028 \times (12'' \times 12'')}{2\ \text{FACES}} = 0.20\ \text{in}^2 \text{ EACH FACE}$$

USE No. 4 BARS @ 12" O.C. (0.2 in²) EACH FACE EACH WAY

SHEAR FRICTION OK BY INSPECTION

l_d = ~~12"~~ 1'-7" MCH

l_{dh} = ~~7"~~ 9"

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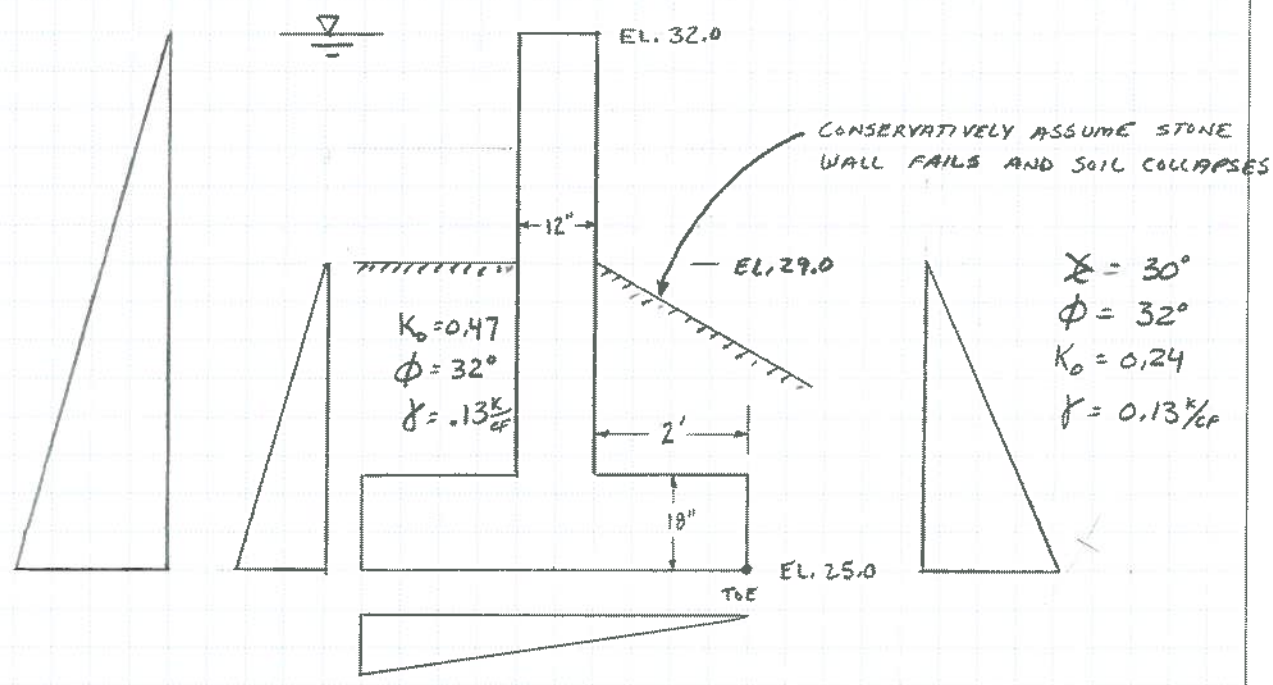
⑥ Cantilever T-Parapet Wall



Forces:

Concrete – Stem = $1' \times 5.5' \times 0.15^k/cf = 0.83^k \times 2.5' = 2.1$ ft-k
Base = $5' \times 1.5' \times 0.15^k/cf = 1.125^k \times 2.5' = 2.8$ ft-k

Soil – u/s = $2' \times 2.5' \times 0.13^k/cf = 0.65^k \times 4' = 2.6$ ft-k
d/s = $2' \times 2' \times 0.13^k/cf = 0.52^k \times 1.25' = 0.6$ ft-k

Water – u/s = $3' \times 2' \times 0.0624^k/cf = 0.37^k \times 4' = 1.5$ ft-k

$3.5^k \downarrow$ $\quad$ 9.6 ft-k ↶

Uplift = $7' \times 5' \times \frac{1}{2} \times 0.0624^k/cf = -1.1^k \uparrow \times 3.33' = -3.7$ ft-k ↷

u/s water = $\frac{1}{2}(7')^2(0.0624^k/cf) = -1.53^k \times 2.33' = -3.6$ ft-k ↷

u/s soil = $\frac{1}{2}(4')^2(0.13^k/cf - 0.0624^k/cf)(.47) = -0.25^k \times 1.33' = -0.3$ ft-k ↷

d/s soil = $\frac{1}{2}(4')^2(0.13^k/cf)(0.24) = 0.25^k \times 1.33' = 0.3$ ft-k ↶

$-1.53^k \rightarrow$

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NET FORCES : VERTICAL = $3.5^k - 1.1^k = 2.4^k \downarrow$

HORIZONTAL = $1.5^k \rightarrow$

$$F.S._{SLIDING} = \frac{2.4^k (\tan\phi)}{1.5^k} < 2.0 \text{ REQUIRED}$$

∴ NEED PASSIVE SOIL

$k_a = 0.31 \Rightarrow 0.16^k \rightarrow$

$k_p = 0.99 \Rightarrow 1.03^k \leftarrow$

$> 0.87^k \leftarrow$

$$F.S._{SLIDING} = \frac{2.4^k (\tan\phi)}{1.5^k - 0.87^k} \geq 2.0$$

$\phi \geq 27.7^\circ$ OK CONSIDERING CONSERVATIVE ASSUMPTION OF 30° D/S SOIL SLOPE

$$F.S._{FLOAT} = \frac{3.5^k}{1.1^k} = 3.2 \quad \text{OK}$$

$$\text{RESULTANT @ } \frac{M}{V} = \frac{9.6 - 3.7 - 3.6 - 0.3 + 0.3}{2.4} = 0.96'$$

$$e = \frac{B}{2} - \frac{M}{V} = \frac{5'}{2} - 0.96' = 1.54'$$

$$\text{BEARING PRESSURE} = \frac{V}{A} \pm \frac{Ve}{S}$$

$$= \frac{2.4^k}{(1')(5')} \pm \frac{2.4^k (1.54')(6)}{(1')(5')^2}$$

$$= 0.48\ ^k/_{SF} \pm 0.89\ ^k/_{SF}$$

$= 1.4\ ^k/_{SF}$ @ TOE

$-0.4\ ^k/_{SF}$ @ HEEL

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RECONSIDER BEARING w/ ACTIVE / PASSIVE FORCES –

U/S SOIL = $0.16^k \times 1.33' = -0.21$ K-FT ↷

D/S SOIL = $1.03^k \times 1.33' = 1.37$ K-FT ↶

$\Sigma M = 9.6 - 3.7 - 3.6 - .21 + 1.37 = 3.5$ FT-K ↶

$\frac{M}{V} = \frac{3.5 \text{ FT-K}}{2.4 \text{ K}} = 1.4'$

$e = \frac{5'}{2} - 1.4' = 1.1'$

BEARING PRESSURE $= \frac{2.4^k}{(1')(5')} \pm \frac{2.4^k (1.1')(6)}{(1')(5')^2}$

$= 0.48 \text{ K/SF} \pm 0.63 \text{ K/SF}$

$= 1.1 \text{ K/SF}$ @ TOE

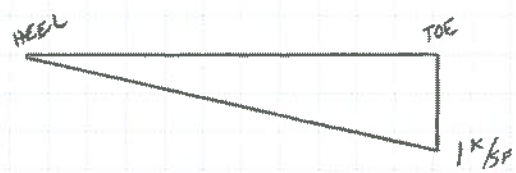
-0.2 K/SF @ HEEL *

* AGAIN, PROBABLY OK CONSIDERING CONSERVATIVE ASSUMPTION OF 30° SLOPE OF D/S SOIL

FOR REINFORCING DESIGN OF BASE ASSUME

1 K/SF BEARING PRESSURE AT TOE

0 K/SF " " " HEEL



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DESIGN STEM REINFORCING –

SOIL LOADS CANCEL

WATER = $\frac{1}{2}(5.5')^2(0.0624\ ^k/cf) = 0.94^k$

$M = 0.94^k \times \frac{5.5'}{3} = 1.73$ FT-K

$M_u = 1.7 \times 1.3 \times \overset{1.0}{\cancel{0.75}} \times 1.73$ FT-K $= \overset{3.8}{\cancel{2.9}}$ FT-K MCH

A_s REQUIRED FOR FLEXURE =

$M_u / \left[\phi f_y \cdot \left(d - \frac{\beta_1 c}{2}\right)\right] \approx \overset{.10}{\cancel{0.08}}\ in^2$

$1.33 \times A_s$ REQUIRED $= 1.33(\overset{.10}{\cancel{0.08}}\ in^2) = \overset{.13}{\cancel{0.1}}\ in^2$

4 BARS @ 12" o.c. $\Rightarrow A_s = 0.2\ in^2 > 1.33\ A_s$ req.

∴ MIN. STEEL PER 10.5.1 DOES NOT APPLY

4 BARS @ 12" o.c. (EACH FACE) MEETS MIN. TEMP. & SHRINKAGE STEEL PER REF. 5

DESIGN BASE REINFORCING –

HEEL –

SOIL = $0.65^k \times 1' = 0.65$ FT-K

WATER = $0.37^k \times 1' = 0.37$ FT-K

CONC. = $0.15 \times 1.5' \times 2' \times 1' = 0.45$ FT-K

UPLIFT = $0.26\ ^k/_{SF} \times 2' \times 1' = -0.52$ FT-K

$(0.44\ ^k/_{SF} - 0.26\ ^k/_{SF}) \times 2' \times \frac{1}{2} \times \frac{2}{3}(2') = -0.24$ FT-K

BEARING = $(0.4\ ^k/_{SF}) \times 2' \times \frac{1}{2} \times \overset{1/3}{\cancel{2/3}} = \overset{-.133}{\cancel{-0.27}}$ FT-K MCH

$M_{NET} = \overset{.57}{\cancel{0.44}}$ FT-K ↶ (TOP FACE IN TENSION)

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TOE — SOIL = $0.52^k \times 1' = 0.52$ FT-K

CONC. = $0.15 \times 1.5' \times 2' \times 1' = 0.45$ FT-K

UPLIFT = $0.17^k/SF \times 2' \times 1/2 \times \cancel{2/3}\ 1/3 = \cancel{-0.12}\ -.06$ FT-K MCH

BEARING = $0.6^k/SF \times 2' \times 1' = -1.2$ FT-K

$(1-0.6^k/SF) \times 2' \times 1/2 \times 2/3(2') = -0.53$ FT-K

$M_{NET} = \cancel{0.88}\ 0.82$ FT-K ↶ (BOTTOM FACE IN TENSION)

VERY SMALL LOADS IN HEEL & TOE

∴ MIN. TEMP & SHRINKAGE STEEL GOVERNS BY INSPECT.

$$\rho = \frac{A_s}{A_g} > 0.0028$$

$$\frac{A_s}{(12'')(18'')} > 0.0028 \Rightarrow A_s > 0.60\ in^2/2\ FACES$$

$$A_s > 0.30\ in^2\ PER\ FACE$$

~~4~~ 5 BARS @ 12" O.C. OK MCH

CHECK SHEAR FRICTION @ CONST. JOINT —

$A_{vf} f_y \mu = \cancel{0.72}\ 0.3\ in^2 \times 60\ ksi \times 0.6 = \cancel{7.2}\ 10.8^k$

$V_u = 1.7 \times 1.3 \times \cancel{0.75}\ 1.0 \times 0.94^k = \cancel{1.6}\ 2.1^k < 7.2^k$ OK MCH

$l_d = \cancel{12''}\ 2'-0''$ MCH

$l_{dh} = \cancel{7''}\ 12''$

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Consider Shear Friction at Const. Joint Between Key and Base

- Conservatively assume that the Shear force on the key is equal to the sum of the driving earth pressure loads, driving water pressure loads, and driving eq. Loads
- Choose Largest loading from all load cases on all dam sections.

Controlling Load Case is either:

	Soil	Water	Eq	
Right side of Dam 100 yr flood	3.0K	11.3K	0	= 14.3K/ft
Right Side of Dam Eq.	5.4K	7.3K	2.3K	= 15.0K/ft

$\phi V_n = .75(A_{VF})(60ksi)(.6) > (1.7)(1.3)(14.3K/ft)$ flood

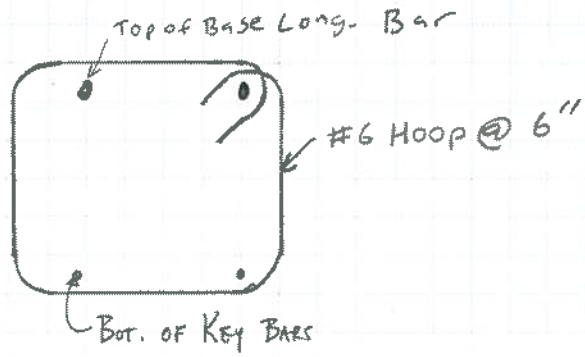
$A_{VF} > 1.17\ in^2/ft$

Extreme Load factor

$.75(A_{VF})(60ksi)(.6) > (1.7)(1.3)(.75)(15.0K/ft)$ earthquake

$A_{VF} \geq .921\ in^2/ft$

- Use #6 hoops @ 6" ($.44in^2 \times 2 \times 2 = 1.76\ in^2/ft$)
- Hoops should enclose min. 2 top of Base Longitudinal bars and both bottom of Key longitudinal bars



For SI: 1 square inch = 645.16 mm^2, 1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated load:

   (1) for passenger cars accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches;

   (2) for mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The weight of books and shelving shall be computed using an assumed density of 65 pounds per cubic foot and converted to a uniformly distributed load; this load shall be used if it exceeds 150 pounds per square foot. The 150 psf load requirement does not apply to libraries that are incidental to other uses.

c. Design in accordance with the ICC *Standard on Bleachers, Folding and Telescopic Seating and Grandstands*.

d. Other uniform loads in accordance with an approved method which contains provisions for truck loadings shall also be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 20 square inches.

f. Minimum concentrated load on stair treads (on area of four square inches) is 300 pounds.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official. See 780 CMR 1608.0. For special-purpose roofs, see 780 CMR 1607.11.2.2.

1607.4 Concentrated Loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in 780 CMR 1607.2 or the concentrated load, in pounds (kilonewtons), given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed on a floor or roof over an area of 2.5 feet square [6.25 ft^2 (0.58 m^2)] and shall be located so as to produce the maximum load effects in the structural members. The aforementioned distribution of concentrated load does not apply for concentrated load applied to framing members, scuttles, skylight ribs, hung ceiling supports, cornices, and similar elements for which there is no deck to distribute load.

1607.5 Partition Loads. In office buildings and in other buildings where partition locations are subject to change, provision for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 pounds per square foot (3.83 kN/m^2). Such partition load shall not be less than a uniformly distributed live load of 20 pounds per square foot (0.96kN/m^2). Partition loads are non-reducible live load.

1607.6 Truck and Bus Garages. Minimum live loads for garages having trucks or buses shall be as specified in AASHTO Standard for H20-44 and HS20-44 Truck loadings, and for lane loads, but shall not be less than 100 pounds per square foot (4.80 kN/m^2), unless other loads are specifically justified. Actual loads shall be used where they are greater than the above specified loads.

1607.6.1 Truck and Bus Garage Live Load Application. Lane loads shall be applied in aisles and ramps, as specified in AASHTO Standard. Truck loads shall be applied in aisles, ramps, in parking spaces, and berths, in positions that will maximize load effects.

1607.6.2 Horizontal Forces. Longitudinal forces and curb forces shall be as specified in AASHTO Standard. Vehicle barriers shall be designed for a concentrated 10,000 lb lateral force acting at a minimum height of two feet-zero inches above a floor or ramp.

1607.7 Loads on Handrails, Guards, Grab Bars and Vehicle Barriers. Handrails, guards, grab bars, and vehicle barriers shall be designed and constructed to the structural loading conditions set forth in 780 CMR 1607.7.

1607.7.1 Handrails and Guards. Handrail assemblies and guards shall be designed to resist a load of 100 pounds per lineal foot for grandstands, stadia, arenas, and similar structures used for public assembly, and 50 pounds per linear foot (pound per foot) (0.73 kN/m) for other uses, applied in any direction at the top and to transfer this load through the supports to the structure.

Exception. For one- and two-family dwellings, only the single, concentrated load required by 780 CMR 1607.7.1.1 shall be applied.

1607.7.1.1 Concentrated Load. Handrail assemblies and guards shall be able to resist a single concentrated load of 300 pounds for grandstands, stadia, arenas, and similar structures used for public assembly, and 200 pounds (0.89 kN) for other uses, applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in 780 CMR 1607.7.1.

1607.7.1.2 Components. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 200 pounds (0.88 kN) on an area equal to one square foot (305 mm^2) including openings and space between rails. Reactions due to this loading are not required to be superimposed with those

3-3. Required Strength

a. General. Reinforced concrete hydraulic structures and hydraulic structural members shall be designed to have a required strength, U_h, to resist dead and live loads in accordance with the following provisions. The hydraulic factor is to be applied in the determination of the required nominal strength for all combinations of axial load, moments and shear (diagonal tension). In particular, the shear reinforcement should be designed for the excess shear, the difference between the hydraulic factored ultimate shear force, V_{uh}, and the shear strength provided by the concrete, ϕV_c, where ϕ is the concrete resistance factor for shear design. Therefore, the design shear for the reinforcement, V_s, is given by

$$V_s \geq \left(\frac{V_{uh} - 1.3\phi V_c}{\phi} \right) \tag{3.1}$$

b. Single Load Factor Method. In the single load factor method, both the dead and live loads are multiplied by the same load factor.

$$U = 1.7(D + L) \tag{3.2}$$

where

U = factored loads for a nonhydraulic structure
D = internal forces and moments from dead loads
L = internal forces and moments from live loads

$$\rightarrow \quad U_h = H_f[1.7(D + L)] \tag{3.3}$$

where

U_h = factored loads for a hydraulic structure
H_f = hydraulic factor.

For hydraulic structures the basic load factor, 1.7, is multiplied by a hydraulic factor, H_f, where $H_f = 1.3$, except for members in direct tension. For members in direct tension, $H_f = 1.65$. Other values may be used subject to consultation with and approval from CECW-ED.

An exception to the above occurs when resistance to the effects of unusual or extreme loads such as wind, earthquake or other forces of short duration and low probability of occurrence are included in the design. For those cases, one of the following loading combinations should be used:

$$U = 0.75U_{W\,or\,E} \tag{3.4}$$

for nonhydraulic structures and

$$U = H_f\left(0.75U_{W\,or\,E}\right) \tag{3.5}$$

for hydraulic structures where

$U_{W\,or\,E}$ = nonhydraulic factored loads including wind or earthquake effects.

c. Modified ACI 318. The load factors prescribed in ACI 318 may be applied directly to hydraulic structures with two modifications. The load factor for lateral fluid pressure, *F*, should be taken as 1.7 instead of the ACI 318 prescribed value of 1.4. Also, for hydraulic structures, the factored load combination for total factored design load, *U*, as prescribed in ACI 318 shall be increased by the hydraulic factor $H_f = 1.3$, except for members in direct tension. For members in direct tension, $H_f = 1.65$.

$$U = 1.4D + 1.7L \tag{3.6}$$

for nonhydraulic structures and

$$U_h = H_f U = H_f\left(1.4D + 1.7L\right) \tag{3.7}$$

for hydraulic structures.

For certain hydraulic structures such as U-frame locks and channels, the live load can have a relieving effect on the factored load combination used to determine the total factored load effects. In this case, the combination of factored dead and live loads with a live load factor of unity

$$U_h = H_f\left(1.4D + 1.0L\right) \tag{3.8}$$

should be investigated and reported in the design documents.

d. Earthquake effects. If resistance to earthquake loads, *E*, are required, the following definitions and load combinations shall apply.

1) Unusual and extreme loads. Earthquake loads are considered either unusual or extreme due to their low probability of occurrence and short duration. Yet, their low probability of occurrence also allows them to be combined with normal operating loads, such as normal operating pool levels for hydraulic structures, when developing load combinations.

Appendix H

Low-Level Outlet Waiver Request Letter

January 5, 2010
Project 07104-1

Mr. William Salomaa
MA DCR – Office of Dam Safety
251 Causeway Street, 8th Floor
Boston, MA 02114

Dear Mr. Salomaa:

Re: Low-Level Outlet Waiver Request
Mill Pond Dam Safety Modifications
Rockport, Massachusetts

The purpose of this letter is to request a waiver from the requirement in the Massachusetts Dam Safety Regulations (302 CMR 10.00) which states that "a conduit shall be provided to drain each reservoir" (302 CMR 10.14 (7)) for Mill Pond Dam in Rockport, Massachusetts. It is our opinion that the addition of a low-level outlet in the remedial design would add significant cost and complexity to the design, permitting, and construction of the dam safety modifications, and could also adversely impact the allocation of funding available for the repairs.

Existing Dam Description

Mill Pond Dam is owned and operated by the Town of Rockport, Massachusetts for historic, aesthetic, and recreational purposes. The dam is about 100 feet long and about 14 feet high at the maximum section. According to the Federal Emergency Management Agency (FEMA) Preliminary Engineering Report (2006), the dam was constructed with a clay core and hand-laid un-mortared stone on the upstream and downstream face. The crest of the dam consists of a stone aggregate surface and mortared fieldstone parapet walls that are about 95 feet long and extend along most of the upstream and downstream faces of the dam. The primary spillway consists of two 2.5-foot square granite block culverts near the right abutment. In its current condition, Mill Pond Dam has no functioning low-level outlet, and may have never had a functioning low-level outlet.

Under the normal pool conditions, the water surface elevation in Mill Pond is approximately 27.3 feet (NGVD 1929). At normal pool, Mill Pond has an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Mill Pond Dam is currently classified as a small, significant hazard dam, and as such, the design storm is the 100 year flood. Under the design storm conditions, the surface area and volume are estimated to be approximately 3.4 acres and 10 acre-feet, respectively. We have estimated that the Mill Pond Dam watershed is about 0.75 square miles, and that the average inflow from the watershed is about 1.2 cubic feet per second (cfs).

Proposed Modifications

GEI is currently preparing remedial designs of Mill Pond Dam for the Town of Rockport. We will submit a Chapter 253 permit application when our design documents are complete. Our proposed dam safety modifications will include a new reinforced concrete gravity retaining wall which will be designed to resist soil and water pressures and will meet the recommended safety factors provided in the Dam Safety Regulations. The existing granite block masonry wall will be re-installed on the downstream face of the reinforced concrete wall, and compacted soil backfill will be placed in the excavation behind the wall. The granite block culverts will be re-installed at the same invert elevation and at their current location near the right abutment. The dam will be designed to overtop during the design storm and will have adequate scour protection at the toe, likely consisting of a concrete splash apron and rip-rap. A parapet wall will be constructed along the left abutment to prevent overtopping in this area, and the ground surface near the right abutment will be stabilized to minimize erosion during flood flows.

Additional Complexity and Cost

Because no functioning low-level outlet currently exists, adding a low-level outlet would require significant excavation and related wetland impacts (and replication) for the installation of an intake at the low point of Mill Pond. Construction of a low-level outlet would also require the installation of a gate structure below the crest of the dam, a penetration through the new concrete retaining wall, and the installation of an outlet on the downstream face of the dam. Installing this outlet would add significant design, construction, and permitting costs not currently planned for the project. The addition of a new outlet on the downstream face of the dam may also negatively affect the historic appearance of the dam and impact funding.

Mechanical Drawdown Plan

Because drawdown would not be required to reduce the loading on the dam and because of the small size of the Mill Pond impoundment (3 acre-feet), it is our opinion that the pond could easily be drawn down with pumps owned or rented by the Rockport Department of Public Works and that a low-level outlet would not be required for drawdown. Once construction is complete, drawdown will only be required for routine maintenance. Therefore, it is our opinion that the drawdown system can be designed for normal pool and normal flows. Given the relatively small size of the watershed, pond, and the normal inflow, our calculations indicate that the pond can be lowered using a mobile pump. The normal inflow is approximately 540 gallons per minute (gpm) (1.2 cfs) and the total volume of the pond at normal pool is approximately 3 acre-feet (1 million gallons). Therefore, assuming normal inflows, a pump operating at 800 gpm (e.g. Rain for Rent DV-100) could drain the pond in about 72 hours (3 days). A detailed Drawdown Plan will be included in the Operation and Maintenance Manual. A larger pump could lower the pond faster. However, a large increase in outflow may adversely impact the stream and other resource areas immediately downstream of the dam.

Request for Waiver from 302 CMR 10.14 (7)

It is our opinion that a mechanical drawdown allows Mill Pond to be lowered in a safe and cost effective manner. Therefore, we request a waiver from the requirement in 302 CMR 10.14 (7) for a low-level outlet at Mill Pond Dam in Rockport, Massachusetts, and request that mechanical means be permitted to drawdown Mill Pond for routine maintenance.

Please feel free to contact me at 781-721-4034 (lwooten@geiconsultants.com) if you have any questions.

Sincerely,

GEI CONSULTANTS, INC.

R. Lee Wooten, PE
Vice President

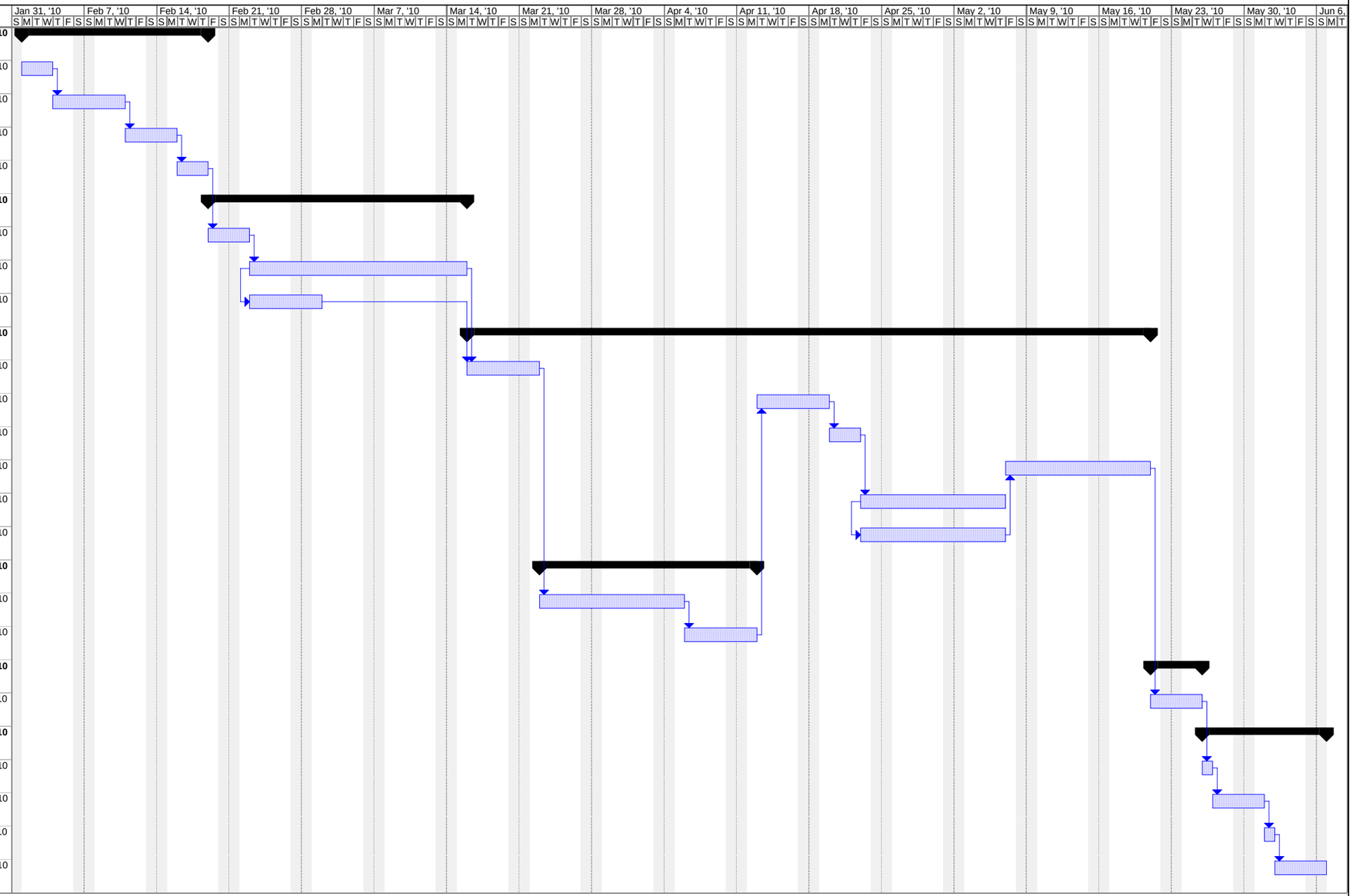
PJS/RLW:rr
M:\PROJECT\2007\07104\Mill Pond Dam\LLO Waiver Request 2010_1_5.docx

Appendix I

Preliminary Construction Schedule

Preliminary Construction Schedule
Mill Pond Dam
Rockport, MA

ID		Task Name	Duration	Start	Finish
1		**Site Preparation**	**14 days**	**Mon 2/1/10**	**Fri 2/19/10**
2		Mobilization	3 days	Mon 2/1/10	Thu 2/4/10
3		Install Erosion Control	5 days	Thu 2/4/10	Thu 2/11/10
4		Remove Trees	3 days	Thu 2/11/10	Tue 2/16/10
5		Dewatering/ByPass Pumps	3 days	Tue 2/16/10	Fri 2/19/10
6		**Demolition**	**17 days**	**Fri 2/19/10**	**Tue 3/16/10**
7		Remove Existing WPA Wall	2 days	Fri 2/19/10	Tue 2/23/10
8		Catalogue and Remove Downstream Face	15 days	Tue 2/23/10	Tue 3/16/10
9		Excavate Embankment	5 days	Tue 2/23/10	Tue 3/2/10
10		**Construct New Retaining Wall**	**48 days**	**Tue 3/16/10**	**Fri 5/21/10**
11		Install Reinforced Concrete Wall	5 days	Tue 3/16/10	Tue 3/23/10
12		Backfill	5 days	Tue 4/13/10	Tue 4/20/10
13		Reinstall Granite Spillway	3 days	Tue 4/20/10	Fri 4/23/10
14		Construct Handrail and Posts	10 days	Fri 5/7/10	Fri 5/21/10
15		Construct Parapet Walls	10 days	Fri 4/23/10	Fri 5/7/10
16		Reinstall Downstream Face	10 days	Fri 4/23/10	Fri 5/7/10
17		**Tiedowns**	**15 days**	**Tue 3/23/10**	**Tue 4/13/10**
18		Install Tiedowns	10 days	Tue 3/23/10	Tue 4/6/10
19		Test Tiedowns	5 days	Tue 4/6/10	Tue 4/13/10
20		**Grading**	**3 days**	**Fri 5/21/10**	**Wed 5/26/10**
21		Grade Channel	3 days	Fri 5/21/10	Wed 5/26/10
22		**Site Restoration**	**8 days**	**Wed 5/26/10**	**Mon 6/7/10**
23		Restore Masonry Wall	1 day	Wed 5/26/10	Thu 5/27/10
24		Regrade Site	3 days	Thu 5/27/10	Tue 6/1/10
25		Seed Disturbed Areas	1 day	Tue 6/1/10	Wed 6/2/10
26		Demobilization	3 days	Wed 6/2/10	Mon 6/7/10



Project: Project Schedule
Date: Tue 10/6/09

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

Volume II – Design Drawings

Volume III – Technical Specifications

Drive Joe: Cell 207-217-7411
7:00

Mill Pond and
Millbrook Meadow Restoration
Rockport, Massachusetts

Date	Time	Gage #1 Reading (ft)	Gage #1 Elevation (ft, NAVD88) (Reading + 7.8)	Gage #2 Reading (ft)	Gage #2 Elevation (ft, NAVD88) (Reading + 10.0)	Gage #3 Reading (ft)	Gage #3 Elevation (ft, NAVD88) (Reading + XX)	Name	Comments
4/28/14	12:15	1.55	9.35	1.94	11.94				
5/1/14	20:00	0.88	8.68	1.56	11.56				
6/4	9:15	1.64		1.84					7-9 am showers
6/6	8:30am	.88		1.58					overnight showers/rain

Gage #1 (Near Culvert) Invert Elevation = 7.8 feet NAVD88

Gage #2 (Near Footbridge) Invert Elevation = 10.0 feet NAVD88

Gage #3 (Mill Pond) Invert Elevation = XX feet NAVD88

MILONE & MACBROOM®

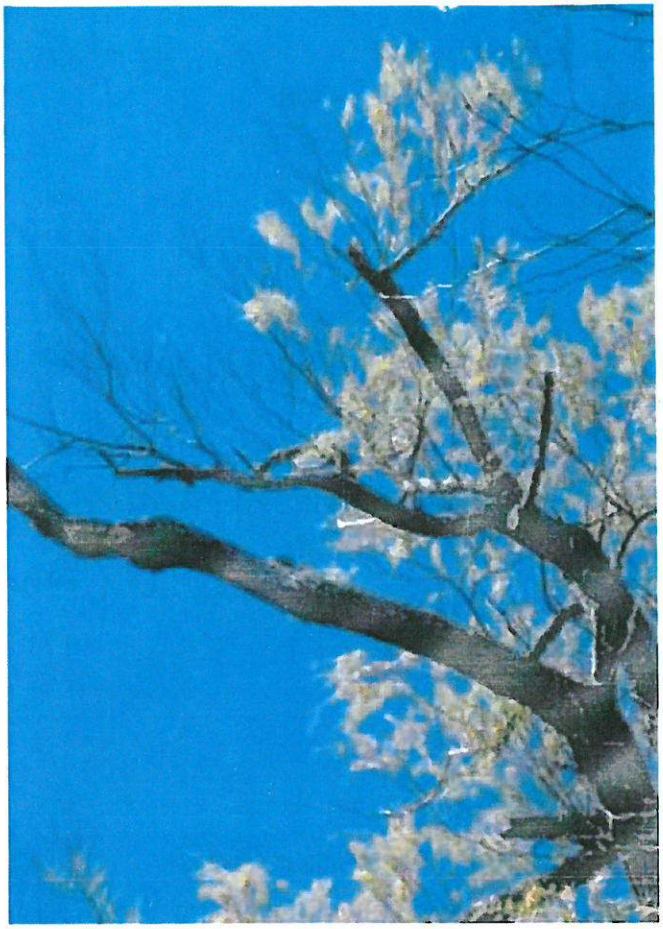
LOWER MILL BROOK NONPOINT SOURCE POLLUTION ASSESSMENT

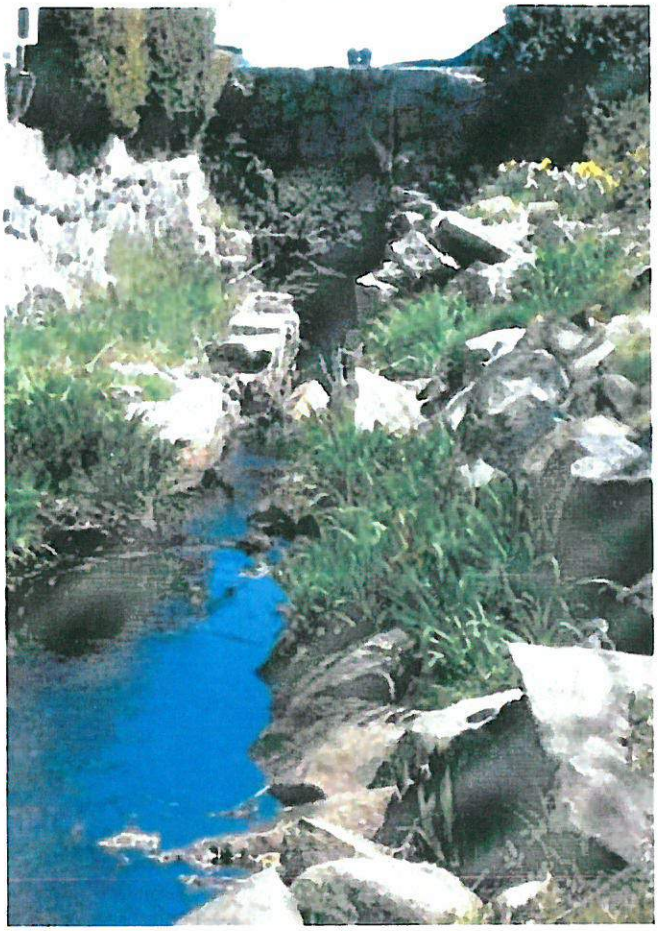
Department of Public Works
Rockport, Massachusetts

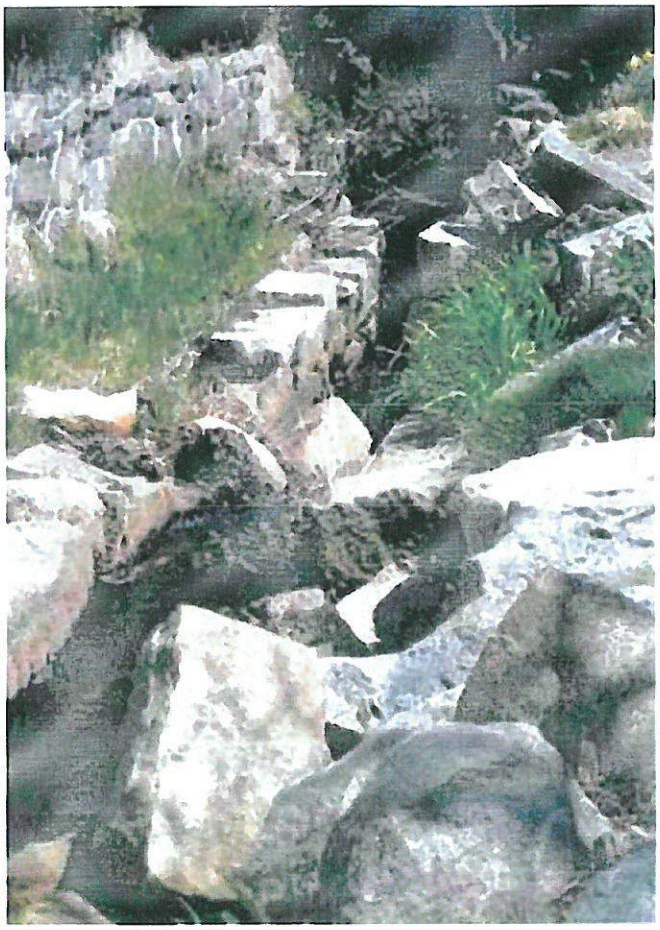
June 2005

Project Funded by:
MA Office of Coastal Zone Management
Coastal Nonpoint Source Pollution Program

Project Partners:
Town of Rockport
MA Office of Coastal Zone Management
USDA Natural Resources Conservation Service
Massachusetts Bays Program Eight Towns and the Bay Committee







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B. Direct Discharge Outfalls: Description, Photos, and Permit History
C. Photos of Selected Erosion and Sedimentation Locations
D. Beach Testing v. Rainfall 1999-2004
E. Water Quality Monitoring Lab Reports
F. Water Quality Monitoring – Results and Comparison
G. Catch Basin Data
H. Catch Basin Cleaning Plan
I. Street Sweeping Plan
J. Erosion and Sedimentation Control Model By-law
K. Post-Construction Stormwater Management Model By-law
L. Structural BMP Recommendation Letter

1. Introduction

1.1 Purpose

The Town of Rockport received funding from the Massachusetts Office of Coastal Zone Management to conduct a Nonpoint Source Pollution Assessment of the lower section of Mill Brook. The purpose of this project was to conduct an assessment of the sediment and bacteria contamination of the lower portion (Poole's Lane north to Front Beach on Sandy Bay) of Mill Brook in Rockport, MA located in the North Coastal Watershed. The goals of the project are to better characterize and evaluate the nature and sources of the contamination, and to look for potential to implement or install best management practices (BMP's) to reduce the contamination reaching the stream and coastal waters. These goals are being met through water quality monitoring at locations throughout the study area, through physical assessment of the drainage areas of the outfalls, through education of town boards and officials, and through undertaking of pilot programs to maximize efficiency of catch basin cleaning and street sweeping practices within the lower Mill Brook watershed. The end products of the proposed project are a physical and wet-weather water quality assessment of the lower portion of Mill Brook and its outfalls; an evaluation of the potential for structural BMP's at specific outfalls including draft designs where applicable, an evaluation of the effectiveness of pilot programs for enhanced catch basin cleaning and street sweeping, and draft by-law language establishing local thresholds for infiltration of runoff from any new impervious surfaces; and for requiring BMP's for sediment control from construction sites within the lower Mill Brook watershed.

Periodic beach closures and a permanently posted warning to swimmers at the mouth of Mill Brook are indications that a pollution problem exists. Increasing visual evidence that the historic Mill Pond on Mill Brook is succumbing to sediment is evidence that a sedimentation problem exists. Rockport is a coastal community with a large portion of its economy being tourist-based, inns, bed and breakfasts, restaurants, shops, and other activities (kayaking tours, sailing tours). One of the focal points of summer tourists in Rockport is its sandy beaches and clear coastal waters. Mill Brook discharges at the western end of Front Beach in the heart of the downtown. Because it is accessible to the many inns and bed and breakfasts in the downtown area, Front Beach is one of the most conspicuous and popular of Rockport's beaches. The other industry, in addition to tourism, that still thrives in Rockport is a fleet of inshore lobster boats and small draggers that depend on the health and bounty of Rockport's coastal waters for their living. Mill Brook also supports an American eel (*Anguilla rostrata*) run up to Loop Pond and rainbow smelt (*Osmerus mordax*) spawning habitat at its mouth. These catadromous and anadromous fish species not only contribute to a healthy marine food web, they are important to recreational and professional rod and reel fishermen and are known to provide an important food source for other commercially important fish species such as striped bass, bluefish and cod.

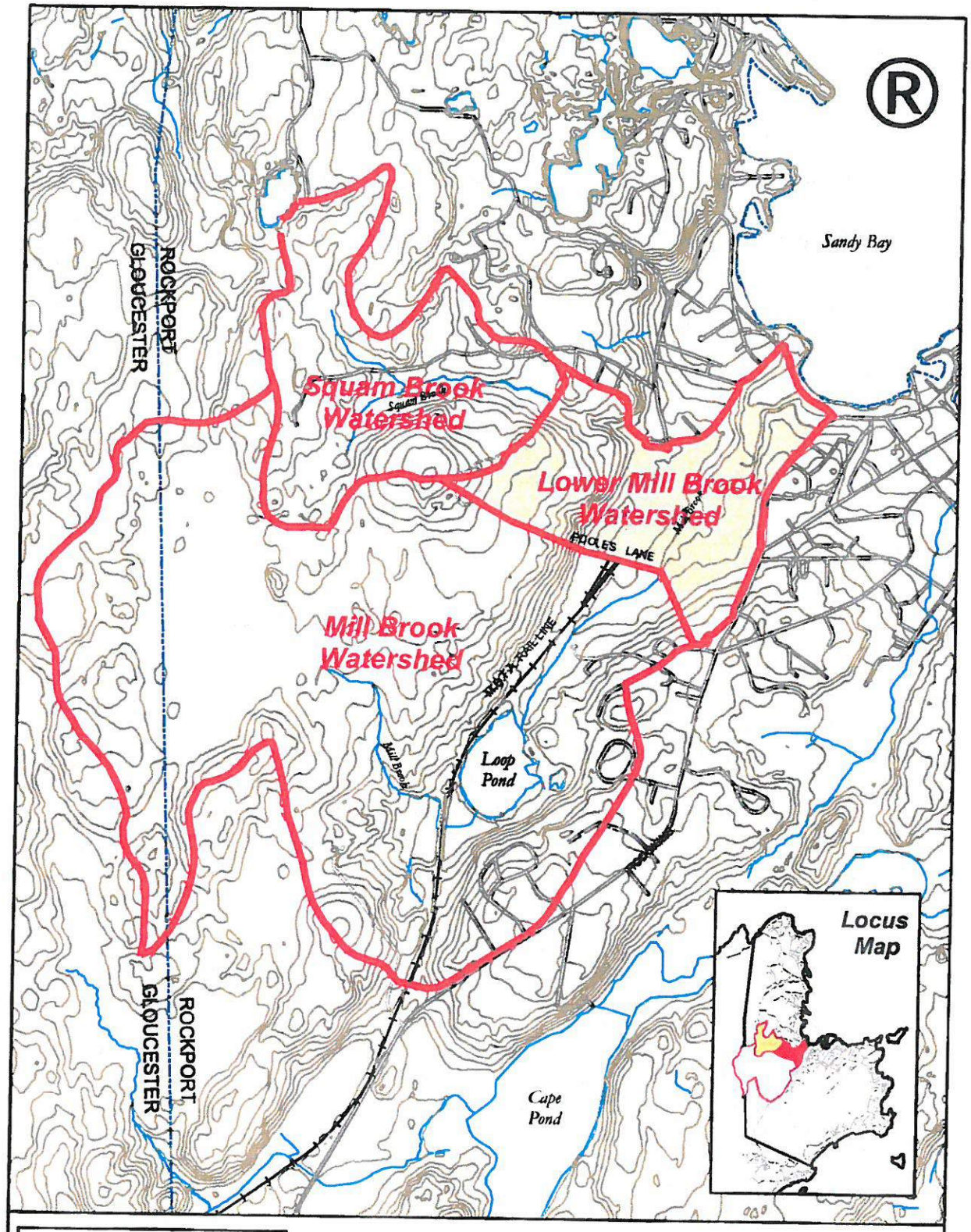
Beach closings and a public perception of contaminated water at Rockport's beaches could have a negative impact to the tourist industry. Sedimentation and excess nutrients, often associated with elevated bacteria levels, can degrade freshwater and coastal resources to the point where they can no longer support healthy populations of American

eels, rainbow smelt and shellfish and other commercially important species that depend on a healthy coastal food web.

1.2 Description of Study Area

Mill Brook is Rockport's largest perennial stream with its headwaters in the undeveloped interior of Cape Ann, actually starting in the neighboring City of Gloucester, and running approximately 1.5 miles to its outlet at Front Beach on Sandy Bay. From its undeveloped origins to the north and west of Loop Pond, Mill Brook runs through a moderately developed watershed between Loop Pond and Poole's Lane at which point it enters an area densely developed with both commercial and residential development. The focus of this project is the lower watershed portion of Mill Brook from Poole's Lane north to its mouth at Front Beach.

The study area is approximately 94 acres and comprises the watershed to the lower section of Mill Brook from Front Beach up to Poole's Lane and including the lower section of Squam Brook up to Squam Hill Court. (See Figure 1, Lower Mill Brook Nonpoint Source Pollution Assessment Study Area.) The Mill Brook watershed (above Poole's Lane) and Squam Brook watershed (above Squam Hill Court) both drain into the Lower Mill Brook study area. The Mill Brook and Squam Brook watersheds are part of the Town of Rockport Watershed Protection Overlay District (WPOD) and have both previously been delineated for watershed protection purposes. These watersheds are included in the Lower Mill Brook nonpoint source pollution assessment through water quality monitoring stations J and L which were included to characterize the inputs from these less-densely developed watersheds.



Legend

- Watershed Boundaries
- Study Area
- Town Boundary
- Rail
- Unpaved Roads
- Paved Roads
- Contour Lines
- Water Bodies
- Streams

0 210 420 840 1,260 1,680 Feet

Figure 1

Lower Mill Brook
Non Point Source Pollution
Assessment Study Area

Cape Ann Mapping

2. NPS Assessment

2.1 GIS Map

One of the tools used on this project to provide analysis and graphical display of the watershed was Geographic Information Systems (GIS). Basemap data collected from MassGIS, and from the Town of Rockport were used with several layers created for this report. Those layers include the Lower Mill Brook watershed study area, water quality monitoring sample collection locations, parcels with septic systems, identified outfalls on Mill Brook, municipal and private catch basins within the watershed, and sub-drainage areas delineated for each water quality monitoring sample collection location. These layers were used to create a comprehensive site map which is included in the insert in the back of this report. The GIS data layers have also been delivered to both CZM and to the Town of Rockport, and can be used for further analysis of the hydraulic features and nonpoint source pollution parameters in this area using GIS. Further analytical techniques could include network tracing, stormwater modeling, sediment loading to the brook, etc. in conjunction with other GIS layers which MassGIS, CZM, or other agencies may have in their data libraries.

2.2 Drainage Area Delineation

The sub-watershed drainage area was delineated for each of 10 of the sampling stations monitored during the study period. (The Mill Brook and Squam Brook watersheds, drainage areas for monitoring stations J and L were previously delineated as explained above.) This was undertaken to help better assess pollution and sediment sources that might be influencing water quality at the sampling stations. In part, because Rockport's historic downtown, which comprises much of the study area, was developed so long ago, few of the drainage systems are documented. This made delineating the drainage areas a challenge, particularly in the vicinity of Railroad Avenue and the commuter railroad station where there is a dense mix of commercial development with fairly expansive impervious surfaces and little documentation of the existing drainage systems.

2.2.1 Methodology

Delineation of the sub-drainage areas was conducted using the following methodology: The boundary of the lower Mill Brook watershed study area and of the sub-drainage areas within the study area were delineated using topographic maps, field observations during precipitation events, and identification of catch basins and outfalls including a limited amount of dye testing of drainage pipes. First, the lower Mill Brook watershed was delineated using topographic maps including USGS topographic maps and Town of Rockport topographic maps (5 foot contour intervals, 1 in.:100 foot scale), using standard watershed delineation techniques The watershed boundary for this section of lower Mill Brook was then further refined based on field reconnaissance. The project partners walked sections of the lower Mill Brook watershed including during precipitation events, observing direction of sheet flow down roads and over impervious surfaces and direction of flow in catch basins. During at least two heavy precipitation events, monitoring dye was introduced to catch basins to determine where along the stream the runoff was being directed.

2.2.2 Description by Area

A brief description of each of the sub-drainage areas within the study area is included in Table 1 including estimated size and number of outfalls and catch basins for each sub-watershed. The drainage areas are depicted on the GIS Map (reference pocket insert) by the lettered, colored polygons associated with each sampling location. The drainage area for sampling station J and L are the Mill Brook and Squam Brook watersheds respectively. A recently completed report, *Wellhead Protection Plan, Mill Brook Watershed, Rockport, MA, 2004*, includes an assessment of pollution threats to the upper Mill Brook watershed. The Mill Brook watershed (sub-drainage area J) is approximately 450 acres of some fairly high density residential development on the edge of a primarily undeveloped forested watershed. The Squam Brook watershed, sub-drainage area L, is approximately 112 acres of unsewered residential and forested watershed.

An estimate of the water quality treatment volume was calculated for 10 of the sub-drainage areas within the study area (Table 1). These volumes were calculated by estimating percentage of several categories of land use for each of the 10 sub-watersheds and multiplying by published estimates (Klein, 1979) of percent imperviousness for each of the land use categories. A runoff depth of 0.5 inches was used to calculate the water quality treatment depth based on the MA DEP Stormwater Guidelines because though Mill Brook empties to a swimming beach, it wasn't clear if it would be classified as a critical area under the guidelines. Water quality treatment volumes for the 10 sub-drainage areas calculated ranged from approximately 18,000 gallons to 92,000 gallons, and was the lowest for sub-drainage area C and the highest for sub-drainage area H (approx. 92,000 gal.) containing a large portion of the commuter railroad station, Whistlestop Mall shopping center, and the Mill Brook Park housing complex and sub-drainage area E (approx. 68,000 gal.) draining the north end of the commuter railroad station, Whistlestop Mall shopping center, Dunkin' Donuts and other commercial establishments adjacent to Railroad Avenue.

2.3 Septic Systems

Septic systems were identified in the Study Area by reviewing records and through discussion with staff of the Rockport Dept. of Public Works and Rockport Board of Health. Where onsite sanitary sewage treatment (septic) systems were identified, notes were made of the most recent testing or pumping of those systems and whether they were found to be failing or not failing to meet Title V requirements. Parcels with septic systems are depicted on the GIS map (reference pocket insert) as parcels shaded in a tan color. Table 1 shows numbers of septic systems by sub-drainage area, and a list of specific parcels with septic systems including any notes on recent inspections, pump-outs or upgrades from the Board of Health records are included in the Appendix A. It was found that within the Study Area, there were 14 parcels on record containing septic systems. Six of the 14 occur off Haddow Road off Summit Avenue. The rest occur off McKays Drive and Squam Hill Court off of Squam Road. Of the 14 identified septic systems, two off of Haddow Road are known to be failing to meet Title V regulations. According to Dept. of Public Works staff, property owners of these two parcels were in the process of having municipal sewer connection plans approved by the town.

Table 1: Description of Sampling Station Locations and Sub-Drainage Areas

	Sampling Station Location	Drainage Area Description	Size in Acres	WQ Trtmnt. Volume (gal.)*	No. of Catch Basins	No. of Outfalls	No of Septic Systems
A	Beach St--downstream	Part of Beach St.; most of King St.; some yards at corner of King and Beach and Maine and Beach	4.83	37,475	13	4	0
B	Beach St--upstream	Millbrook Meadow up to Mill Pond; Mill Lane up to Maine St.; associated yards	5.64	28,178	4	3	0
C	Mill Pond--downstream at dam	Mill Pond with adjacent land from King St. to Maine St.	5.39	18,074	3	0	0
D	Mill Pond—upstream behind Union Cemetary	Top of Mill Pond to Henderson Crt; west to King St.; east to Maine St.	9.45	34,255	5	1	0
E	Henderson Ct—Mill Brook above Squam Brook tributary	Parts of Railroad Ave. west and east of stream up to Maine St.: Henderson Crt.; commercial and railroad parking areas off Railroad Ave; part of Whistlestop Mall	8.32	68,450	6 (+3 Private)	10	0
F	Henderson Ct—Squam Brook	Northwest end of Railroad Ave. up to Summit Ave.; yards off Railroad Ave.	5.57	32,479	3 (1)	0	0
G	Railroad Ave—immediately upstream	Isinglass Place and other commercial buildings and parking; Anchorage town houses; residential southeast to Poole's Lane and Main St.	4.2	38,289	0 (8)	10	0
H	200m upstream of Railroad Ave	Railroad station; Whistlestop Mall; Mill Brook Park Housing; Evans Field; weat to Summit Ave; east to Poole's Ln. ?	13.78	92,511	0 (12)	4	5
I	Poole Ln—immediately downstream	Poole's Ln; part of Evans Field; west up to Hospital Hill; east just shy of Main St.	22.26	45,784	6 (2)	3	1
J	Poole Ln—immediately upstream	Mill Brook Watershed	448.64	Not Calculated	Not Counted	Not Counted	41
K	Squam Brook—immediately down stream of Summit Ave	Summit Ave. down to base of Lowest Ln	14.43	44,960	2 (1)	0	8
L	Squam Brook—end of Squam Hill Ct.	Squam Brook Watershed	112.07	Not Calculated	0	None Identified	35

* Water Quality Treatment Volume is based on a 0.5 inch rain event.

Upstream of the study area on Squam Brook, the Squam Brook watershed is completely unsewered and all residential development relies on onsite subsurface sanitary sewage (septic) systems. There were 35 septic systems identified in this watershed. Upstream of sampling station J is the Mill Brook watershed. The majority of the Mill Brook watershed is undeveloped. The municipal sewer system does extend to parts of the more densely developed sections of this watershed, primarily off of upper Main Street. The exception is the unsewered Hodgkins Road area which is a fairly densely developed residential neighborhood on fairly steep slopes just above Loop Pond on Mill Brook. The *Mill Brook Wellfield Wellhead Protection Plan (2004)* identified 36 parcels with septic systems within this watershed. A revised estimate after a closer inspection of the watershed is 41 parcels with septic systems.

2.4 Outfalls

The *Mill Brook Illicit Discharge Detection and Elimination Summary Report* prepared for the Rockport Dept. of Public Works by Woodard & Curran in 2003 identified seventeen direct outfalls on the section of Mill Brook from Front Beach up to Poole's Lane. Additional outfalls were identified in a walk of the stream with the project team and on subsequent walks of the stream by the study team during the study period. A total of 35 direct pipe discharge outfalls are identified in this report. The outfalls are identified and numbered on the GIS map (reference pocket insert). Table B-1 contains a description of the location and characteristics of each outfall pipe with notes of observations made during reconnaissance. Appendix B contains photos of most direct discharge outfall pipes identified on the GIS Map and in Table B-1.

2.4.1 Description

A number of the identified direct outfalls on the lower section of Mill Brook are of municipal origin. Four drains discharge to the brook under Beach Street, 3 under Railroad Avenue and 3 under Poole's Lane. Many of these are part of the municipal storm drainage system. However, it is likely that at least one of the discharges under Poole's Lane originates from the commuter rail station. At least one other discharge, Outfall #7, located approximately 200 feet upstream from Beach Street connected to a catch basin draining Millbrook Meadow, is part of the municipal storm drainage system.

Several of the other outfalls appear to be discharging stormwater from adjacent commercial and residential properties. The section of town between Poole's Lane down to and including the north side of Railroad Avenue is one of the most densely commercial areas of town. This is also the area of the highest concentration of outfalls. (See GIS map.) This includes a sizeable shopping center including an IGA grocery store, pharmacy, Laundromat, and variety store. The commuter rail station, another smaller shopping center, Isinglass Place, Dunkin' Donuts, a hardware store, an oil company, and an automotive repair garage are all located in this area as well. A public housing apartment complex and a privately-owned townhouse complex also abut the brook in this area.

A number of pipes appear to be discharging from sumps or basements inside adjacent commercial and/or residential buildings. Several of these pipes were observed to have

dry-weather discharge. At least two of these appeared to be connected to automatic pumps because they were observed to discharge at irregular intervals. Other pipes are likely draining residential yards adjacent to the brook. There were no visual indications of direct sewage outfalls observed.

2.4.2 Permit History

A review of the Rockport Conservation Commission records was conducted to determine the permit history of the direct discharge outfalls. A summary of all permits and determinations under the MA Wetlands Protection Act and Rockport Wetlands By-law for property or activities adjacent to Mill Brook within the study area is located in Table B-2. A total of 23 permit actions were identified. Of the 23, only two appeared to have any reference to direct discharge outfalls on the brook. These included the installation of a 6-inch diameter "interceptor" drain to drain groundwater from a residential property just north of Railroad Avenue, and a permit for a substantial building and drainage renovation at Isinglass Place commercial center located just south of Railroad Avenue, which included two 12-inch PVC discharge pipes draining a series of catch basins, located in the Isinglass Place parking lot. These permits correspond with Outfall #13 or 14 and 25 and 28, respectively. In summary, 3 of the 35 identified direct discharge outfalls within the study area have any permit history on record with the Rockport Conservation Commission.

2.5 Monitoring Erosion at Outfall Locations

Monitoring erosion at outfall locations was a specific recommendation of the *Mill Brook Illicit Discharge Detection and Elimination Summary Report (2003)*. Monitoring was conducted by walking the entire section of stream in the study area at least once and most parts of the stream a number of times during the course of this study. Photos were taken and observations noted during these reconnaissance surveys. Photos are included where available in Appendix C. Areas of particular concern were:

1. A discharge to the bank above the stream approximately 100-feet upstream of Beach Street in Millbrook Meadow was noted by Woodard & Curran in the *Illicit Discharge Detection Summary Report*. This site was visited several times during the course of the study. No significant increase of erosion was observed. (Appendix C, Figure C-1).
2. One of the reasons for undertaking this nonpoint source pollution assessment is the town's concern over the noticeable accumulation of sediment at the head of Mill Pond. Sediments from upstream sources that are carried by the stream are deposited when the stream's velocity slows upon entering the pond. Physical removal of these sediments is likely an impracticable alternative for the town from a financial and logistical perspective. Reducing upstream sediment inputs to slow the rate of accumulation and extend the life of the pond may be the town's only practicable alternative.
3. A number of municipal catch basins were found to be located directly over and therefore discharge stormwater and sediments directly into stream culverts. These included Catch Basin # CB-4 on Beach Street, CB-24 and CB-25 on Poole's Lane,

and CB-33, and CB-34 on Railroad Avenue, and CB-45 at the base of Summit Avenue.

4. A cement retaining wall serving as the south side of an open culvert for the Squam Brook tributary just after it passes under the base of Summit Avenue (at sampling station K) is beginning to cave in toward the stream. This retaining wall is likely privately owned. If it were to collapse at some point in the future, it would cause a significant release of sediment into the stream. Since this location is upstream of Mill Pond, if it were to collapse it is likely that this would be another large dose of sediment added to the already sediment-choked pond. (Appendix C, Figure C-2).

5. One of the tenants at the Isinglass Place commercial plaza just upstream from Railroad Avenue is a day care facility for infants and young children. The facility's "play area" located behind the building abuts a cement retaining wall forming the west bank of the stream. Imported sand from the play area has created a significant sediment source to the stream. A deep sand pile has accumulated in the stream channel at the base of the retaining wall directly below the play area. Sand from this source has likely been transported downstream as well. (Appendix C, Figure C-3).

6. A section of the stream between the Rockport Housing Authority Millbrook Park and Isinglass Place properties (just upstream from sampling station H) across from the Whistlestop Mall parking lot is eroding. It appears that an attempt to impound the stream into a small pond by building a berm across the stream bed has created an erosional situation. Its path blocked by the berm, the force of the stream is in the process of blowing out the west bank of the stream in that location. As the bank is eroded, a large amount of sediment is being released into the stream, adding more sediment to the stream channel and ultimately to Mill Pond where it is retained. (Appendix C, Figure C-4).

7. A significant source of sediments includes the upstream watersheds flowing into the study area. In particular, the Squam Brook watershed has steep slopes and roads with no curbs or drainage systems. These conditions can lead to enhanced overland flow and elevated sediment transport in the runoff. The Squam Brook watershed in particular was identified in a letter from the Natural Resources Conservation Service (NRCS, Oct. 2003) as an area to focus erosion control and sedimentation BMP's.

2.6 Beach Testing Results v. Rainfall 1999 - 2004

Comparison of bacteria levels at the mouth of Mill Brook and Front Beach and rainfall was made by plotting bacteria levels collected for beach testing purposes with rainfall levels measured at a rain gage in Rockport for 1999-2004. These plots can be viewed in Appendix D. For 1999 and 2000, fecal coliform was tested for as the indicator of bacteria-borne pathogens for both the stream and different locations along the beach. In 2001, the target bacteria groups tested for changed to E. coli for the stream (freshwater) and Enterococci for the beach (marine water). Acceptable limits of bacteria for swimming waters are 200 colonies of fecal coliform per 100ml., 235 colonies E. coli per 100ml. and 104 colonies and 61 colonies of Enterococci for marine and fresh water respectively. In the summer of 1999, there were frequent exceedances of the acceptable bacteria limits acceptable for swimming at the mouth of Mill Brook and at least two (2) at Front Beach despite the fact that there were few rainfall events between June and

September. In the summer of 2000, there were again frequent exceedances at the mouth of Mill Brook and at least one exceedance (exceeding 7,500 colonies fecal coliform per 100ml.) measured at the middle of Front Beach. The summer of 2000 exhibited more frequent rain events, and the one beach exceedance appears to be associated with a 0.75 inch rain event. In 2001, no water quality monitoring samples were collected at the mouth of Mill Brook in coordination with the Front Beach monitoring. There was one exceedance (140 colonies of Enterococci per 100ml. on July 23) not apparently associated with a rain event. During 2002, one exceedance for the mouth of Mill Brook occurred on July 10 in association with a 0.5 inch rain event. The summers of 2003 and 2004 saw no exceedances in the marine waters of Front Beach. The summer of 2003 saw four (4) exceedances at the mouth of Mill Brook with the highest reaching a count of 2000 colonies E. coli per 100ml. on July 22 associated with a 0.8 inch rain event. In 2004, only one exceedance at the mouth of Mill Brook was detected prior to commencement of sampling for this study. Over the 1999-2004 periods, it appears that exceedances of acceptable bacteria limits have reduced in number and frequency at Front Beach and the mouth of Mill Brook. An initial comparison showed that elevated bacteria counts were found both during wet and dry periods, implying that there may be more than one vector for bacteria to the beach.

2.7 Water Quality Monitoring

Three (3) rounds of wet-weather water quality monitoring were conducted at twelve sampling locations on the lower Mill Brook during this study period. From each sampling location for each round, water samples were analyzed for four different types of bacteria (Total Coliform, Fecal Coliform, E.coli, Enterococci) and Total Suspended Solids (TSS). In addition, on one occasion, Optical Brightener monitoring was conducted at 5 locations within the study area.

2.7.1 Bacteria and TSS Analysis

After a preliminary investigation and evaluation of the study area, twelve initial sampling locations were identified along the lower section of Mill Brook in coordination with the project partners including the MA Office of Coastal Zone Management, Massachusetts Bays Program, Woodard & Curran, Rockport Dept. of Public Works and Rockport Board of Health. Sampling stations were chosen:

- to coordinate with previous monitoring studies of the stream;
- to divide the study area into logical sub-drainage units;
- to facilitate feasibility of sampling.

Sample collection stations are shown on the GIS Map (insert).

Wet-weather water quality monitoring, collecting water samples from the stream during precipitation events, was proposed because the results would best reflect effects of stormwater impact as pollutants from adjacent land is washed into the stream from runoff. No water quality monitoring specifically targeting wet-weather events had been done on Mill Brook. Wet-weather sampling was also proposed in order to compare the results with targeted dry-weather sampling that had been done the previous year for the *Mill Brook Illicit Discharge Detection and Elimination Summary Report (2003)*. To

qualify as a wet-weather sample, water samples were collected during a precipitation event that occurred after a minimum of 72-hours of dry (no precipitation) weather, and the precipitation event had to be a minimum of a 0.25-inch rain event with rain falling at a minimum of 0.10 inches per hour. Samples were collected following standard bacteria sampling protocols. Collected samples were analyzed at Biomarine of Gloucester, MA, a state-certified laboratory, using *Standard Methods for the Examination of Water and Wastewater, 20th Edition, 1998 and EPA Method 1600, March 2000.* Laboratory data sheets are included in Appendix E.

Results 1st Wet-Weather Sampling Round - September 8, 2004. (Table F-1, Figures F-1 through F-5)

Water Quality monitoring results for the September 8th sampling event showed high counts of all four types of bacteria analyzed. Total Coliform ranged from 4,000 colonies per 100ml. to 180,000 colonies per 100ml. with the lowest counts found at sampling station C just below Mill Pond, and the highest counts found at sampling station D just above Mill Pond. Fecal Coliform counts ranged from 2,300 colonies per 100ml. to 47,000 colonies per 100ml. with the lowest counts again at sampling station C just below Mill Pond and the highest counts found at sampling station F on the Squam Brook tributary just before it entered Mill Brook off Henderson Court. E.coli counts ranged from 1,300 colonies per 100ml. to 16,000 colonies per 100ml. again with lowest and highest values found at sampling station C and F, respectively. Enterococci values ranged from 430 colonies per 100ml. to 16,000 colonies per 100ml. with the lowest counts found at sampling station B just upstream of Beach Street and the highest counts again found at sampling station F on Squam Brook off Henderson Court. Total Suspended Solids (TSS) counts ranged from 19 to 497 mg/L with the lowest counts found at sampling station G just upstream of Railroad Avenue and the highest at sampling station K on Squam Brook at the base of Summit Avenue.

2nd Wet-Weather Sampling Round - November 24, 2004. (Table F-1, Figures F-1 through F-5)

Bacteria counts from the November 24th monitoring events were considerably lower than the September 8th event except for Total Coliform. Total Coliform counts ranged from 2,400 colonies per 100ml. to 190,000 colonies per 100ml. with the lowest counts found at sampling station L on Squam Brook off of Squam Hill Court and the highest counts found at sampling stations D and E on Mill Brook upstream of Mill Pond and off Henderson Court. Fecal Coliform counts ranged from 50 colonies per 100ml. to 2,900 colonies per 100ml. with lowest counts found again at sampling station L on Squam Brook and highest counts found at sampling station A at the mouth of Mill Brook at Front Beach. (Here the duplicate sample measured 2,900 colonies per 100ml., almost 3 times higher than the 1,000 colonies per 100ml. of the original sample for this location.) E.coli counts ranged from 10 colonies per 100ml. to 1,900 colonies per 100ml. with lowest counts again found at sampling station L and highest counts found at sampling station D above Mill Pond. Enterococci counts ranged from 20 colonies per 100ml. to 3,400 colonies per 100ml. with lowest counts found at sampling station L and highest at sampling station A. (The duplicate sample measured 3,400 colonies per 100ml., compared to the 2,500 colonies per 100ml. of the original sample for this location.) TSS

counts ranged from 9 to 312mg/L with the lowest counts found at sampling station L and the highest at sampling station D upstream from Mill Pond.

3rd Wet-Weather Sampling Round - May 7, 2005 (Table F-1, Figures F-1 through F-5).

Bacteria and TSS counts from the May 7th monitoring event were considerably lower than the two previous monitoring events. Total Coliform counts ranged from 1,500 colonies per 100ml. to 7,400 colonies per 100ml. with lowest counts found at sampling station A at the mouth of Mill Brook at Front Beach and the highest counts found at sampling station E on Mill Brook downstream of Railroad Avenue. (The duplicate sample for sampling station A measured 1,500 colonies per 100ml., less than half the 3,800 colonies per 100ml. of the original sample.) Fecal Coliform counts ranged from 70 colonies per 100ml. to 750 colonies per 100ml. with lowest counts found at sampling station F on the Squam Brook tributary before it enters Mill Brook and highest counts found at sampling station E on Mill Brook downstream from Railroad Avenue just upstream from where the Squam Brook enters Mill Brook. E.coli counts ranged from 40 colonies per 100ml. to 670 colonies per 100ml. with lowest counts found at sampling station L on Squam Brook off Squam Hill Court and the highest counts found at sampling station D upstream from Mill Pond. Enterococci counts ranged from 10 colonies per 100ml. to 160 colonies per 100ml. with lowest counts found at sampling station J just upstream of Poole's Lane and highest counts found at sampling station A at the mouth of Mill Brook. (The duplicate sample measured 60 colonies per 100ml., less than half the 160 colonies per 100ml. counted in the original sample.) TSS counts ranged from 4 to 30mg/L with the lowest values found at sampling stations A at the mouth of Mill Brook and C downstream of Mill Pond and the highest values found at sampling station G upstream of Railroad Avenue.

Comparison of Wet-Weather Monitoring Rounds - Bacteria

Water quality monitoring results from the Sept. 8, 2004 sampling event show elevated bacteria counts at almost every location. Sampling stations B and C downstream of Mill Pond for all bacteria measured and sampling station L off Squam Hill Court on Squam Brook for all but total coliform were less elevated than other stations, but still considered elevated based on levels considered safe for swimming. In comparison, monitoring results from the November 24, 2004 sampling event show a significant decline in all bacteria types except for total coliform which continue to be quite high, most notably at sampling stations D and E upstream of Mill Pond and below Railroad Avenue. Sampling station L off of Squam Hill Court had the lowest counts with sampling stations B and C downstream of Mill Pond and sampling station K at the base of Summit Avenue on Squam Brook having some of the lower counts. All bacteria counts dropped significantly between the November 24, 2004 and May 7, 2005 sampling events, although the majority of sampling locations still had bacteria counts in excess of the acceptable limit for swimming waters for all bacteria types. Acceptable levels for swimming are considered to be 1,000 colonies total coliform per 100ml., 200 colonies fecal coliform per 100ml., 235 colonies E. coli per 100ml. and 61 colonies Enterococci (for freshwater) per 100ml.

Factors that could be contributing to the variation in bacteria counts between wet-weather sampling events are: seasonal land use practices by humans and animals, water temperature in the stream, stream baseflow, water table depth, rainfall intensity, length of

time since last significant precipitation event, timing of sampling within the rain event, and stormwater management BMP's. The September 8, 2004 sampling event took place the week immediately following Labor Day weekend. Land use of beaches, parks, roads, parking areas, seasonal residences, etc. would likely have been close to its peak of the summer season just prior to the sampling event. Elevated levels of human (and animal) activity without adequate stormwater management BMP's can result in elevated pollutant levels in the stream at this time of year. Water temperature affects the rate that bacteria reproduce and multiply. Streamwater temperatures in early September can be expected to be much higher than in late November or early May, and thus bacteria will be multiplying more quickly at this time of year. Rainfall intensity and duration, as well as the length of time since the last precipitation event, can impact pollutant levels in the stream. Higher intensity storms, and/or storms of a longer duration, are more likely to produce runoff over a greater proportion of the watershed which can result in greater pollutant delivery to a stream. Also, a longer period of time since the last significant rainfall will allow for more pollutants to build up within the watershed before being "washed" into the stream by a precipitation event. Rainfall intensity is related to both the volume and duration of precipitation. It can be highly localized and would require a dense network of more sophisticated equipment to take more precisely timed measurements of a rainfall event over time. For this study, the depth measurement taken at a single rain gage was used to estimate precipitation volume over the entire watershed area. Comparison of the three events shows total rainfall depths of 0.58 inches, 0.49 inches and 1.28 inches respectively. In the case of duration since last significant rainfall, defined as a rainfall event equally or exceeding 0.25 inches, a duration of 18 days had passed since a significant rainfall event before the 9/8/2004 event, a duration of 11 days since a significant event prior to the 11/24/2004 event, and 8 days prior to the 5/7/2005 event. Relative rainfall amounts and length of time since last rainfall can be viewed in Figure F-6 where E. coli bacteria results for five sampling stations are plotted against a rainfall curve over the sampling period. Duration since previous rain event could be one of several factors that might help explain the higher bacteria counts of the 9/8/2004 event. Though actual intensity could have been higher for a smaller storm, the precipitation

The quantity of water in the stream at the time of sampling can affect bacteria counts through dilution. Since the wet-weather sampling protocol for this study required that there be 72 hours without rain before a sample could be taken, Mill Brook was likely in a "baseflow" condition just before each sample. "Baseflow" is the water in the stream during periods without rainfall, and it is supplied by the adjacent alluvial aquifer. In other words, it is ground water discharge. Bacteria counts are measured in numbers of colony forming units or "colonies" of a certain group of bacteria per 100 ml. of water collected. Even if the rate of delivery of a certain type of bacteria through any of a number of different vectors were consistent across different sampling events, the quantity of water in the stream at the time of the sampling event may cause the sampling results to be quite different because of dilution.

In the case of Mill Brook over the period of sampling, we do not know the baseflow quantity in the stream because there is not an active stream gage in this area. However, we can make assumptions based on knowledge of a typical annual hydrograph for a stream in this region, which can be supported by comparisons with a "proxy" stream that is gaged. Typically, a good proxy stream will have a similar watershed size, relief, land

use and land cover, geology, and climate. A typical stream in this region under normal or average annual rainfall conditions will follow a fairly predictable baseflow pattern throughout the year, where baseflow will typically be lowest in August and September, will gradually rise starting in October, and will peak in April before evaporation and transpiration begins to draw down the ground water tables again. This pattern is corroborated by the baseflow periods evident in the annual hydrograph of a proxy stream (USGS 01104455, Stony Brook, Unnamed Tributary 1, near Waltham, MA) located approximately 30 mi. from Rockport which has a comparable watershed size, topography and land use. The proxy stream follows the expected baseflow curve with the lowest baseflow observed in August, rising gradually through the fall and then peaking in early spring. The only variation observed in the proxy hydrograph is that in the spring of 2005, the peak in annual baseflow was delayed from the 3-year average of early April, and actually occurred in early May at approximately the same time as the 5/7/2005 sampling event in Rockport. Baseflows increased approximately 50 percent between early September and late November and rose by a factor of five between November and early May. Though the actual baseflow will not be the same between Rockport's Mill Brook and the proxy stream, the relative rate of baseflow and stream volume can be extrapolated for comparison purposes. This pattern of lowest baseflow during the September sampling round, somewhat higher in the November sampling round, and then much higher in the May sampling round may help explain some of the variation between bacteria counts observed in Mill Brook between sampling events. The very large increase in stream flow between November and May could explain a lot of the large variation between the bacteria counts from those two sampling events, with the concentration of bacteria strongly diluted by a much larger baseflow volume in May. The smaller relative variation in baseflow between the September and November sampling events (a 50 percent rise in the proxy stream) does not appear to entirely explain the significant drop in bacteria counts between those dates.

One final factor that should be considered is use of stormwater management BMP's. In particular, practically all of the municipally owned catch basins in the lower Mill Brook watershed study area were cleaned out between the September and November sampling dates. It is unknown how much impact this might have had on bacteria levels in the stream, but it may help explain some of the drop in bacteria levels. In conclusion, it is likely that several factors help explain some degree in the variation between bacteria counts between sampling events. These include a rise in baseflow volume tending to "dilute" bacteria concentrations, a drop in seasonal activity of land use by humans and animals, drop in water temperature (at least from the early September sampling date), length of time since last significant precipitation event, and catch basin cleaning in the lower Mill Brook watershed. It doesn't appear that the overall rainfall amount as measured by a rain gauge had a significant effect on bacteria counts at least for these specific events sampled.

Comparison of Wet-Weather Monitoring Rounds - TSS (Table F-1, Figure F-5)

Other than at two sampling stations, sampling stations K and L on Squam Brook with higher TSS measurements in the September sampling event, TSS measurements were highest during the November sampling event and in almost all cases, were significantly lower in the May sampling event. Some of the factors that might explain a variation in

TSS load measured in a stream include more natural factors such as intensity and volume of precipitation event and length of time since last significant runoff event. Other more human manipulated factors might include land use changes creating or exacerbating a sediment source to the stream, physical disturbance in the stream channel causing resuspension, and use of stormwater management BMP's which might be expected to reduce sediment sources to the stream. In this case, it is curious to note that the highest TSS measurements (other than the two stations in September that were noted above) were observed in the sampling round that occurred after most of the municipal catch basins were cleaned in the study area watershed. The November sampling round occurred three to four weeks following catch basin cleaning, so it isn't believed that the elevated TSS measurements were a direct result from catch basin cleaning. One possible explanation might be that catch basins with very high sediment volumes might have acted as "check" dams, preventing sediment from being flushed through drainage pipes into the stream. Cleaning the "clogged" basins may have allowed sediment to flow more freely through drainage pipes to be discharged to the stream. This conjecture is speculative. This phenomenon is a difficult one to explain.

Comparison of Wet and Dry-Weather Water Quality Monitoring within the Study Area

In 2002 and 2003, Woodard & Curran (W&C) conducted dry-weather water quality monitoring at five locations on the lower section of Mill Brook for the *Mill Brook Illicit Discharge Detection and Elimination Summary Report (2003).* Dry-weather monitoring was conducted for the same four types of bacteria (Total Coliform, Fecal Coliform, E.coli, Enterococci) in December, 2002, July 2003, and September 2003 at sampling locations corresponding closely with sampling stations used in this study, sampling station A Beach St-Downstream, sampling station C Mill Pond-Downstream, sampling station D Mill Pond Upstream, sampling station J Poole's Lane Upstream and sampling station K Squam Brook-Summit Ave Downstream. Figure F-7 shows a visual comparison of E. coli counts for the three dry-weather monitoring events with the three wet-weather monitoring events conducted as part of this study. In general, bacteria counts were much higher on the first wet-weather monitoring event and somewhat higher in the second wet-weather monitoring event than during the dry-weather monitoring with the exception of sampling station D Upstream of Mill Pond. Bacteria counts at sampling station D exhibited fecal coliform and E. coli counts somewhat comparable to the November 2004 wet-weather sampling during the July and September, 2003 dry-weather sampling. One interesting note is the dramatic variation in sampling results between the September 2004 wet-weather sampling and the September 2003 dry-weather sampling at these five stations.

2.7.2 Optical Brightener Testing

One week-long round of optical brightener testing was conducted in September, 2004. Optical brightener sampling involves anchoring a sampling pad in a flowing or standing water body for a period of time and then collecting the sampling pad and "reading" it for signs of fluorescent white dye after drying. Fluorescent white dye is commonly found in detergents. A positive reading for fluorescent white dye can indicate a source of sewage or gray water from a home or business. The test is used where high bacteria counts or nutrient measurements are found to help identify if they are from a human or non-human source.

Optical Brightener media were deployed in the field at five locations from September 1 to September 8, 2004. The five locations corresponded to water quality sampling locations as specified in Table F-2. The sampling period also corresponded to the first wet-weather WQ monitoring event of September 8, 2004 as reported above. Each clean gauze pad was deployed in either a plastic coated cage or plastic mesh bag and anchored in the stream using fishing line. At the end of the sampling period, each pad was retrieved, placed in a clean plastic sealable bag, and returned to the office where they were dried on a clothesline before being read under a black light for presence of fluorescent white dye. Optical brightener testing was done in close coordination with Rockport Board of Health staff who have performed this testing before, and results were read by an experienced Board of Health employee. Results as reported in Table F-2 were negative - no presence of fluorescent white dye detected, from four of the five sampling locations and a "possible" positive from one location, sampling station G just upstream from Railroad Avenue. No detection of the presence of fluorescent white dye at four locations only a "possible" presence at one location helps support a theory that high bacteria levels in these parts of the stream are more likely from a stormwater source than from a failing septic system or illicit sewer connection draining into the brook.

2.7.3 Preliminary Conclusions Drawn from Water Quality Monitoring Results

1. Very high bacteria counts were observed for the September 8, 2004 wet-weather water quality monitoring event. Comparison with dry-weather monitoring results from 5 of the same sampling stations from September 2003 suggest that wet-weather conditions may explain at least part of this striking variation. Other environmental and human-driven factors may also be at play.

2. With few exceptions, bacteria counts dropped significantly from the September 8, 2004 monitoring event to the November 24, 2004 monitoring event, and dropped significantly again from the November 24 monitoring event to the May 7, 2005 monitoring event. It is thought that the dilution factor of a rising stream baseflow may have played a significant role in these sampling results. Other factors, including change in seasonal land use, stream water temperature, length since previous rain event and cleaning of catch basins in the lower Mill Brook watershed may have also played a role.

3. Sampling Station D Upstream of Mill Pond consistently exhibits some of the higher bacteria counts across monitoring events (both wet- *and* dry-weather events) and bacteria types. Sampling Station D is receiving stream inputs from both Mill Brook and the Squam Brook tributary and is downstream of Railroad Avenue and its associated commercial district. Urban wildlife activity may also be contributing a bacteria source to this thick, wooded section of the brook.

4. Sampling Station C downstream of Mill Pond consistently has some of the lowest bacteria counts across the three monitoring events and across the four different types of bacteria. It appears that over the duration of the sampling event, the residence time of water in Mill Pond serves to both dilute and detain suspended bacteria.

5. Other than high counts of total coliform bacteria during the September sampling event, sampling station L off of Squam Hill Court also exhibited some of the lower bacteria counts between the three sampling events. This is interesting to note in relation to the fact that the Squam Brook watershed, the sub-drainage area for sampling station L, is unsewered and entirely served by septic systems for residential wastewater treatment.

6. Other than high counts at sampling stations K and L on the Squam Brook tributary at the September water quality monitoring event, TSS measurements were much higher during the November sampling event than the September or May sampling events. This observation is somewhat unexpected based on the fact that the catch basins in the lower Mill Brook watershed were cleaned between the September and November sampling events, which one may have expected to result in lower TSS measurements in the stream.

7. Almost entirely negative results (other than a "possible" positive at sampling station G upstream of Railroad Avenue) from optical brightener testing suggests that the elevated bacteria counts observed are more likely related to nonpoint source pollution factors than illicit sewer connections or failing septic systems in the lower Mill Brook watershed. However, additional periodic optical brightener testing may help confirm this.

3. Development of Nonstructural BMPs

3.1 Sediment Budget

Data was collected to construct a rough course sediment budget for the study area for the winter of 2004-2005. Throughout the winter of 2004-2005, estimates were made of the amount of sand deposited on the streets within the study area due to hazardous road conditions. The municipal catch basins within the study area were cleaned out in the first and second weeks of October of 2004 and then again in the second and third weeks of May 2005. This allowed the town to estimate the rate of sediment accumulation within each catch basin over a known period of time (approximately 7 months). Also, estimates were made on sediment removal from the study area from street sweeping conducted in the fall before the winter sanding season and again in the spring. Forty-four (44) municipal catch basins were identified within the study area. Twenty-three (23) privately-owned catch basins were also identified.

Catch Basin Cleaning – 1st Round. (See Appendix G for data sheets.) The total amount of sediment removed from the study area from the October 2004 round of catch basin cleanings was estimated at approximately 370 cubic feet (13.5 cubic yards.). The largest amounts of sediment were removed from catch basin # 5, 13, 14, 16 and 28 located at King Street and Beach Street, King Street, Holbrook Court, and Poole's Lane respectively, each with 25 cu. ft. or greater of sediment. Because there were no definitive records on the last time any of these catch basins had been cleaned, it isn't possible to extrapolate rates of accumulation from this data. The catch basins that were the most full of sediment were catch basin # 2, 5, 13 – 16, 28 and 30 which ranged from 25% - 50% full. At least one study (*Pitt, 1985*) concluded that catch basins can capture sediments up to approximately 60% of the sump volume. When sediment fills greater than 60% of their volume, catch basins no longer reliably capture sediment from storm flows. They can lose as much sediment to resuspension and washout as they capture. Because percent volume was calculated using total basin volume instead of more correctly as sump volume, it can be assumed that any basin that has reached 50% capacity is no longer effectively removing sediment from stormwater flows.

Catch Basin Cleaning – 2nd Round. (See Appendix G for data sheets.) The total amount of sediment removed from the catch basins in May 2005 were approximately 310 cu. ft. (11.5 cu. yds.), eighty-four percent (84%) of the volume that was removed the previous fall. The largest volumes were removed from catch basin #12 (not cleaned in the fall because it was not accessible) with 25 cu.ft., #'s 13 and 14 with 19 cu.ft., all from King Street. The catch basins with the highest average daily rate of accumulations (greater than 0.05 cu.ft. per day) between the two cleanings were catch basin #'s 1, 3, 5 though 7, 13 through 15, 23, and 28 through 31. Appendix G shows the catch basin data and highlights those catch basins with the most sediment removed, the highest percent full, or the highest average daily accumulation rate between the two cleanings. Those catch basins highlighted should be targeted for a more intensive cleaning schedule (up to monthly), while the others may function adequately with fewer cleanings per year.

Street Sweeping. Street sweeping was conducted in the study area in late October of 2004 and again in late April of 2005. It was estimated that 2 cubic yards of street sediments were removed from the area on each occasion.

Winter Street Sanding. An estimate of sand introduced to the study area through street sanding was made by reviewing Dept. of Public Works records. It was estimated that throughout the winter of 2004-2005 that approximately 25 loads of sand and salt were spread on streets within the study area. When the proportion of each 6 cu. yard. load that is comprised of salt is subtracted, this number translates into approximately 135 cubic yards of sand added to the study area. When compared to the approximately 2 yards removed in late April by street sweeping and the approximate 11.5 cubic yards removed in early May by catch basin cleaning, the amount of sediment that was "recovered" by catch basin cleaning and street sweeping represents approximately ten percent of the sediment known to have been deposited. Year after year of street sanding with only a 10 percent recovery rate from catch basin cleaning and street sweeping adds up very quickly to a significant sediment source to a watershed. It can be assumed that some portion of that is entering Mill Brook through the drainage system and through overland flow. Attenuation of these sediments (combined with sediments from many other sources) in Mill Pond will eventually cause the loss of this valuable and cherished resource.

3.2 Catch Basin and Culvert Cleaning Plan

A catch basin cleaning plan was developed to provide guidance to the town in managing the cleaning and sediment removal of municipal catch basins. The Catch Basin Cleaning Plan is located in Appendix H. The plan gives guidance on reasons for cleaning catch basins, on handling and disposal of catch basin cleanings and a catch basin cleaning schedule and includes a template catch basin cleaning reporting sheet. There does not appear to be a definitive source or a definitive schedule for cleaning catch basins. Different sources included guidance that basins should be cleaned a minimum of one to two times per year (*Aronson, 1983 in an EPA publication*), should be cleaned a minimum of four times a year (*MA DEP Stormwater Policy*), or should be cleaned up to five times a year (*NRCS pers. Com.*). There does seem to be consensus among sources that more frequent cleanings (up to once a month) maximizes the amount of sediment collected on an annual basis and thus reduces the amount that has the potential to be washed downstream through resuspension. Part of municipal stormwater management involves weighing the cost and efficiency of sediment recovery of more frequent cleanings against other forms of stormwater BMP's such as spending funds on retrofitting catch basins with deeper sumps to hold more sediments or more frequent street sweeping.

Promoting the use of the reporting sheet (template provided) will allow the town to keep track of which basins fill more quickly and which may need to be cleaned more frequently. At some point in the future if Rockport adopts a GIS-based mapping system, the information collected on the Reporting Sheets may be able to be incorporated into a town-wide GIS system to track sediment collection on a town-wide basis.

3.3 Street Sweeping Plan

A street sweeping plan was developed to provide guidance to the town in removing sediment and debris from municipal streets. The Street Sweeping Plan is located in Appendix I. The plan provides guidance on reasons for sweeping streets, disposal of

street sweepings, and on a street sweeping schedule and includes a template reporting sheet for use by the Dept. of Public Works highway crew. As with the catch basin cleaning, there doesn't appear to be a definitive street sweeping schedule. The recommended frequency ranges from monthly with more frequent sweepings as needed (*NRCS, pers. Com.)* to weekly (*Claytor, 1999*). More frequent street sweeping may help reduce the number of annual catch basin cleanings needed to maintain optimal catch basin treatment capacity.

3.4 Stormwater Control By-law

The Town of Rockport is required to comply with the National Pollution Discharge Elimination System (NPDES) Stormwater Phase II general permit and the NPDES Phase II plan adopted by the town to meet the requirements of this program. NPDES Phase II is implemented by the U.S. Environmental Protection Agency (EPA) and the MA DEP. Under the NPDES Phase II permit and the Town of Rockport NPDES Phase II plan, the town is required to adopt a by-law that will regulate erosion and sediment control off of construction sites and will regulate the post-construction impacts of stormwater off of any new or redevelopment construction. As part of the Lower Mill Brook Nonpoint Source Pollution Assessment project, language from by-laws and ordinances developed for or enacted by other municipalities was reviewed for purposes of developing a draft by-law for the town of Rockport.

As part of this undertaking, project partners met with the town Planner, the Building Inspector who also serves as the town's Zoning Agent, the Director of Public Works, the Conservation Agent and the Board of Health Agent. They also met with the Rockport Planning Board, at their meeting of May 19, 2005 to discuss steps for the adoption of a stormwater control by-law for the town of Rockport. Project partners have also met with a representative from the City of Gloucester's Engineering Department which is in the process of updating the city's stormwater by-law. Further meetings with the Rockport Planning Board and City of Gloucester Engineering Dept. will be held in an effort to further refine a model by-law to meet the needs to the town of Rockport. Benefits for Rockport of working with the City of Gloucester as they update their own stormwater by-law are to take advantage of the expertise from a municipality that has already been implementing stormwater controls for several years; the town of Rockport benefiting from the greater staffing and financial resources of the much larger neighboring community; and the possibility of consistency between stormwater protections between the neighboring communities strengthening the protection for many of their shared resources.

Although there are some minimum requirements that must be incorporated into a stormwater management by-law to comply with the NPDES Phase II stormwater permit requirements, the town has a fair amount of latitude to choose how it administers the by-law. Some of the minimum requirements are that the by-law must:

1. Prohibit non-stormwater discharges into the municipal drainage system, including enforcement procedures and actions;
2. Control construction site runoff:
    a. apply to sites of 1 acre or more of disturbance;

b. require erosion and sedimentation controls;

c. include a process for public input;

d. ensure site plan review, including pre-construction review;

e. considers potential water quality impacts;

f. require control of construction waste; provide for site inspections; and

g. has procedures for enforcement of control measures.

3. Control Post Construction Stormwater Management from New Development and Redevelopment

   a. apply to sites of 1 acre or more of disturbance;

   b. require apply to projects that newly discharge runoff to the municipal stormwater drainage system;

   c. apply to projects that disturb 1 acre of land or more;

   d. require permanent stormwater controls to minimize water quality impacts;

   e. ensure long-term operation and maintenance of structural stormwater controls.

Some of the more important choices that town officials will need to make in drafting and adopting a stormwater management by-law are: Should it be a general or zoning by-law? Which town department or board should administer it? How should fees be collected, held and appropriated? There are of course pros and cons on both sides of the argument for a zoning or general by-law. Several points came out of reviewing model language and other literature on the subject. The more important issues are related to grandfathering provisions of zoning by-laws, enforcement provisions are different, and zoning by-laws would allow issuance of building permits and certificates of building occupancy to be contingent on compliance.

There are several factors that make both the Dept. of Public Works and the Rockport Planning Board the appropriate department or board for administering the by-law. Factors in favor of the Dept. of Public Works administering the stormwater by-law include the fact that it is a relatively large department in Rockport and might be better able to assume the added review requirements, and the Dept. of Public Works has staff that have training and experience in using and reviewing runoff calculations and hands-on experience in implementing stormwater management procedures. Factors in favor of the Planning Board administering such a by-law (if the stormwater by-law were to be adopted as a zoning by-law), are that the Planning Board already administers Site Plan Review and Sub-division Control and other zoning by-laws. The Planning Board has the ability to take consulting fees from applicants and could therefore hire an outside reviewer to provide the technical review. Potentially, some of the review tasks could be split between the Dept of Public Works and the Planning Board, but in any case, both departments should have input into the process. Both the Conservation Commission and Board of Health are likely too small, one full- or part-time agent with part-time support staff, to assume an additional review process. Both of these boards should have input into the review of the applications though.

Another aspect of the stormwater management and erosion and sediment control by-law that the town can choose are size thresholds for triggering review or requiring a permit. NPDES Phase II Stormwater permit requires that a minimum land disturbance area of 40,000 square feet trigger review and permit requirements. Many towns that have adopted stormwater management and erosion and sediment control by-laws have chosen to reduce this threshold of total land disturbance area in order to have the ability to implement runoff and sedimentation controls on a larger proportion of construction projects. If the town were to choose a much lower land disturbance threshold, it could also consider establishing lower filing requirements for the smaller projects that have less potential to create runoff and sedimentation impacts. Other aspects of a stormwater management by-law that the town could implement are regulations that encourage Low Impact Development (LID) and use of disturbance minimizing techniques and natural landscape features to encourage infiltration over or in conjunction with more traditional structural stormwater management techniques.

3.4.1 Construction Site Erosion and Sediment Control

The purpose of a construction site erosion and sediment control by-law is be to require construction sites of a determined threshold size to meet established performance standards for controlling erosion and sediment transport off the site. Currently only construction sites immediately adjacent to wetland resource areas which are required to receive an Order of Conditions from the Conservation Commission or development regulated by the Planning Board under Subdivision Control or Site Plan Review have any requirements for using erosion or sediment control BMP's. Yet a fair number of development or redevelopment projects that can include a significant amount of land disturbance and have the potential for large amounts of sediment transport are not regulated. In many of these cases, the construction site will be upslope of a catch basin which can introduce large quantities of sediment-laden runoff to the town drainage system which can quickly overburden the town's drainage infrastructure and ultimately end up in sensitive wetlands or coastal waters.

In Appendix J, draft stormwater by-law language is presented that includes a component requiring that an erosion and sediment control plan be approved as part of a Stormwater Management Permit. This model was developed by the Pioneer Valley Planning Commission.

3.4.4 Post-Construction Runoff

The purpose of a stormwater management by-law regulating post-construction stormwater runoff is be to require impacts to meet established performance standards minimizing impacts from post-construction runoff to sensitive wetland and coastal resource areas, to the town's drainage and sediment collection infrastructure and to adjacent properties. Most model by-law language requires proponents of development projects above a specified land-disturbance or building footprint threshold size to implement a combination of non-structural and structural BMP's such that impacts from post-construction are equivalent to or less than runoff from pre-construction conditions.

In Appendix K, a model stormwater by-law is presented that was developed for the neighboring south shore communities of Marshfield, Plymouth, and Duxbury. This model by-law which is still undergoing review and fine revisions was funded by the

Massachusetts Office of Coastal Zone Management (MA CZM) and is based on a model first drafted by MA DEP to meet the Massachusetts Stormwater Policy Guidelines. One strength of this model is that it the body of the by-law contains only the core principles of its purpose and functions. Associated regulations, also still under development, will provide the details of administration. This model, however, focuses almost entirely on post-construction runoff, and does not address construction site erosion and sedimentation control. If the Town of Rockport were to adopt this or similar model by-law language, it could choose to add construction site erosion and sediment control to the post-construction provisions of this model, or it could choose to adopt separate, stand-alone but compatible by-laws.

4. Structural BMP Feasibility and Design

The firm of Woodard &Curran was hired to evaluate the feasibility of implementing structural BMP's to reduce sediment and pollutant loads entering Mill Brook within the Study Area. In addition to their own familiarity with the lower Mill Brook watershed from their work on the *Mill Brook Illicit Discharge Detection and Elimination Summary Report (2003), Woodard & Curran* used the data collected by the project partners to conduct their feasibility analysis, including wet-weather water quality monitoring results for bacteria and suspended solids, mapping of sub-drainage areas within the study srea, mapping of catch basins and outfalls within the study area and data collected on depth, diameter, and inflow and outflow pipes for each catch basin and sediment volume removal from each catch basin.

Woodard & Curran's recommendations for structural BMP's in the lower Mill Brook watershed included a phased approach, dependent on town financial and staffing resources for implementation. These recommendations included:

1. All catch basins in the lower Mill Brook watershed area be equipped with hoods. A hood is an inverted elbow or similar device that is installed over the outlet pipe. It is designed to retain oils and other floatables within the catch basin sump to prevent them from flowing downstream to the storm drain outfall. These could cost approximately $250 per hood plus installation costs and require a minimum of a 3-foot deep sump for installation.

2. Catch basin sumps be a minimum of four (4) feet deep. The sump is the distance from the invert of the outlet pipe to the bottom of the catch basin. The total depth recorded for the following catch basins was four feet or less and should be targeted for increased sump depth: CB-2, CB-3, CB-16, CB-17, CB-19, CB-23, CB-30, CB-35, CB-37, and CB-38. However, some of these catch basins may not be "true" catch basins (e.g., direct discharges or infiltrating catch basins); these structural improvements should be implemented only after further inspection by the Rockport Department of Public Works. This would most likely require the complete replacement of the existing catch basin with a new pre-cast catch basin with an estimated cost of approximately $5,000 per replaced catch basin. However, costs could be significantly higher due to presence of shallow depth to bedrock in many Rockport locations.

3. Erosion control should be implemented as needed from Pooles Lane to Railroad Avenue. Mill Brook's natural stream banks have eroded at one location in this area in particular adjacent to the Rockport Housing Authority Mill Brook Park housing complex. (See also description under Section 2-5(6) under Monitoring Erosion and Sedimentation.) W&C recommends using "soil bioengineering" techniques, such as BioLog® or BioD-RollTM. These natural biodegradable products could be used to provide the needed protection until establishment of mature vegetation, which will resist erosion forces. These methods not only provide strong, sustainable, aesthetically pleasing water edges, but also provide habitats for aquatic & terrestrial wildlife and filter run off water. The cost in materials would be approximately $350 per 30 foot section of stabilized stream bank.

4. Drainage system modifications should be made where catch basin discharge directly to the brook. The following five (5) "catch basins" do not have sumps; rather

they discharge through the street directly into Mill Brook: CB-4 on Beach Street, CB-24 and CB-25 on Poole's Lane, and CB-33, and CB-34 on Railroad Avenue (also CB-45 at the base of Summit Avenue.) In some cases, inlet pipes were also observed in the catch basins, indicating that additional drainage is discharging untreated to the brook at this point. W&C recommends that these catch basins be relocated to one side of Mill Brook. These catch basins, once relocated, should meet the requirements of a hooded deep sump catch basin as described in this letter and in the Massachusetts *Stormwater Policy Handbook* (March 1997). Relocating these direct discharges into catch basins would require installation of one (and possibly two) new catch basins to capture this runoff before it enters the brook. As noted above, installation of a new catch basin would cost approximately $5,000.

5. Installation of Vortechnics® Stormwater Treatment Systems in an off-line configuration in or near Railroad Avenue on culverted portions of both Mill Brook and Squam Brook. W&C chose Vortechnics® instead of other brands of water quality BMPs because the Town of Rockport has already installed Vortechs® units in another area of town (Thatcher Road). Town staff is familiar with the operation and maintenance required for these BMPs. This unit removes contaminated sediment, floating hydrocarbons, and debris from stormwater through the use of centrifugal force and gravity settling. It is also equipped with baffles for flow control to eliminate turbulence and provide positive removal efficiencies throughout the full range of operation. The Vortechs® unit can achieve up to 80% total suspended solids removal. Depending upon size, the cost of the units would range from $40,000 to $50,000 *per unit* with additional costs for design and installation by a hired contractor.

Woodard & Curran's letter addressing feasibility of structural BMP's is included in Appendix L. These recommendations provide a valuable plan for the town to consider in using structural BMP's to help address stormwater and sedimentation issues in the lower Mill Brook watershed. In a meeting between the consultant and town of Rockport Dept. of Public Works director and staff, some of the constraints to implementing some of the recommendations were discussed. Some of these constraints included:

1. A minimum of a 3 foot sump was required to install a hood and not all catch basins in the lower Mill Brook watershed have a 3 foot deep sump.
2. In many locations, it may not be possible to install catch basins with the recommended 4-foot sump without involving expensive and possibly impracticable ledge removal.
3. One area noted in particular where bank stabilization efforts were recommended between Poole's Lane and Railroad Avenue is not under town control. This area is on the edge of two properties adjacent to the brook, one privately owned and one state-owned and under control of the local Housing Authority. If the property owners were willing, the town could offer to partner with them to address the erosion and sedimentation issue created by the eroding banks.
4. Installation of new catch basins (approximately $5,000 per unit) or the larger more sophisticated and effective Vortechs® units (approximately $40,000 to $50,000 per unit) have high costs associated with them. Individual catch basin cost may not seem high. However, a large number of them (at least 15 individual

basins) have been recommended for replacement. This could represent a large budget item for a town to undertake in any one year. With this in mind, the town could target replacement of a smaller number (two or three) a year to spread costs out over several budget cycles. The town could also pursue funding opportunities through state or federal grant programs focused on addressing sedimentation and pollution issues to coastal resources.

5. Installation of Vortechs® stormwater treatment units in Railroad Avenue on both the main branch of Mill Brook and the Squam Brook tributary could block traffic from passing on this heavily used thoroughfare in downtown Rockport for up to several days and disrupt traffic for up to a couple of weeks. To reduce the impacts to residents and businesses, the town could work with owners of the Whistlestop Mall and the Massachusetts Bay Commuter Railroad to re-route traffic around the work where possible.

6. It was identified by a senior Dept. of Public Works employee that drainage pipes in the Railroad Avenue area, in particular the northwest end of Railroad Avenue in Sub-Drainage Area F, which includes the culverted portion of Squam Brook, are significantly undersized. Undersized drainage pipes could reduce the effectiveness of a new stormwater treatment unit installed in that area. The town should evaluate the drainage pipes in this area and consider replacing them with appropriately-sized drainage pipes in conjunction to the installation of a Vortechs® stormwater treatment unit in this area or in a phased approach.

5. Outreach & Education of Town Boards/Officials

The project partners have been meeting and interacting with town staff on a regular basis regarding the nonpoint source pollution assessment of the lower Mill Brook during the project period. Town staff have made some valuable contributions to the project through in-kind matching services. At the same time, project partners have used these interactions as opportunities to impress on town staff the importance of controlling pollutant sources to Mill Brook.

During the project period, project partners also individually met with town boards and committees, including the Watershed Protection Committee, the Planning Board, the Conservation Commission, the Board of Health and the Dept. of Public Works Commissioners to inform them about the study and preliminary results and discuss with them the role that their respective committees can play in helping to control pollution sources to Mill Brook and other water bodies in town. At the culmination of the project, a presentation was made by the project partners at an advertised meeting of the Board of Dept. of Public Works Commissioners where the results and recommendations of the project were presented to the DPW Commissioners and the public.

6. Recommendations

6.1 Septic Systems

Continue to investigate whether there may be any accessory dwellings relying on septic systems for sewage disposal. Review of the Board of Health and Dept. of Public Works records identified only 14 parcels containing septic systems within the lower Mill Brook watershed. Town officials have suspected that there may be additional lots in particular on the west side of Mill Brook just above Mill Pond where town records may show that the dwelling is connected to the municipal sewer, but an auxiliary dwelling or summer cottage may exist on the lot that is not connected to the sewer. Town officials should continue pursuing the potential that additional dwellings may exist that are not connected to the municipal sewer in the study area.

6.2 Direct Discharge Outfalls

1. Map connectivity of municipal drainage system within the study area using GIS. As part of the Lower Mill Brook nonpoint source pollution assessment study, the catch basins were mapped in the study area, and critical information was collected about each basin, including diameter, depth, type of basin (sump, infiltration or direct discharge), and number, orientation and character of all inflow and outflow pipes. It was beyond the limits of this study to map the exact connectivity of each catch basin. However, the collected information gives the town a strong start in undertaking this important task. Mapping the connectivity of the catch basins in the downtown area is part of the Town of Rockport NPDES Phase II Stormwater Plan and should be completed as part of this plan.

2. Determine where the catch basins off Holbrook Court discharge within the watershed. One outfall pipe was discovered leading from a yard off King Street Court, but no municipal drainage pipes were observed behind Holbrook Court. Some of the catch basins on Holbrook Court were observed to accumulate sediments fairly rapidly and therefore, it might be assumed, also discharge some overflow and resuspended sediments to the downstream wetland. This is immediately upstream from the head of Mill Pond, it could be having a significant impact on the pond.

3. Request that the commuter railroad operator (Massachusetts Bay Commuter Railroad) submit plans and information to the town documenting the location and character of the railroad station drainage structures and where they discharge. The town has no documentation of the drainage structures at this railroad station and train layover facility. This is probably one of the areas in town of posing a high risk of possible contamination to surface waters through stormwater discharge to the adjacent Mill Brook. The town should require that the commuter railroad operator submit regular operation and maintenance (O&M) reports on their maintenance and cleaning of these drainage systems, and require mandatory reporting of any observation of contaminants that have entered the drainage system.

4. Smoke test individual pipes to determine their source. Require all applicable outfalls to become properly permitted under the MA Wetlands Protection Act, Rockport Wetlands By-law and federal regulations if applicable.

5. Adopt a town by-law forbidding any new, untreated outfalls to discharge into Mill Brook (and other streams in town.)

6. Remove or relocate the pipe blocking the flow of the stream and retaining debris in the culvert under Beach Street. An approximately 18 inch diameter metal pipe passes through the culvert under Beach Street. This pipe catches debris, reduces the hydraulic capacity of the culvert, increasing the potential for upstream flooding, and could impede or impact upstream eel passage.

6.3 Erosion and Sedimentation

1. Encourage the King Street property owner who is directing runoff from his or her property to the bank above Mill Brook in Mill Brook Meadow to use a dry well, vegetated swale or other means of infiltration rather than promoting erosion and overland flow on a steep bank leading to a stream. The Rockport Conservation Commission may be able to accomplish this through contacting the property owner and educating them about the possible impacts of the discharge and alternative techniques of addressing their runoff issues. Adopting a stormwater management by-law may help reduce the incidences of such discharges in the future.

2. The town should take whatever steps it can and follow all practicable recommendations to reduce the amount of sediments entering Mill Pond. Mill Pond is acting as a sink for upstream stormwater sediments. Sediments accumulating at the head of Mill Pond will gradually reduce the size of the pond and slowly convert it from an open water pond into an emergent wetland over time. Physical removal of these sediments is likely an impracticable alternative for the town from a financial and logistical perspective. Reducing upstream sediment inputs to slow the rate of accumulation and extend the life of the pond may be the town's only practicable alternative and should be made a high priority.

3. Address catch basins discharging sediment- and pollutant-laden stormwater directly to Mill Brook. See Structural BMP recommendation letter by Woodard & Curran.

4. Proactively address the collapsing retaining wall on the channelized portion of Squam Brook at the base of Summit Avenue. The town should investigate ownership of the retaining wall that separates the adjacent residential property from the stream channel at this location to determine if any easements exist. The town should work with the property owner through whatever means are available to repair or replace the retaining wall to prevent the cumulative sedimentation that is currently taking place and to prevent an all-out collapse that could release a huge load of sediment to the brook and create a significant flooding event in this area.

5. Address the sand deposition from the child day care facility play area accumulating in Mill Brook. The Rockport Conservation Commission or Dept. of

Public Works should contact the facility manager and discuss with them ways that the sand could be better contained on the site and prevented from deposition and accumulation in the stream. Pulling the play area away from the edge of the retaining wall by several feet and using a low landscaping tie retaining wall to contain sand in the play area, creating a modest vegetated buffer zone between the play area and the stream channel would help reduce the sand deposition.

6. Address eroding banks on section of stream adjacent to Rockport Housing Authority property. This erosion appears to be caused or exacerbated by a berm erected across the stream channel. The Rockport Conservation Commission should work with the Rockport Housing Authority which manages the land to remove any structures that are causing or worsening erosion of stream banks. The town should work with the Housing Authority to stabilize the banks to prevent further sedimentation of Mill Brook. See also Structural BMP recommendations letter by W&C for stream bank stabilization techniques.

6.4 Water Quality Monitoring

1. Conduct additional wet- and dry-weather Water Quality monitoring within the lower Mill Brook watershed during the summer and fall to determine if the high bacteria counts observed in the September 2004 sampling were an anomalous or predictable occurrence. Future sampling could most likely safely eliminate the total coliform count in order to reduce the cost of sampling since the total coliform indicator is the most general of the bacteria tests conducted and the least likely to be a strong indicator of human-derived pollutants.

2. The GIS data layers should be utilized for further analysis of the hydraulic features and nonpoint source pollution parameters in the study area. Further analytical techniques using GIS could include spatial representation of water quality monitoring results, network tracing, stormwater modeling, sediment loading to the brook, etc. in conjunction with other GIS layers which may be available from MassGIS, CZM, or other agency data libraries.

6.5 Street Sanding

When replacing or upgrading street sanding equipment, purchase equipment that allows the operator to easily gage and select application rates. Allowing the operators to easily gage and manipulate application rates between areas that require heavier sanding and those that require less could save the town money by more efficiently applying sand where it is most needed and could result in less sand being deposited within a given watershed. Based on the low recovery rate (less than 30 percent for the lower Mill Brook watershed for the winter of 2004-2005) from street sweeping and catch basin cleaning, winter street sanding could be having a significant impact on sediment load to sensitive wetland resources.

6.6 Catch Basin Cleaning

1. Increase the frequency of catch basin cleaning in the lower Mill Brook watershed and other areas where municipal drainage systems drain to critical areas such as freshwater streams or wetlands, water supply watersheds or coastal waters to the extent practicable, a minimum of once to twice a year.

2. Encourage town highway maintenance crews to fill out catch basin data sheets (sample in Appendix H) for each basin each time it is cleaned to start compiling a record of rates of accumulation to help determine which basins may need to be cleaned out more frequently than others.

3. Collect information on depth of sump (depth to bottom of catch basin from bottom of outflow invert), type of system (basin-to-basin or basin-to-manhole) and basin construction (pre-cast concrete, stone, brick, etc). These parameters were added to the model reporting sheet after the initial round of catch basin cleaning and data collection. It is important to collect this information for all catch basins within the lower Mill Brook watershed.

4. Make an effort to clean and collect data on CB 41 and CB 44 which were not cleaned as part of this study. CB 41 is located in the middle of Mill Brook Meadow and collects overland flow and flooding from the meadow and transfers it to Mill Brook via an outlet on the brook (Outfall #7). It was not cleaned during the October nor May catch basin cleaning rounds during this study because the ground around the catch basin was too saturated and would have lead to too much ground disturbance to take the vactor truck into the meadow to clean it. Sediments should be removed from this basin either by hand or via the vactor truck at a time when the ground is either very dry or frozen. CB 44 is located in the parking area to the commuter railroad station, just up from the entrance to the Ace Hardware store. It was initially mistaken as a railroad operator-maintained basin during this study, and therefore not cleaned. This basin is on town land and should be made part of the town's regular cleanings.

5. Work with private land owners within the lower Mill Brook watershed to have privately-owned catch basins cleaned and maintained on a regular basis.

6. Request that private land owners within the lower Mill Brook watershed keep a log of the maintenance and cleaning of their privately-owned catch basins and submit this information to the town on a regular (annual) basis.

6.7 Street Sweeping

1. Encourage highway maintenance crews to use the template street sweeping log sheet provided with the Street Sweeping Plan or an equivalent reporting sheet, to track street sweeping activities and estimate recovery rates.

2. Sweep streets regularly in areas that drain to sensitive wetland or coastal resource areas. More frequent street sweeping may help reduce the number of annual catch basin cleanings needed to maintain optimal catch basin treatment capacity and may reduce the amount of sediment entering wetland resources through drainage systems or overland flow.

3. When the town replaces or upgrades its existing street sweeping equipment, it should choose new equipment that has been demonstrated to remove finer particles and associated pollutants. Older models may pick up large-grain sized sands and other large debris, but have been demonstrated to have very low removal efficiency for finer particles or other pollutants.

6.8 Stormwater By-law

Implement an erosion and sediment control and stormwater management by-law(s) that requires construction sites and post-construction new development and redevelopment to meet established performance standards and provides for the administration, review and enforcement of the by-law by the town-approved authority, including the collection of reasonable fees and issuing of appropriate permits. The model by-laws provided in the Appendix J&K provide strong models on which to base its by-law(s). Town officials, particularly the Rockport Planning Board through it's interest in implementation of the zoning by-law and the Dept. of Public Works as the town entity responsible for complying with NPDES Phase II stormwater permit requirements, should work together with other town boards and departments including town counsel to draft by-law language that will be effective in minimizing runoff and sediment related impacts. The town should also use the resources of MA CZM which has offered its resources to assist the town in adopting a stormwater management by-law.

6.9 Structural BMP's

1. Install hoods in all catch basins in the lower Mill Brook watershed where adequate depth of sump allows. A hood is an inverted elbow or similar device that is installed over the outlet pipe. It is designed to retain oils and other floatables within the catch basin sump to prevent them from flowing downstream to the storm drain outfall. These are relatively inexpensive and easy to install. However, one constraint is that it requires a minimum of a 3-foot deep sump for installation. Not all catch basins in the lower Mill Brook watershed have sufficient sump depth to accommodate a hood.

2. Catch basin sumps be a minimum of four (4) feet deep. Approximately ten catch basins were identified in the study area with total depths 4 feet or less. Since the sump depth (the distance from the invert of the outlet pipe to the bottom of the catch basin) is less than the total depth, it can be assumed that these basins have less than the 4-foot recommended sump depth. Extending the sump depth would most likely require the complete replacement of the existing catch basin with a new pre-cast catch basin with an estimated cost of approximately $5,000 per replaced catch basin. However, costs could be significantly higher due to presence of shallow depth to bedrock in many Rockport locations. In many locations, it may not be possible to install catch basins with the recommended 4-foot sump without involving expensive and possibly impracticable ledge removal.

3. Erosion control should be implemented as needed from Pooles Lane to Railroad Avenue. Mill Brook's natural stream banks have eroded at one location in this area in particular adjacent to the Rockport Housing Authority Mill Brook Park housing complex. (See also description under Section 2.5 Monitoring Erosion and Sedimentation.) W&C recommends using "soil bioengineering" techniques, such as BioLog® or BioD-RollTM. These natural biodegradable products could be used to provide the needed protection until establishment of mature vegetation, which will resist erosion forces. These methods not only provide strong, sustainable, aesthetically pleasing water edges, but also provide habitats for aquatic & terrestrial wildlife and filter run off water. The cost in materials would

be approximately $350 per 30 foot section of stabilized stream bank. One constraint is that the land on which the erosion is taking place is operated by the local Housing Authority and is not under town control. The town could offer to partner with the Housing Authority to address the erosion and sedimentation issue created by the eroding banks.

4. Drainage system modifications should be made where catch basin discharge directly to the brook. Six (6) "catch basins" were identified that do not have sumps; rather they discharge through the street directly into Mill Brook. In some cases, inlet pipes were also observed in the catch basins, indicating that additional drainage is discharging untreated to the brook at these points. These catch basins should be relocated to one side of Mill Brook. When relocated, they should be made to meet the all requirements of a hooded deep sump catch basin as prescribed in the Massachusetts *Stormwater Policy Handbook* (March 1997). Relocating these direct discharges into catch basins would require installation of one (and possibly two) new catch basins to capture this runoff before it enters the brook. As noted above, installation of a new catch basin would cost approximately $5,000. Replacing a large number of catch basins at one time could represent a large budget item for a town to undertake in any one year. With this in mind, the town should target replacement of a smaller number (two or three) a year to spread costs out over several budget cycles. The town should also pursue funding opportunities through state or federal grant programs focused on addressing sedimentation and pollution issues to coastal resources.

5. Vortechs® Stormwater Treatment Systems should be installed in an off-line configuration in or near Railroad Avenue on culverted portions of both Mill Brook and Squam Brook. These units remove contaminated sediment, floating hydrocarbons, and debris from stormwater through the use of centrifugal force and gravity settling. It is also equipped with baffles for flow control to eliminate turbulence and provide positive removal efficiencies throughout the full range of operation. The Vortechs® unit can achieve up to 80% total suspended solids removal. The Town of Rockport has installed Vortechs® units in another area of town (Thatcher Road), and town staff is already familiar with the operation and maintenance required for these BMPs. Depending upon size, the cost of the units would range from $40,000 to $50,000 *per unit* with additional costs for design and installation by a hired contractor.

6. Installation of Vortechs® stormwater treatment units in Railroad Avenue on both the main branch of Mill Brook and the Squam Brook tributary could block traffic from passing on this heavily used thoroughfare in downtown Rockport for up to several days and disrupt traffic for up to a couple of weeks. To reduce the impacts to residents and businesses, the town should work with owners of the Whistlestop Mall and the Massachusetts Bay Commuter Railroad to re-route traffic around the work where possible.

7. It was identified by a senior Dept. of Public Works employee that drainage pipes in the Railroad Avenue area, in particular the northwest end of Railroad Avenue in Sub-Drainage Area F which includes the culverted portion of Squam Brook, are significantly undersized. Undersized drainage pipes could reduce the

effectiveness of a new stormwater treatment unit installed in that area. The town should evaluate the drainage pipes in this area and consider replacing them with appropriately-sized drainage pipes in conjunction to the installation of a Vortechs® stormwater treatment unit in this area or in a phased approach.

8. Installation of new catch basins (approximately $5,000 per unit) or the larger more sophisticated and effective Vortechs® units (approximately $40,000 to $50,000 per unit) have high costs associated with them The town should pursue funding opportunities through state or federal grant programs focused on addressing sedimentation and pollution issues to coastal resources.

7. Summary

The Lower Mill Brook assessment has assisted the town in addressing nonpoint source pollution issues in this coastal watershed. Periodic high bacteria counts measured in the waters of Front Beach and the mouth of Mill Brook and increasing visual evidence of sediment accumulation threatening Mill Pond have made town officials and residents aware that pollution issues exist. This study has helped identify the possible pollution sources and given the town some of the tools it will need to address the issues.

Through this project, the town has created a Geographic Information System (GIS) based map of the lower Mill Brook watershed including all identified septic systems, municipal and private catch basins, and direct discharge outfalls within the watershed, allowing for spatial analysis of these parameters. Three (3) rounds of targeted wet-weather water quality monitoring were conducted, one of which recorded a particularly high spike in bacteria counts during a rain event in early September, 2004. The subsequent two (2) monitoring events in November 2004 and May 2005 recorded lower counts but still elevated over acceptable limits established for swimming waters. As part of the nonstructural BMP development of the project, the town conducted catch basin cleaning and street sweeping in the watershed where data was collected on the catch basin infrastructure and on sediment collection. Comparison with records for winter street sanding highlighted a striking discovery that only approximately ten percent (10%) of the sand deposited through winter street sanding was "recovered" during the spring round of catch basin cleaning. This left over 120 cubic yards of sediment from one source unaccounted for in the watershed after only one winter. Catch Basin Cleaning and Street Sweeping Plans with template recording and data sheets were developed to help guide the town in the maintenance of these systems and encourage detailed record keeping to help develop a targeted approach to street sweeping and catch basin cleaning. Model by-law language was reviewed with two (2) models in particular being focused on for their strengths in the areas of construction site erosion and sediment control and post-construction stormwater management BMP's. Elevating awareness of town officials and residents on stormwater issues and BMP's that can be used to address them was a goal of the project. This was conducted throughout the study through working closely with town staff from different departments and meeting with individual boards in public meeting forums. Continued outreach and public discussion between town officials and the pubic will be necessary in particular to work towards a comprehensive stormwater management by-law that will be adopted and be successful for the town of Rockport.

Finally, many valuable recommendations were developed during the course of this study. In particular, recommendations for structural BMP improvement could help improve the town's recovery and removal rate of sediments from the lower Mill Brook watershed before they are able to enter the sensitive wetland and coastal resource areas. Some of the recommendations for structural BMP implementation have constraints including the physical constraint of very shallow bedrock throughout much of Rockport and ever-present financial constraints of a small town budget. Spreading the implementation of the recommendations over time and pursuing state and federal funding opportunities will help address some of these constraints. A comprehensive approach including reducing the source of sediment and other pollutants through erosion and sediment control, minimizing runoff from development, regular street sweeping and catch basin cleaning,

and continuing to monitor and pursue the source of any wastewater detected in the watershed as well as improving treatment of these pollutants that do enter the town's stormwater system by upgrading the infrastructure as practicable, should help reduce impacts to the town's sensitive and valued freshwater and coastal resources.

References

Aronson, G., D. Watson, and W. Pisaro. 1983. *Evaluation of Catch Basin Performance for Urban Stormwater Pollution Control.* EPA-600/2-83-043

Klein, R., Urbanization and Stream Quality Improvement", in Water Resources Bulletin, American Water Resources Assoc., Vol. 15, No. 4, August 1979.

Massachusetts Dept. of Environmental Protection (MA DEP), "Fact Sheet: Management of Catch Basin Cleanings", April 2004

Massachusetts Dept. of Environmental Protection (MA DEP), Stormwater Policy Handbook, Vol. 1: Stormwater Management, 1997

Mill Brook Illicit Discharge Detection and Elimination Summary Report, Woodard & Curran for the Town of Rockport, 2003.

Mineart, P. and S. Singh. 1994. *Storm Inlet Pilot Study.* Woodward-Clyde Consultants. Alameda County Urban Runoff Clean Water Program. Oakland, CA

Pitt, R. 1985. *Bellevue Urban Runoff Project.* Final Report

Stormwater Manager's Resource Center, "Pollution Prevention Fact Sheets: Catch Basins"

Wellhead Protection Plan: Mill Brook Watershed, Rockport, Massachusetts, Town of Rockport, 2004

Appendix A: Parcels on Septic Systems in the Lower Mill Brook and Squam Brook Watersheds

Table A-1

Lower Mill Brook Watershed -- Parcels on Septic

MAP	LOT	ADDRESS	Y/N?	SEPTIC Last Inspected?	P/F?
11	25A	11 Haddow Rd.	y		
11	25B	9 Haddow Rd.	y		
11	25C	8 Haddow Rd.	Y	Inspctd. In 2004	Passed
11	25D	6 Haddow Rd.	Y		
11	25E	4 Haddow Rd.	Y	Inspctd in 2005	Failed
11	36	5 Haddow Rd.	Y	Inspctd. In 2004	Failed
11	52D	7 McKay's Dr.	Y	Inspctd. In 1991	Passed
11	52E	9 McKay's Dr.	Y		
11	52J	8 McKay's Dr.	Y	Inspctd in 1996	Passed
11	52K	10 McKay's Dr.	Y		

Table A-2

Squam Brook Watershed Parcels on Septic

MAP	LOT	ADDRESS	Y/N	SEPTIC Date	Title 5?
4	1	Squam Rd.	Y		
4	1A	Squam Rd.	Y		
4	2	Off Squam Rd.	N		
4	2A	37 Squam Rd.	Y		
4	2B	43A & 43B Squam Rd.	Y		
4	3	80 Squam Rd.	Y		
4	3A	Squam Rd.	N		
4	3B	Old Squam Path	N		
4	3C	78 Squam Rd.	Y		
4	4	61 Squam Rd.	Y		
4	10	Johnson's Quarry	Y		
4	15	Johnson's Quarry	N		
4	18	Johnson's Quarry	N		
4	21	Johnson's Quarry	N		
6	1	Gloucester Watershed	N		
11	23	Off Summit Ave.	N		
11	54	5 Squam Hill Crt.	Y		
11	55	7 Squam Hill Crt.	Y	1997	Y
11	57	6A & 6B Squam Hill Crt.	Y		
11	60	9 Squam Rd.	Y	1982	
11	61	11 Squam Rd.	Y	2004	Pumped
11	62	13 Squam Rd.	Y	2004	Pumped
11	63	15 Squam Rd.	Y		
11	64	17 Squam Rd.	Y		
11	65	19 Squam Rd.	Y		
11	66	21 Squam Rd.	Y		
11	67	23 Squam Rd.	Y		
11	69	25 Squam Rd.	Y		
11	70	27 Squam Rd.	Y		
11	71	Squam Rd.	N		
11	72	31 Squam Rd.	Y		
11	73	33 Squam Rd.	Y		
11	73A	35 Squam Rd.	Y		
11	75	Squam Rd.	N		
11	76	Squam Rd.	N		
11	76A	Squam Rd.	N		
11	77	26 Squam Rd.	Y		
11	78	22 Squam Rd.	Y		
11	78A	24 Squam Rd.	Y		
11	80	2 Reilly's Ln.	Y		
11	81	20 Squam Rd.	Y		
11	82	18 Squam Rd.	Y	1998	Y
11	88	16 Squam Rd.	Y		
11	89	29 Squam Hill Rd.	Y		
11	90	31 Squam Hill Rd.	Y		
11	91B	35 Squam Hill Rd.	Y		
11	91C	37 Squam Hill Rd.	Y		

Appendix B: Direct Discharge Outfalls; Description, Photos, and Permit History

Table B-1: Outfalls Identified on Lower Mill Brook

Figure No.	Outfal I No.	W&C Report	Location/ Description	Size (in.)	Material	Notes
B-1	1	1	Under Beach St. 10' above downstream headwall; east side.	10	Iron	
B-2	2	*	Under Beach St., 25' above downstream headwall; source east side.	14	Clay	
B-2	3	*	Under Beach St., 35' above downstream headwall; source both sides.	8	Metal	abandoned pipe?
B-3	4	*	Under Beach St., 15' above upstream headwall; source west side.	18	Reticulated aluminum	
B-4	5	2	Channel wall 15' upstream of Millbrook Meadow headwall; source east	10	Reticulated aluminum	New pipe?
B-5	6	*	50' upstream of meadow headwall; source east side.	2	PVC	Has been observed w/ dry weather discharge; Observed 11/28/04 w/ strong discharge.
B-6	7	*	Across from catch basin in meadow: source catch basin. East side.	8	Metal	Half-submerged. End of pipe broke up.
-	8	12	Coming from a yard on King St. Crt.; source west side.	6	PVC	
B-7	9	13	Stormdrain outfall upstream of bridge at 9 RR Ave.; source west side	12	Clay	
B-8	10	14	110' downstream of RR. Ave. headwall; source west side.	8	Corrogated metal	Pipe exiting embankment at a stee slope. Appears to be from Rockpo Service Stn. Building.
-	11	*	Pipe coming from yard; source east side.	1	White PVC	Coming from yard?
B-9	12	15	80' below downstream headwall at RR. Ave.; source east side.	4	PVC	Pipe exiting yard behind RR Ave.
B-10	**13**	**16**	**60' below downstream headwall at RR. Ave.; source east side.**	**4**	**Corrogated plastic**	**From yard behind RR Ave.**
B-10	**14**	**17**	**60' below downstream headwall at RR. Ave.; source east side.**	**4**	**Corrogated plastic**	**From yard behind RR Ave.**
B-11	15	*	Under RR Ave. 38' below upstream headwall . East side.	18	Clay	Some water observed in pipe.
-	16	*	Under RR Ave. 33' below upstream headwall. East side.		Catch basin	Direct outfall from street with 24" concrete piipe inlet from west.
-	17	*	Under RR Ave. 7' below upstream headwall. East side.		Catch basin	Direct outfall from street with 24" concrete piipe inlet from west.
B-12	18	*	At base of upatream RR Ave. headwall; source east side.	5	Metal	Dry weather discharge observed; invert at water level.
B-13	19	4	23' up from upstream headwall at RR Ave.; source east side.	18	Corrogated metal	
B-14	20	5	46' up from upstream headwall at RR Ave.; source west side.	2	Corrogated plastic hose	Coming from restaurant?
B-14	21	6	46' up from upstream headwall at RR Ave.; source west side.	4	PVC	
B-15	22	*	58' up from upstream headwall at RR Ave.; source west side.	5	Asbestos?	Half-submerged.
B-16	23	*	81' up from upstream headwall at RR Ave.; source west side.	6	PVC	Discharges parallel to brook.
B-17	24	*	92' up from upstream headwall at RR Ave.; source west side.	1.5	White PVC	
B-18	**25**	**7**	**124' up from upstream headwall at RR Ave.; source west side.**	**12**	**PVC**	**Source Isinglass parking lot.**
B-19	26	8	134' up from upstream headwall at RR Ave.; source west side.	2	PVC	Pipe hidden in stone wall of channe signs of recent discharge.
B-20	27	9	142' up from upstream headwall at RR Ave.; source east side.	12	Corrogated metal	In concrete wall of channel 8' upstream from #26
B-21	**28**	**10**	**235' up from upstream headwall at RR Ave.; source west side.**	**12**	**Green PVC**	**Half submerged pipe: source Isinglass parking lot. Lots of orange floculents; dry discharge**
B-22	29	*	Located far end of Isinglass parking lot; source west side.	12	Green PVC	Half-submerged pipe; source
B-23	30	*	Millbrook Park property; source west side.	2	PVC	source fish pond?
B-24	31	*	15' downstream of footbridge; source west side.	6	Clay/ Asbestos	
B-25	32	11	Under cap stone; 10' upstream from footbridge; source east side.	18	Corrogated plastic	Ties into catch basin in Millbrook Park parking lot.
-	33	*	Under Poole's Ln.		Catch basin	Direct discharge from road; additioı inlets
-	34	*	Under Poole's Ln.		Manhole	
-	35	*	Under Poole's Ln.		Catch basin	Direct discharge from road; additioı inlets

* Not referenced in W&C report.

Bold typeface is used to highlight those outfalls with known permits. In the case of Outfalls #13 and 14, it is not possible to determine from the permit which of these two discharge pipes the permit applies to.

Figure B-1: Outfall #1

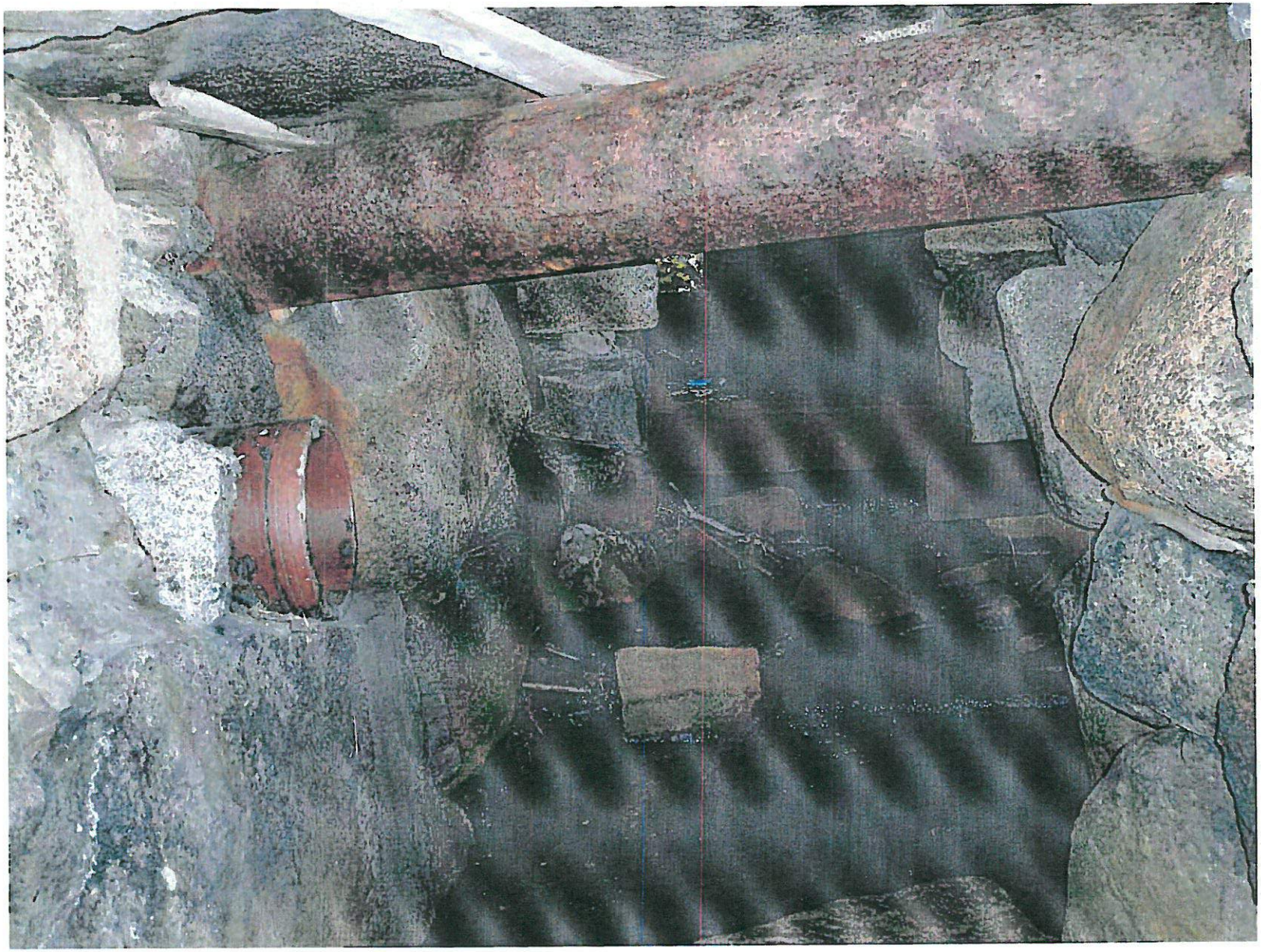


Figure B-2: Outfalls #2 and #3

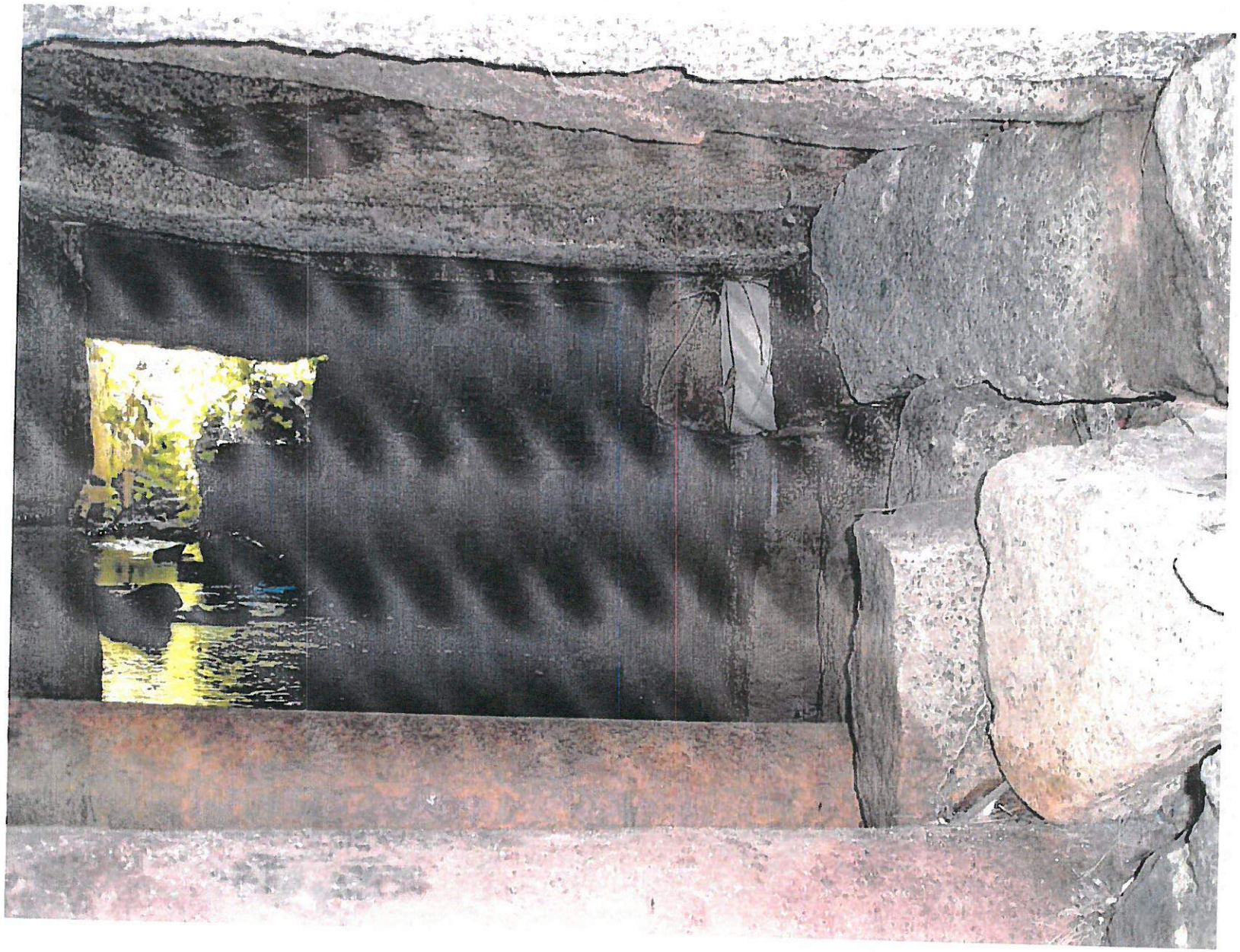


Figure B-3: Outfall # 4

Figure B-4: Outfall #5

Figure B-5: Outfall #6

Figure B-6: Outfall #7

Figure B-7: Outfall #9

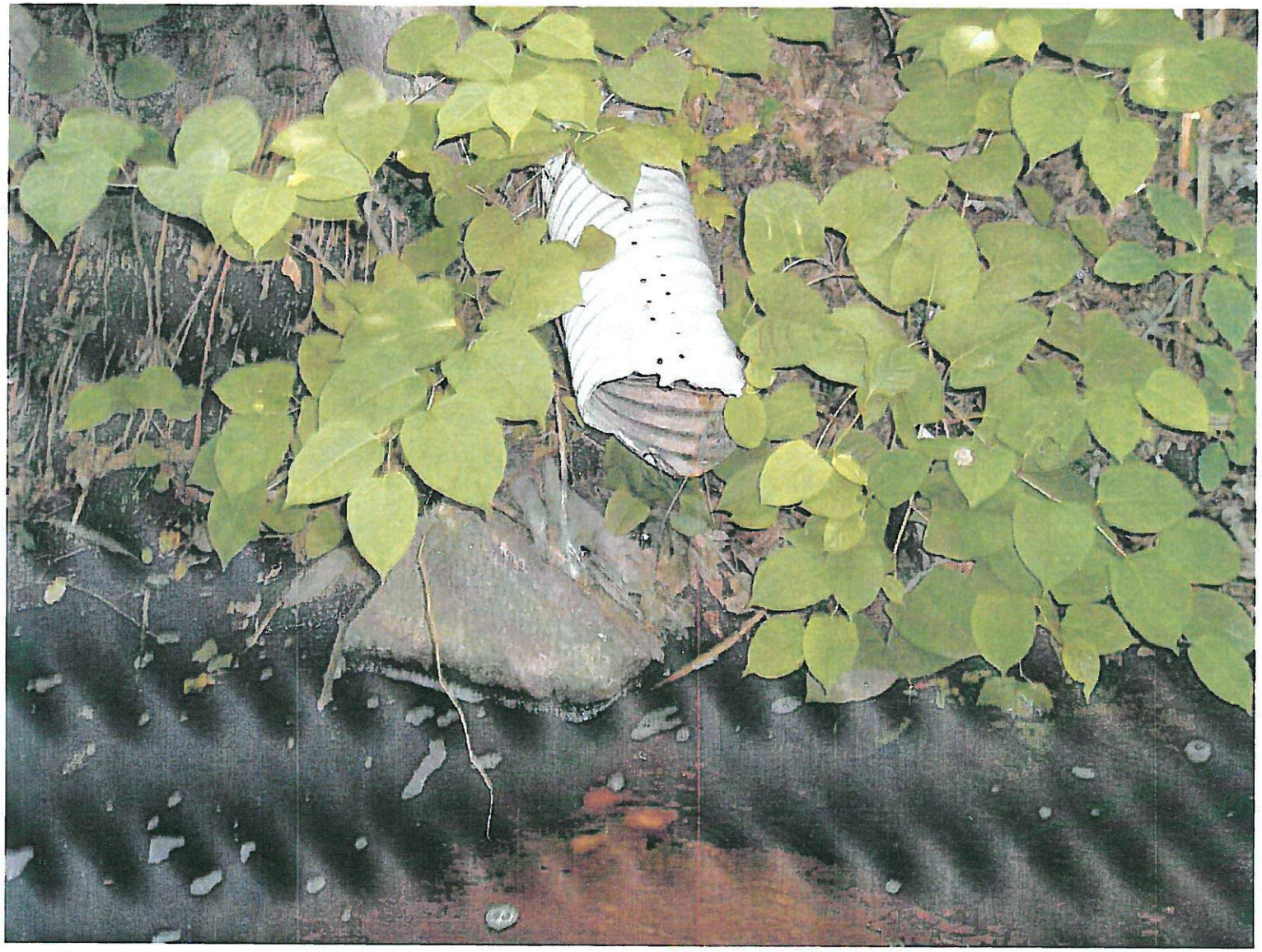


Figure B-8: Outfall #10

Figure B-9: Outfall #12

Figure B-10: Outfall #13 and #14

Figure B-11: Outfall # 15

Figure B-12: Outfall #18

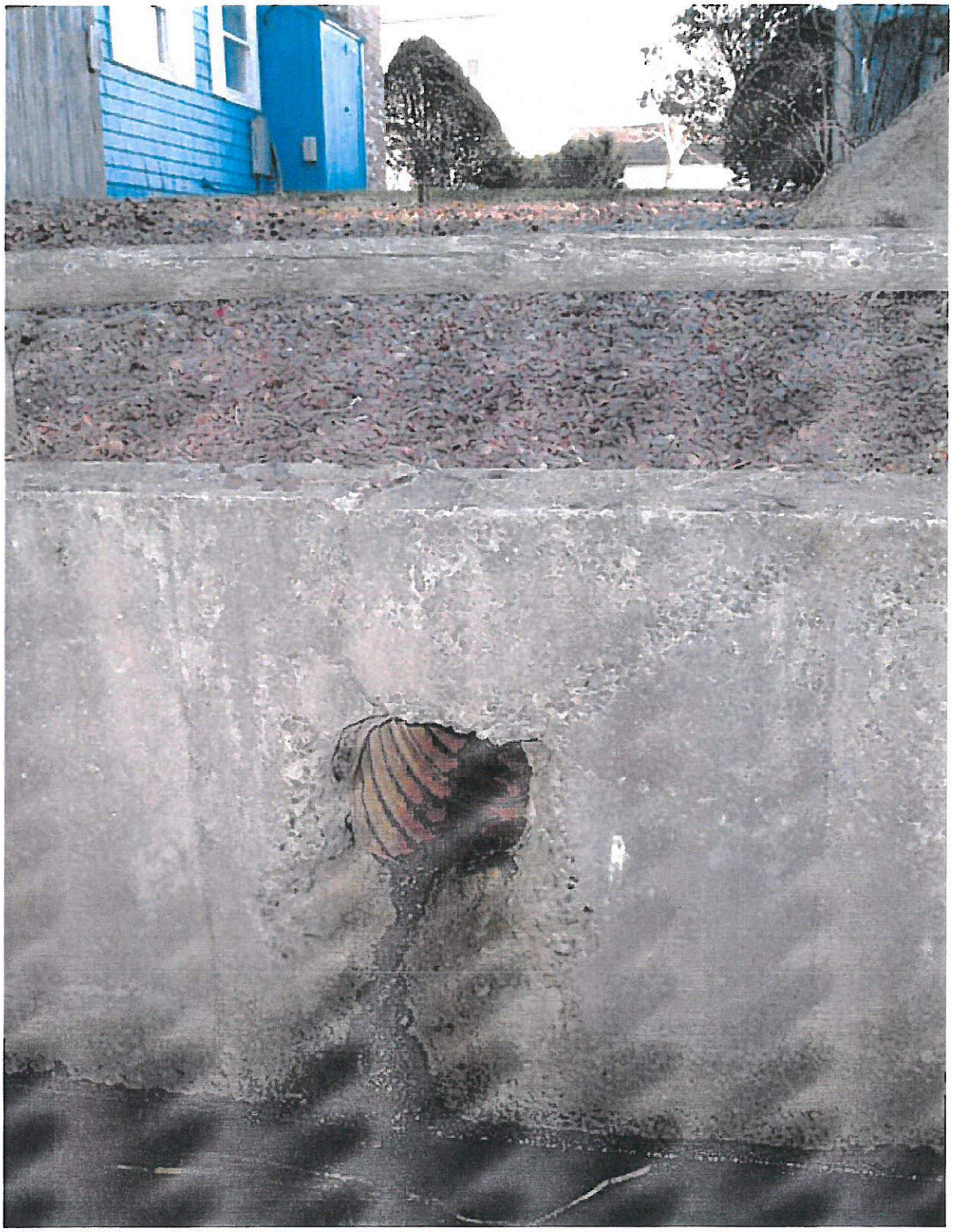


Figure B-13: Outfall #19

Figure B-14: Outfall #20 and #21

Figure B-15: Outfall #22

Figure B-16: Outfall #23

Figure B-17: Outfall #24

Figure B-18: Outfall #25

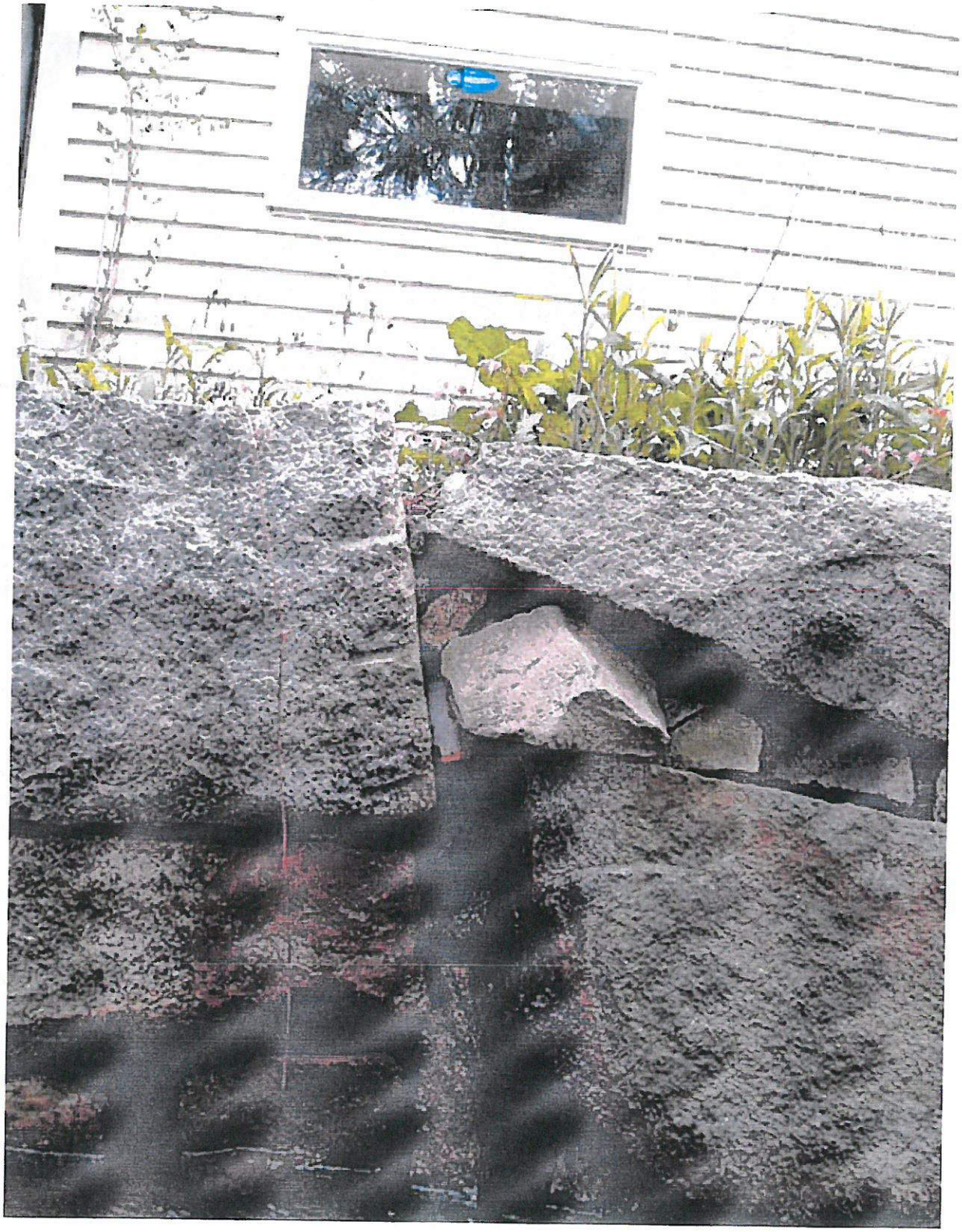


Figure B-19: Outfall #26

Figure B-20: Outfall #27

Figure B-21: Outfall #28

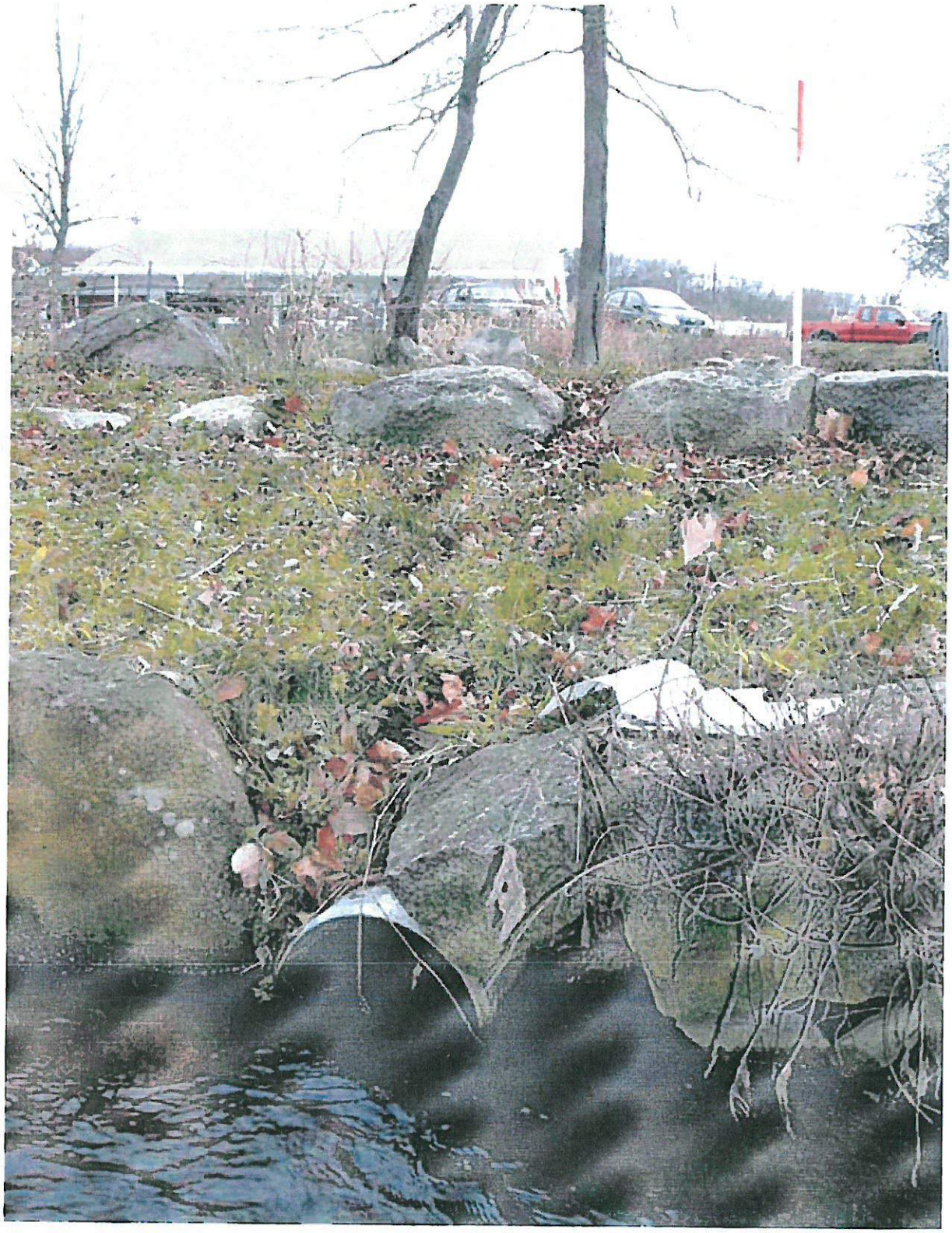


Figure B-22: Outfall #29

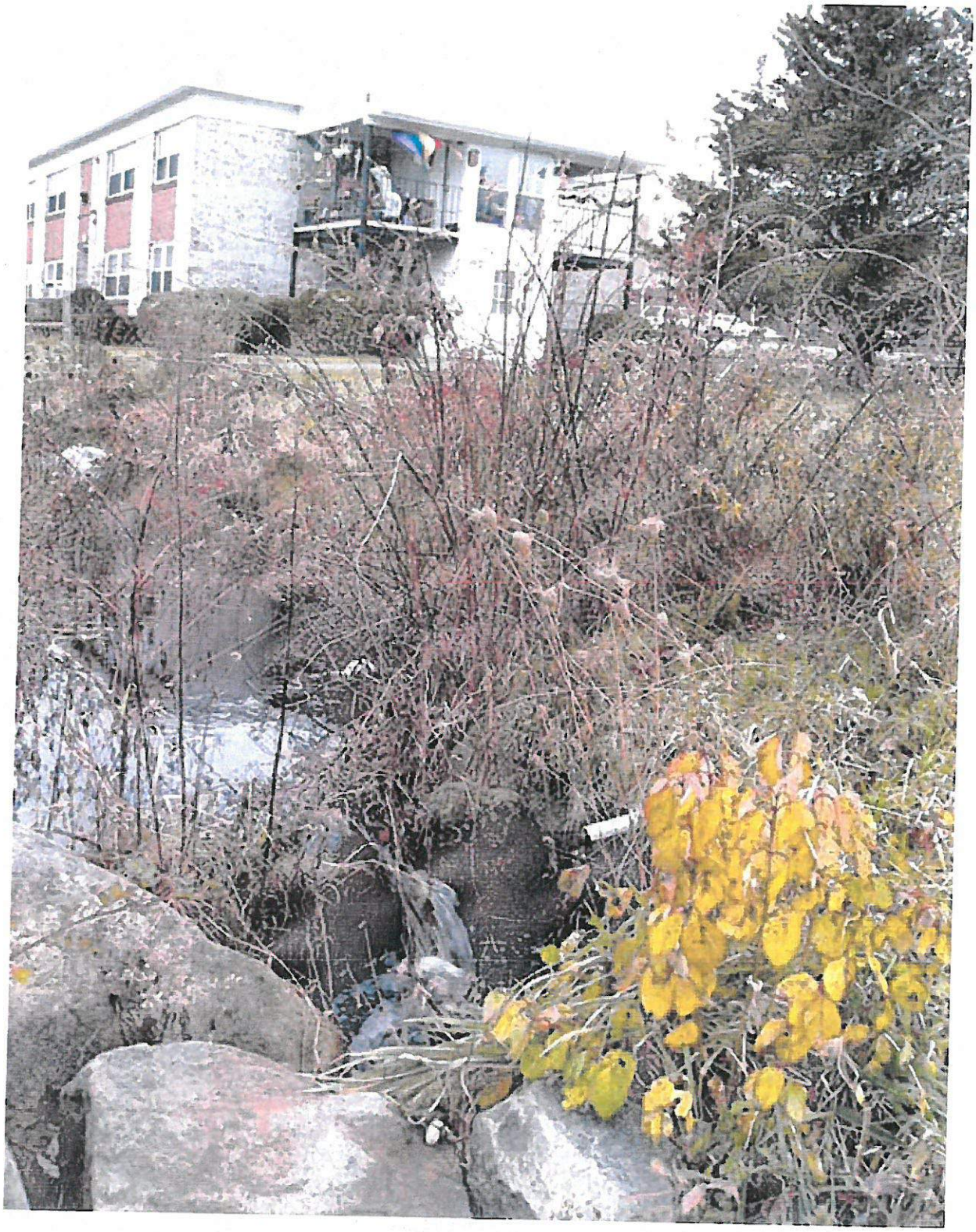


Figure B-23: Outfall #30

Figure B-24: Outfall #31

Figure B-25: Outfall #32

Table B-2

Environmental Permits for Outfalls in Lower Mill Brook Study Area

MAP	LOT	ADDRESS	OWNER	PERMIT TYPE	PERMIT #	DATE	PLANS?	ACTIVITY	OUTFALL Y/N?	OUTFALL #
11	38	5 Summit Ave.	Res.	Determ. of App.	RD #03-09	12/12/2003	Y	Construct porch, stairs, ramp	N	
11	51	Twin Light Circle	Res.	Order of Conditions	DEP #62-184	7/26/1994	Y	Construct 2 SFH's	N	
11	51	Twin Light Circle	Res.	Order of Conditions	DEP #62-200	5/27/1995	Y	Construct roads and utilities for residential subdivision; includes stormwater drainage and outfall to streambank.	N	
11	51	? Twin Light Circle	Res.	Determ. of App.	RD #00-22	7/13/2000	Drawing	Regrade rear yard; construct fence	N	
11	51H	? Twin Light Circle	Res.	Order of Conditions	DEP #62-400	7/12/2004	Y	Construct addition to existing house	N	
11	51N	16 McKay's Dr.	Res.	Determ. of App.		6/21/1983	Drawing	Construct new SFH	N	
12	51N	17 McKay's Dr.	Res.	Determ. of App.	RD #94-11	12/8/1994	Drawing	Construct new 2-car garage	N	
18	**170**	**17 Railroad Ave. (Isinglass Place)**	**Com.**	**Order of Conditions**	**DEP #62-90**	**4/21/1989**	**Y**	**Addition to commercial bldg., repair drainage, install new drainage, rip-rap stream bank, construct retaining wall on bank**	**Y**	**25,28**
18	**183**	**16 Railroad Ave.**	**Res.**	**Order of Conditions**	**DEP # 62-88**	**9/18/1988**	**Drawing**	**Install 6" PVC "interceptor" drain**	**Y**	**13or14**
18	184	18 Railroad Ave.	Com.	Determ. of App.	RD #98-10	4/23/1998	Drawing	Remove trees threatening bldg. fndtn. within 100' of brook	N	
18	187A	Henderson Crt.	Res.	Determ. of App.	RD #00-12	10/4/2001	?	Install 6' x 8' garden shed in Riverfront Area	N	
18	187B	Henderson Crt.	Public	Determ. of App.	RD #00-15	5/17/2000	N	Town installing new water line and hydrant	N	
18	301	4 King St. Crt.	Res.	Order of Conditions	DEP #62-298	6/8/1999	Y	Connect existing dwelling to municipal sewer	N	
18	302	3 King St. Crt.	Res.	Order of Conditions	DEP #62-298	6/8/1999	Y	Connect existing dwellings to municipal sewer	N	
18	303	1 King St. Crt.	Res.	Determ. of App.		3/24/2003	Drawing	Construct attached deck	N	
18	321	7 King St.	Res.	Determ. of App.		1/3/1992	N	Construct deck addition	N	
18	322	5 King St.	Res.	Determ. of App.		6/14/2000	Building plan	Construct building addition	N	
18	324	Beach & King Sts.	Com.	Order of Conditions	DEP #62-397	3/12/2003	Y	Demolish greenhouse and construct 2-story rental unit	N	
				WPA Emergency Cert.		6/24/2004	N	Repair roof damaged by fire?	N	
18	325	Millbrook Meadow	Public	Determ. of App.		1987	Drawing	Remove wooden stairs; construct granite stairs	N	
				Order of Conditions	DEP #62-119	9/24/1991	Drawing	Redistribute fill within 100' of brook	N	
				Determ. of App.	RD #99-6	5/20/1999	N	book	N	
				Order of Conditions	DEP #62-382	6/27/2002	Y	Restoration of man-made pond; construct natural stone eel ramp; native plantings	N	

Bold typeface is used to highlight permits that include stormwater discharge outfalls.

Appendix C: Photos of Selected Erosion and Sedimentation Locations

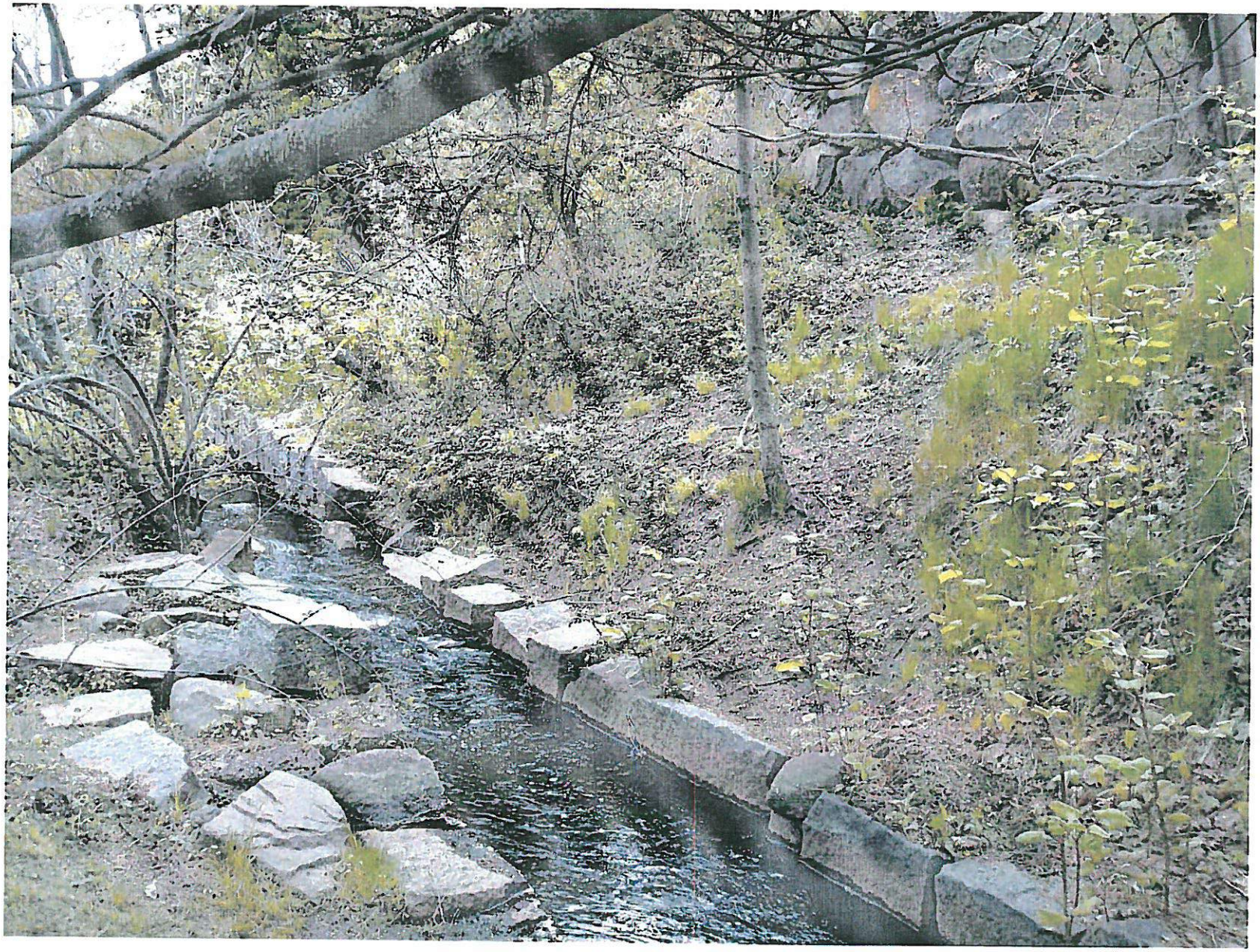


Figure C-1: Some erosion of bank behind King St. residence

Figure C-2: Collapsing retaining wall at base of Summit Ave.

Figure C-3: Accumulation of sand from play area behind Isinglass Place

Figure C-4: Eroding stream bank adjacent to Mill Brook Park

Appendix D: Beach Testing v. Rainfall 1999 – 2004

Figure D-1: Rainfall -v- Millbrook Fecal Counts
1999

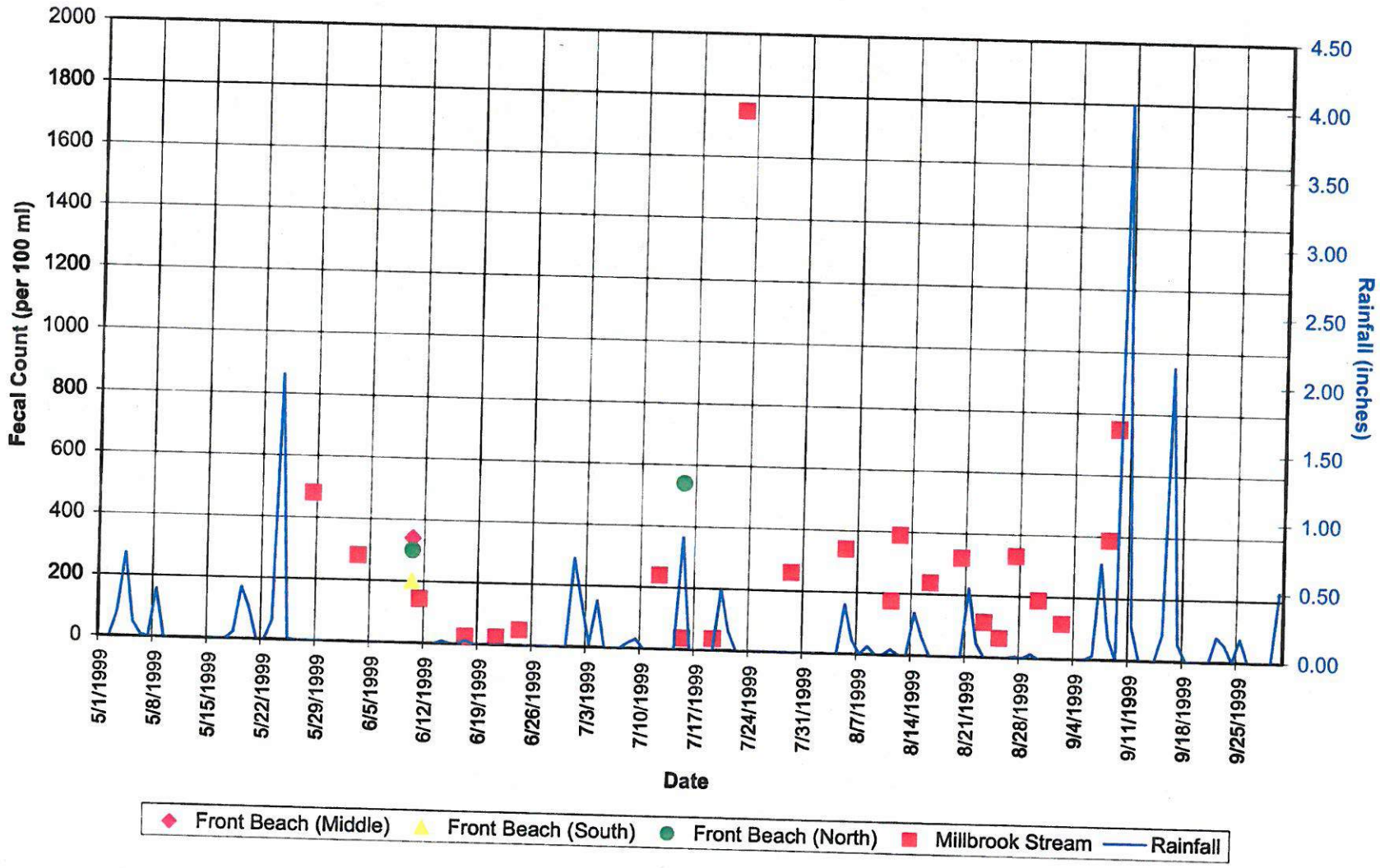


Figure D-2: Rainfall -v- Millbrook Fecal Counts
2000

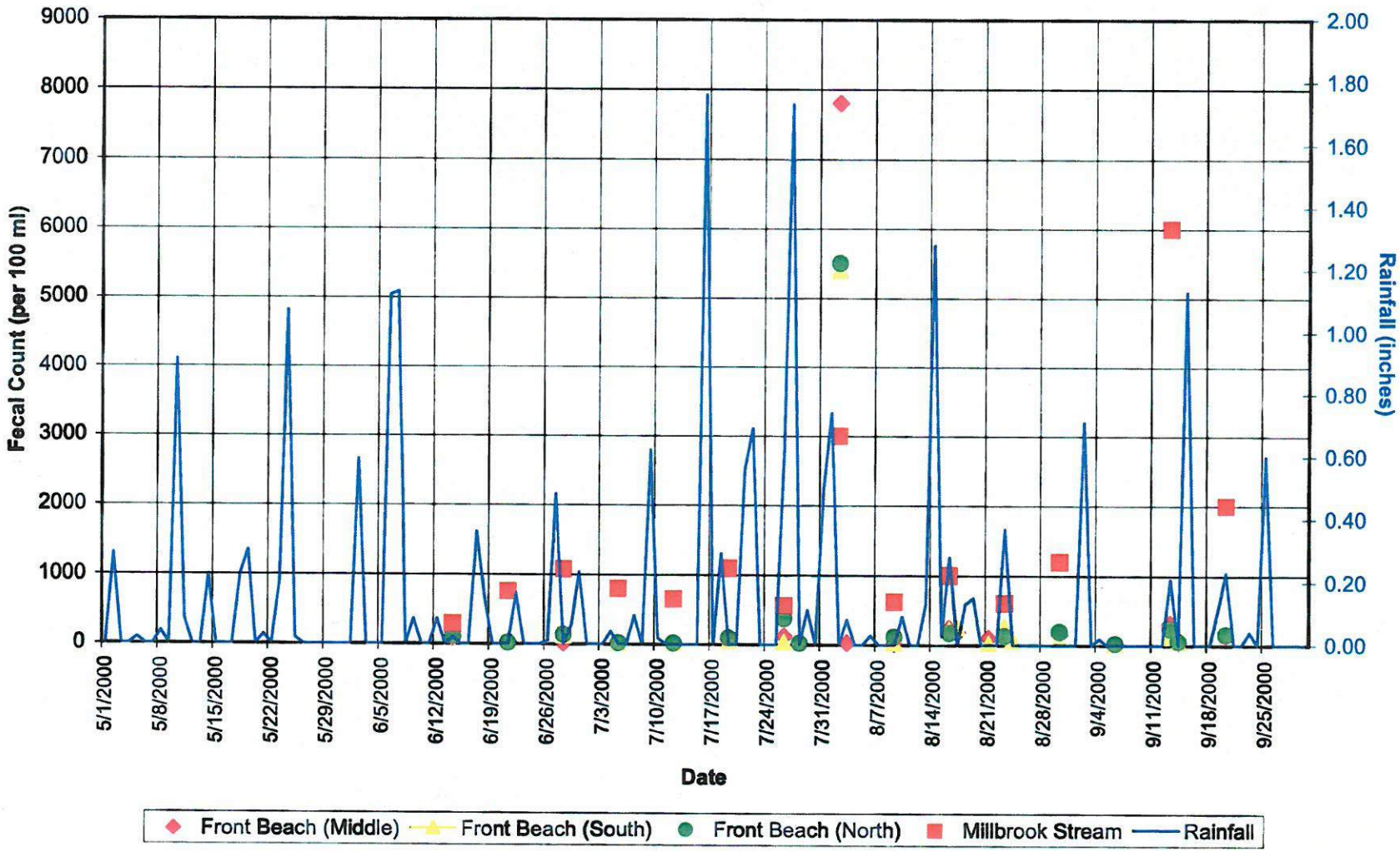


Figure D-3: Rainfall -v- Millbrook Bacteria Counts
2001

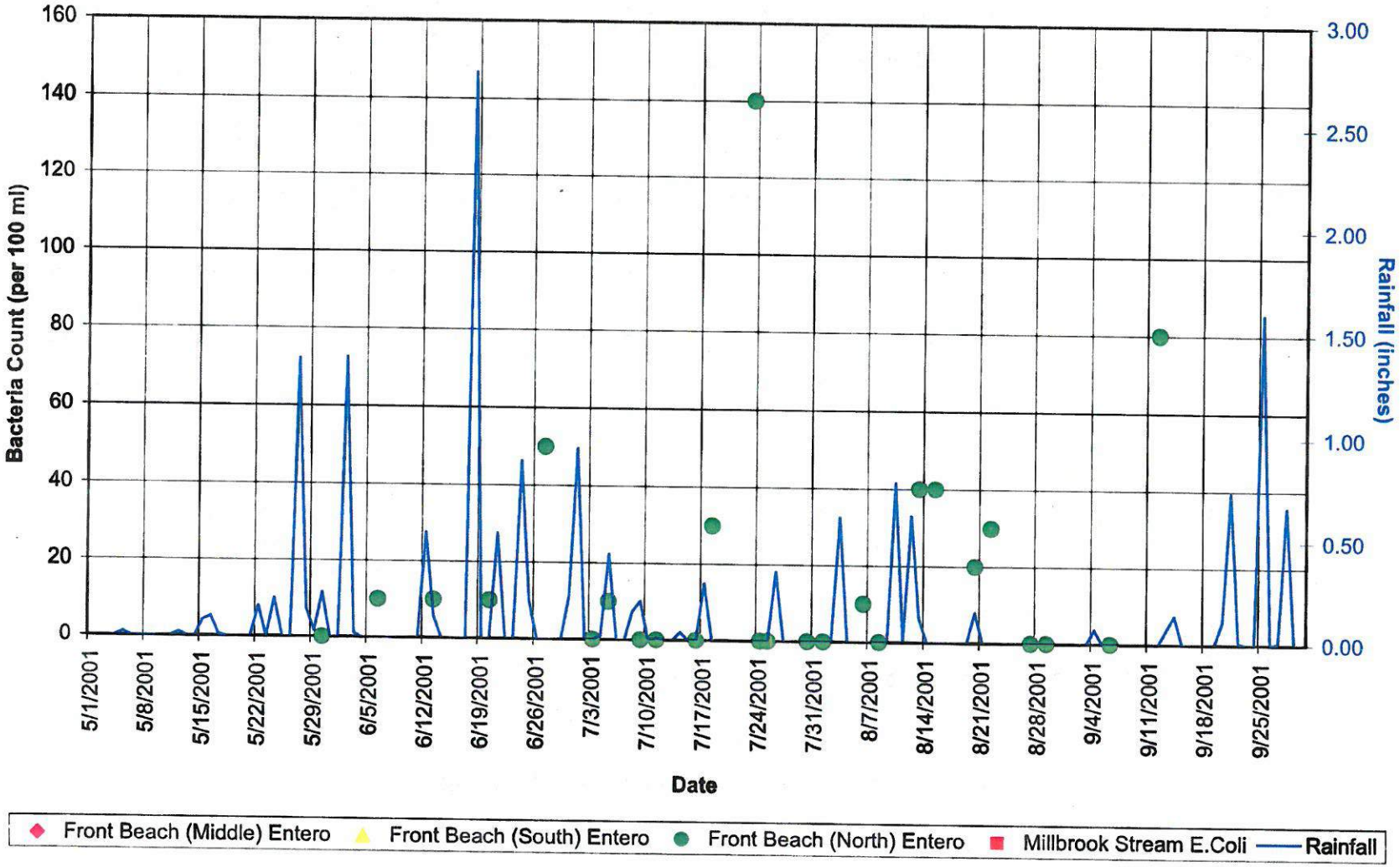


Figure D-4: Rainfall -v- Millbrook Bacteria Counts
2002

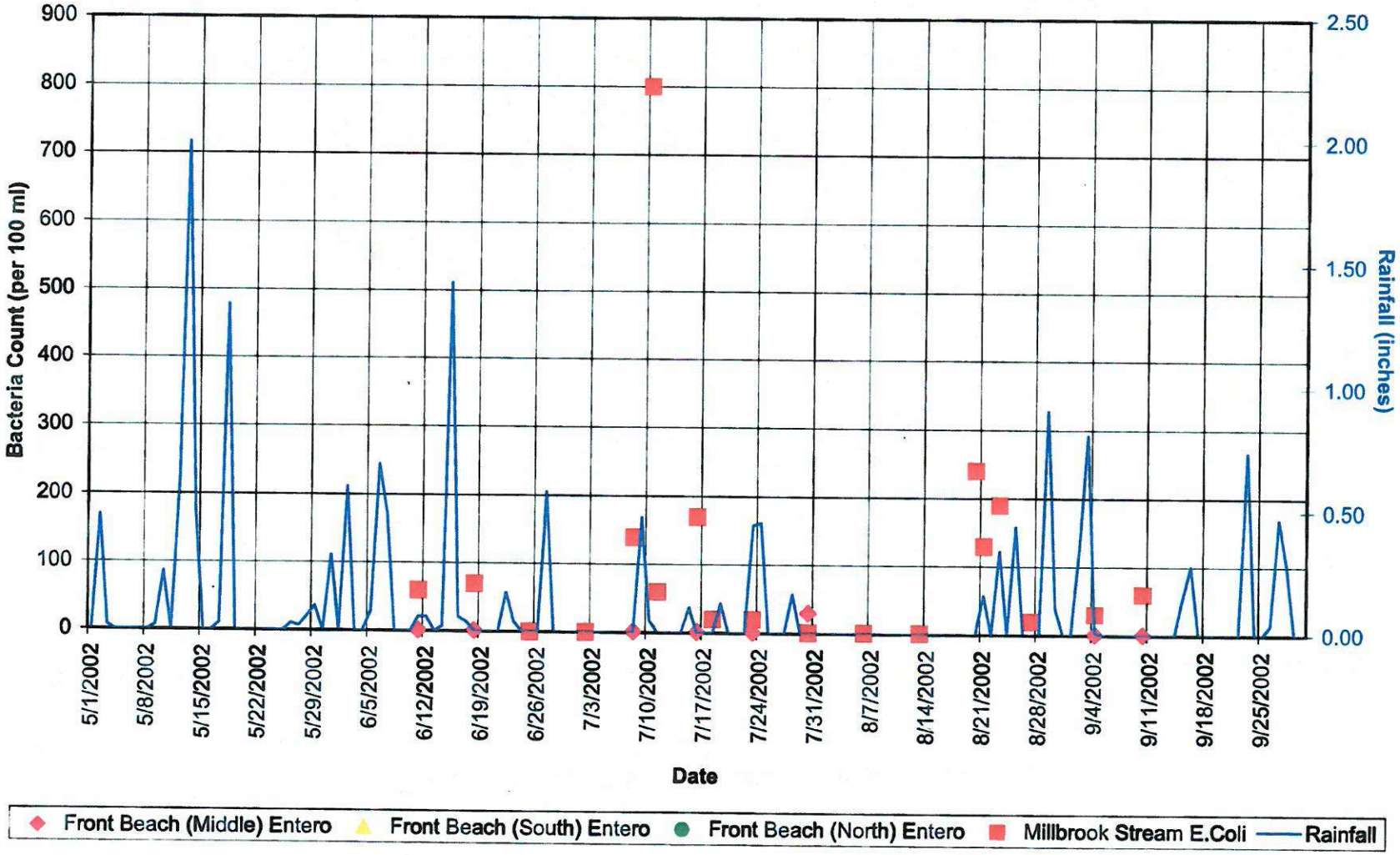


Figure D-5: Rainfall -v- Millbrook Bacteria Counts
2003

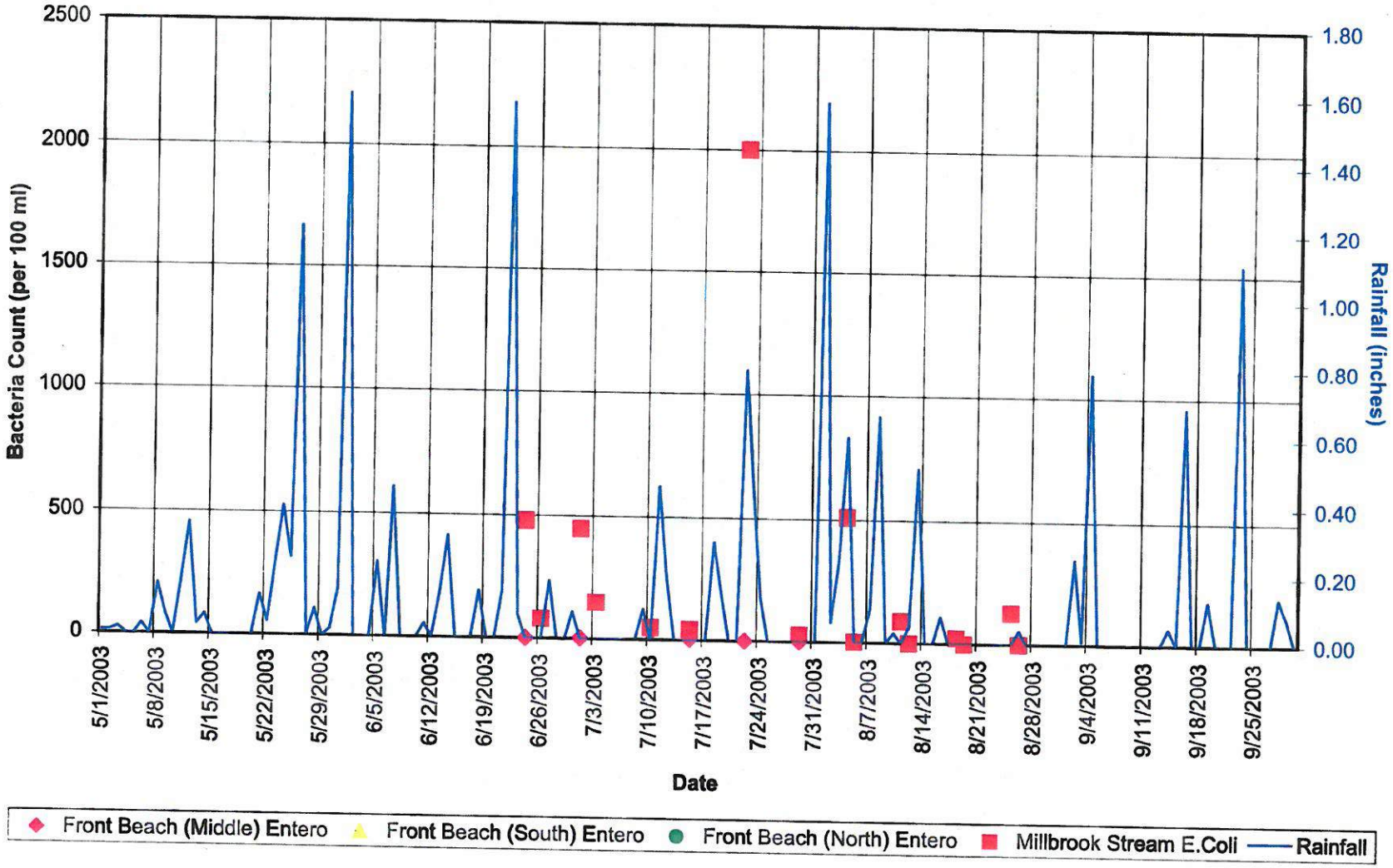
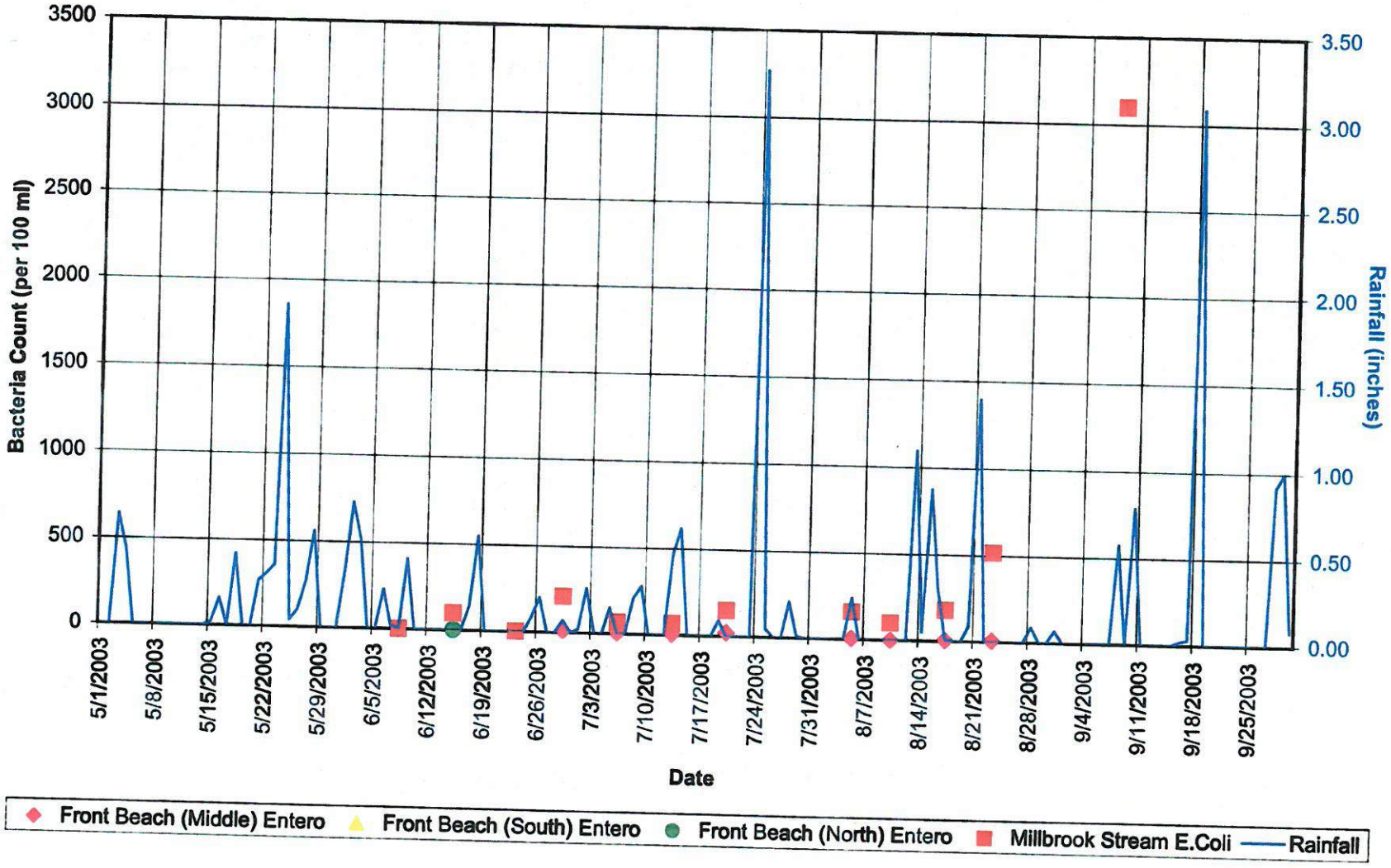


Figure D-5: Rainfall -v- Millbrook Bacteria Counts 2004



Appendix E: Water Quality Monitoring Lab Reports

Report No.: 31463
September 14, 2004

FINDINGS:

Biomarine ID	Rockport Site ID	Time Sampled	Total Coliform per 100 mL	Fecal Coliform per 100 mL	E. coli Per 100 mL	Enterococci per 100 mL	TSS
31463G	(A) Beach St downstream	1200	53,000	8,000	3,100	6,300	30
31463H	(B) Beach St upstream	1207	23,000	6,500	2,000	430	39
31463I	(C) Millbrook Pond downstream	1214	4,000	2,300	1,300	1,600	21
31463J	(D) Mill Pond upstream	1235	180,000	15,000	8,000	14,000	41
31463K	(E) Henderson Ct Millbrook	1300	66,000	27,000	9,000	14,000	24
31463L	(F) Henderson Ct Squam	1307	130,000	47,000	16,000	16,000	34
31463M	(G) Railroad Ave upstream	1310	68,000	18,000	8,600	12,000	19

METHODS: *Standard Methods for the Examination of Water and Wastewater, 20th Edition, 1998* and EPA Method 1600, March 2000.

Jim Gerrard/Assistant Lab Manager

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Rockport Department of Public Works
34 Broadway
Rockport, MA 01966

Report No.: 32026

RE: BACTERIOLOGICAL ANALYSES OF SURFACE WATERS
SAMPLING: Non-potable water samples delivered by Julie McMahon on November 24, 2004 at 9:25 p.m.
NOTE: TSS results will be available December 2, 2004.

FINDINGS:

Biomarine ID	Bottle ID	Rockport Site ID	Time Sampled	Total Coliform per 100 mL	Fecal Coliform per 100 mL	E. coli per 100 mL	Entercocci Per 100 mL	TSS
32026A	12	(A) Beach St downstream	0805	16,000	1,000	1,000	2,500	265
32026B	13	(B) Beach St upstream	0805	19,000	140	130	300	23
32026C	7	(C) Millbrook Pond downstream	0805	10,000	500	100	500	76
32026D	11	(D) Mill Pond upstream	0805	170,000	1,900	1,900	1,300	312
32026E	3	(E) Henderson Ct Millbrook	0805	110,000	1,200	800	1,300	134
32026F	5	(F) Henderson Ct/ Squam	0805	32,000	1,900	1,700	300	97
32026G	10	(G) Railroad Ave upstream	0805	68,000	1,600	1,600	1,000	117

Report No.: 32026
November 29, 2004

FINDINGS:

Biomarine ID	Bottle ID	Rockport Site ID	Time Sampled	Total Coliform per 100 mL	Fecal Coliform per 100 mL	E. coli Per 100 mL	Enterococci per 100 mL	TSS
32026H	15	(H) Railroad Ave 200m	0805	48,000	1,100	1,000	200	105
32026I	4	(I) Poole Ln downstream	0805	34,000	1,200	500	900	99
32026J	2	(J) Poole Ln upstream	0805	38,000	1,400	1,100	1,300	92
32026K	8	(K) Squam Brook Summit Down	0805	18,000	500	400	200	192
32026L	1	(L) Squam Brook Squam Hill Ct	0805	2,400	50	<10	20	9
32026M	14	(A) Dup	0805	49,000	3,000	1,600	3,400	261
32026N	9	(E) Dup	0805	190,000	1,200	1,100	400	131

METHODS: *Standard Methods for the Examination of Water and Wastewater, 20th Edition, 1998* and EPA Method 1600, March 2000.

Jim Sarcione/Assistant Lab Manager

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Rockport Department of Public Works
34 Broadway
Rockport, MA 01966

May 12, 2005
Report No.: 32950

RE: ANALYSES OF SURFACE WATERS

SAMPLING: Non-potable water samples delivered by Julie McMahon on May 7, 2005 at 7:45 a.m.

FINDINGS:

Biomarine ID	Bottle ID	Rockport Site ID	Time Sampled	Total Coliform per 100 mL	Fecal Coliform per 100 mL	E. coli per 100 mL	Entercocci Per 100 mL	TSS
32950A	13	(H) Railroad Ave 200m	0729	4,300	570	380	110	7
32950B	9	(I) Poole Ln Downstream	0741	6,100	470	410	70	9
32950C	10	(J) Poole Ln Upstream	0743	2,700	520	450	<10	6
32950D	11	(K) Squam Brook Summit Down	0732	4,000	140	50	80	20
32950E	14	(L) Squam Brook Squam Hill Ct	0738	3,500	90	40	50	16.5
32950F	2	(A) Dup	N/A	1,500	260	220	60	3
32950G	5	(F) Dup	N/A	4,300	120	50	60	21

Report No.: 32950
May 12, 2005

FINDINGS:

Biomarine ID	Bottle ID	Rockport Site ID	Time Sampled	Total Coliform per 100 mL	Fecal Coliform per 100 mL	E. coli Per 100 mL	Enterococci per 100 mL	TSS
32950H	1	(A) Beach St Downstream	0700	3,800	270	220	160	4
32950I	4	(B) Beach St Upstream	0700	3,700	270	180	110	8
32950J	3	(C) Millbrook Pond Downstream	0705	2,300	270	270	50	4
32950K	8	(D) Mill Pond Upstream	0715	4,000	700	670	50	9
32950L	7	(E) Henderson Ct Millbrook	0719	7,400	750	540	150	15
32950M	6	(F) Henderson Ct Squam	0723	4,300	70	70	80	23
32950N	15	(G) Railroad Ave Upstream	0725	3,800	660	640	100	30

METHODS: *Standard Methods for the Examination of Water and Wastewater, 20th Edition, 1998* and EPA Method 1600, March 2000.

John Gaccione/Assistant Lab Manager

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Appendix F: Water Quality Monitoring – Results and Comparison

Table F-1

Lower Mill Brook Wet Weather Water Quality Sampling

Sampling Station	8-Sep-04					24-Nov-04					7-May-05				
	Total Coliform	Fecal Coliform	E. coli	Entero	TSS	Total Coliform	Fecal Coliform	E. coli	Entero	TSS	Total Coliform	Fecal Coliform	E. coli	Entero	TSS
(A) Beach St downstream	53,000	8,000	3,100	6,300	30	16,000	1,000	1,000	2,500	265	3,800	270	220	160	4
(A) DUP*	-	-	-	-	-	49,000	2,900	1,600	3,400	261	1,500	260	220	60	3
(B) Beach St upstream	23,000	6,500	2,000	430	39	19,000	140	130	300	23	3,700	270	180	110	8
(C) Millbrook Pond downstream	4,000	2,300	1,300	1,600	21	10,000	500	100	500	76	2,300	270	270	50	4
(D) Mill Pond upstream	180,000	15,000	8,000	14,000	41	170,000	1,900	1,900	1,300	312	4,000	700	670	50	9
D DUP*	170,000	17,000	13,000	10,900	36	-	-	-	-	-	-	-	-	-	-
(E) Henderson Ct Millbrook	66,000	27,000	9,000	14,000	24	110,000	1,200	800	1,300	134	7,400	750	540	150	15
(E) DUP*	-	-	-	-	-	190,000	1,200	1,100	400	131	-	-	-	-	-
(F) Henderson Ct Squam	130,000	47,000	16,000	16,000	34	32,000	1,900	1,700	300	97	4,300	70	70	80	23
(F) DUP*	-	-	-	-	-	-	-	-	-	-	4,300	120	50	60	21
(G) Railroad Ave upstream	68,000	18,000	8,600	12,000	19	68,000	1,600	1,600	1,000	117	3,800	660	640	100	30
(H) Railroad Ave 200m	70,000	11,000	11,000	10,000	25	48,000	1,100	1,000	200	105	4,300	570	380	110	7
(I) Poole Ln downstream	100,000	12,000	12,000	12,000	36	34,000	1,200	500	900	99	6,100	470	410	70	9
(J) Poole Ln (upstream)	100,000	14,000	12,000	12,000	41	38,000	1,400	1,100	1,300	92	2,700	520	450	10	6
(K) Squam Brook Summit down	130,000	13,000	11,000	10,000	497	18,000	500	400	200	192	4,000	140	50	80	20
(L) Squam Brook Squam Hill Ct.	130,000	7,300	3,300	2,400	137	2,400	50	10	20	9	3,500	90	40	50	16.5

* DUP = a duplicate sample taken at the same station for quality assurance of sample collection and analysis techniques.

Table F-2

Lower Mill Brook Optical Brightener Testing -- 9/1/2004 - 9/8/2004

Rockport Site ID	Date Deployed	Date Retrieved	Reading
(B) Beach St--Upstream	9/1/2005	9/8/2005	Negative
(C) Millbrook Pond--Downstream	9/1/2005	9/10/2005*	Negative
(G) Railroad Ave--Upstream	9/1/2005	9/8/2005	"Possible" Positive
(J) Poole Ln--Upstream	9/1/2005	9/8/2005	Negative
(K) Squam Brook--Summit Ave Downstream	9/1/2005	9/8/2005	Negative
(L) Squam Brook--Squam Hill Crt	9/1/2005	9/8/2005	Negative

* This sample was retrieved later than others due to too high flows at this location on 9/8/2005.

Figure F-1: Lower Mill Brook Wet Weather Sampling
Total Coliform

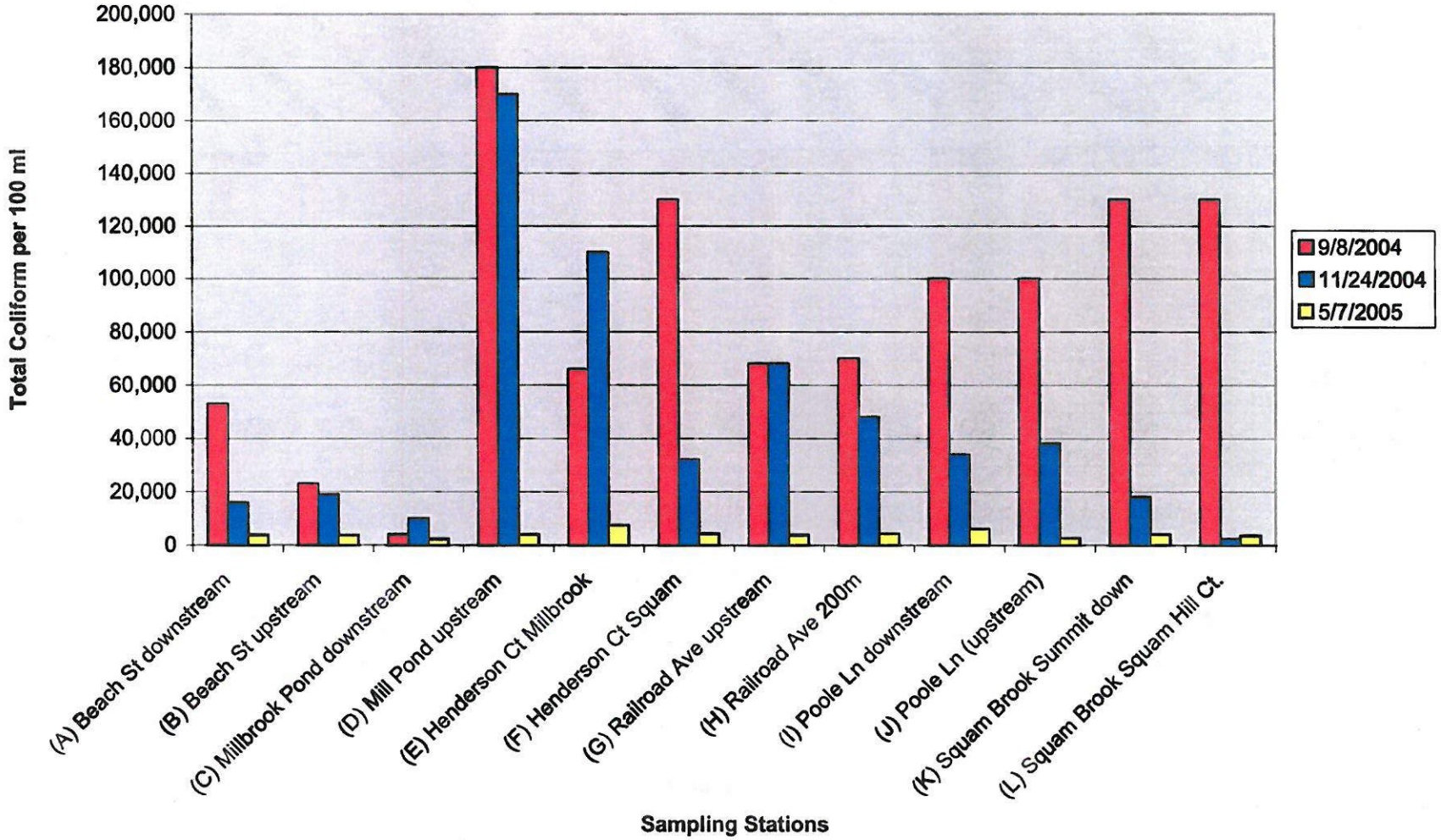


Figure F-2: Lower Mill Brook Wet Weather Sampling
Fecal Coliform

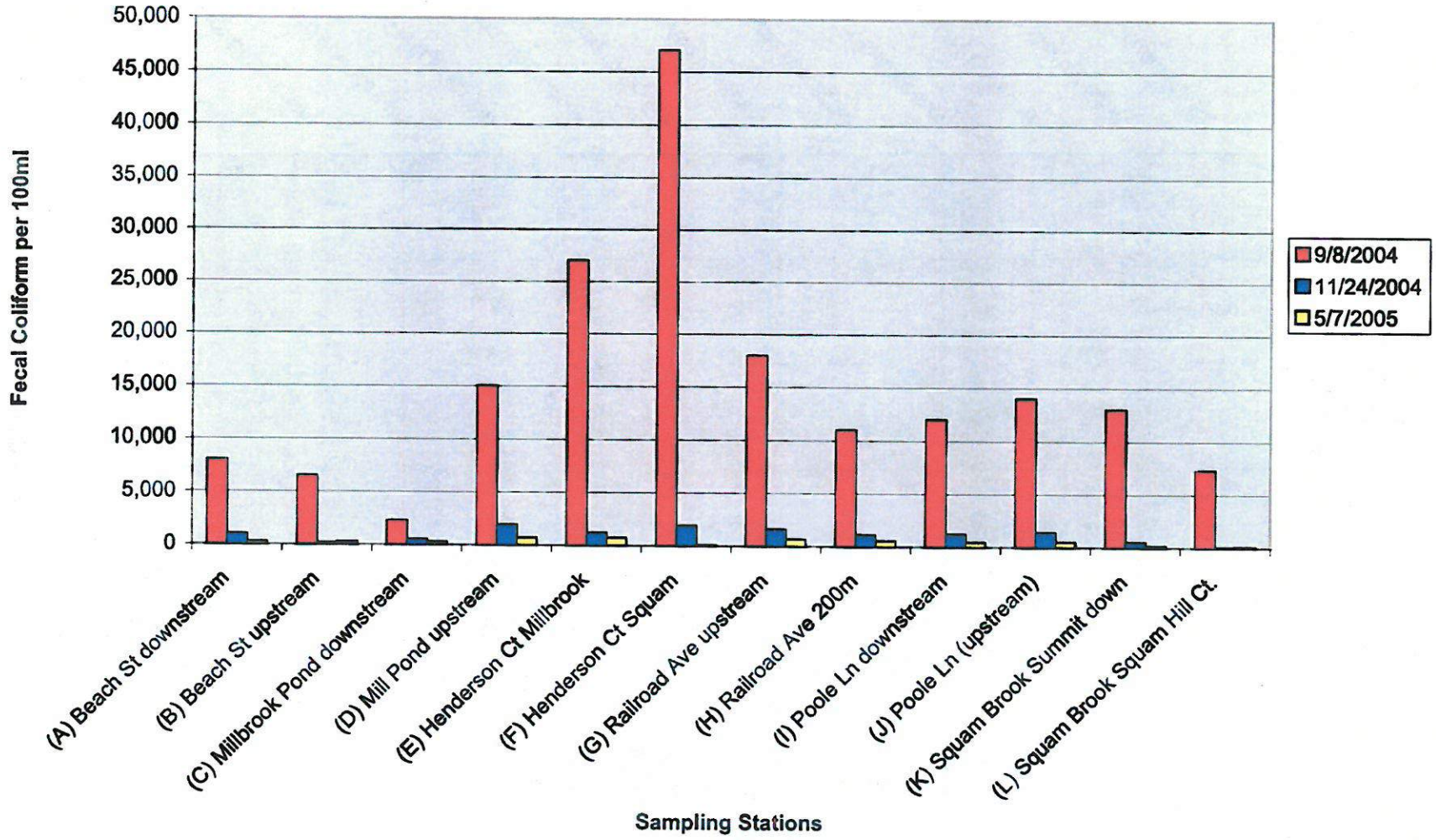


Figure F-3: Lower Mill Brook Wet Weather Sampling
E. coli

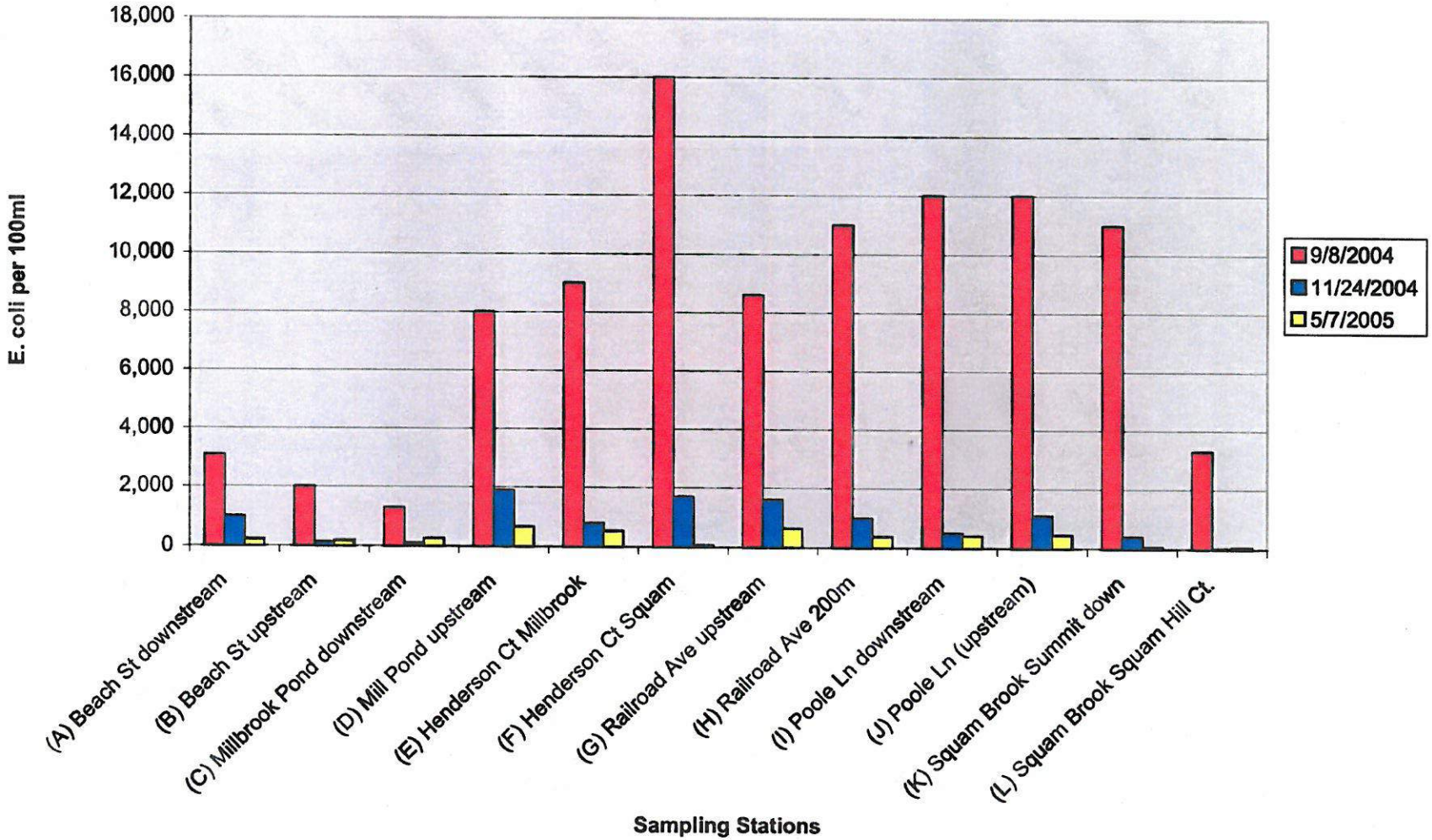


Figure F-4: Lower Mill Brook Wet Weather Sampling
Enterococci

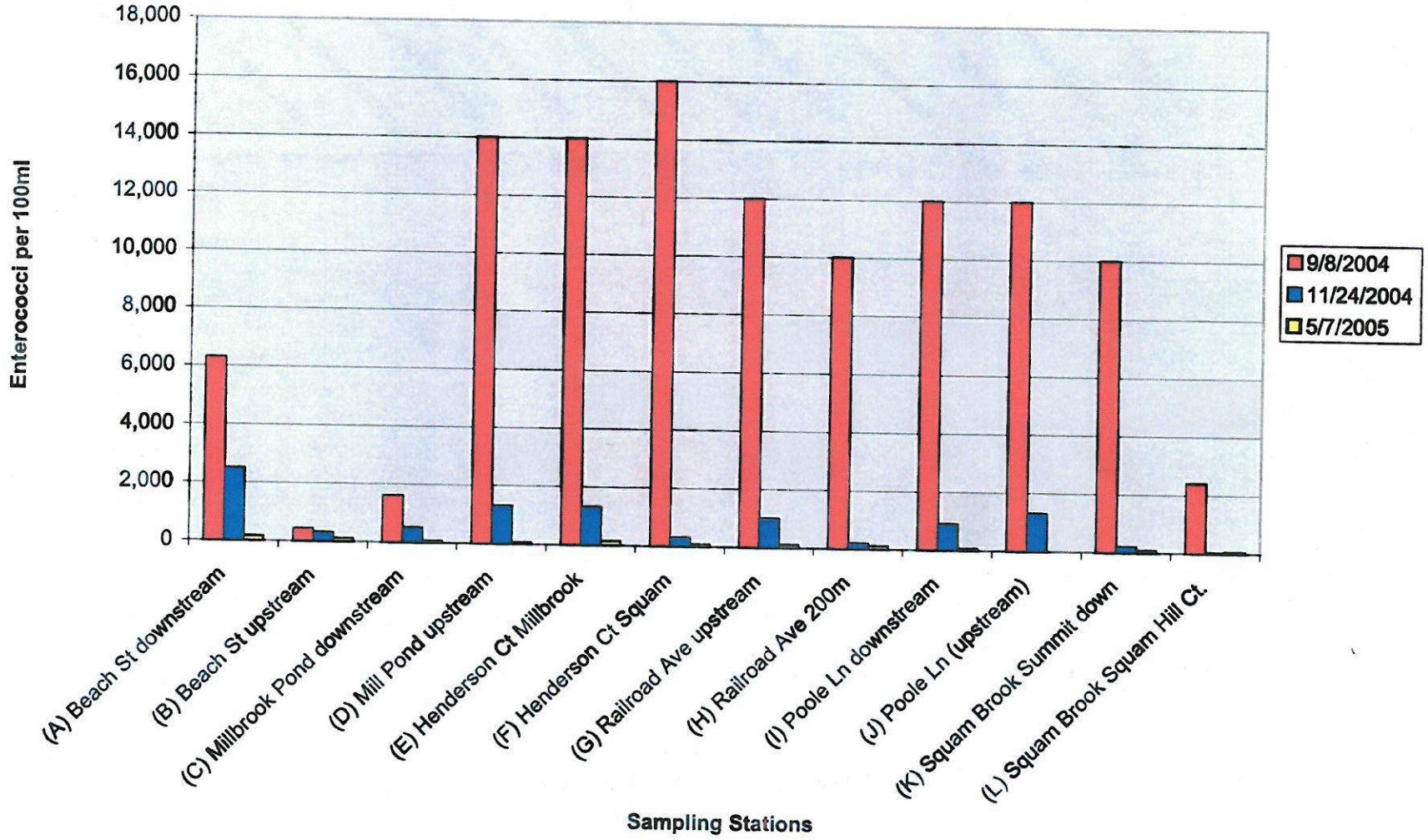


Figure F-5: Lower Mill Brook Wet Weather Sampling
Total Suspended Solids (TSS)

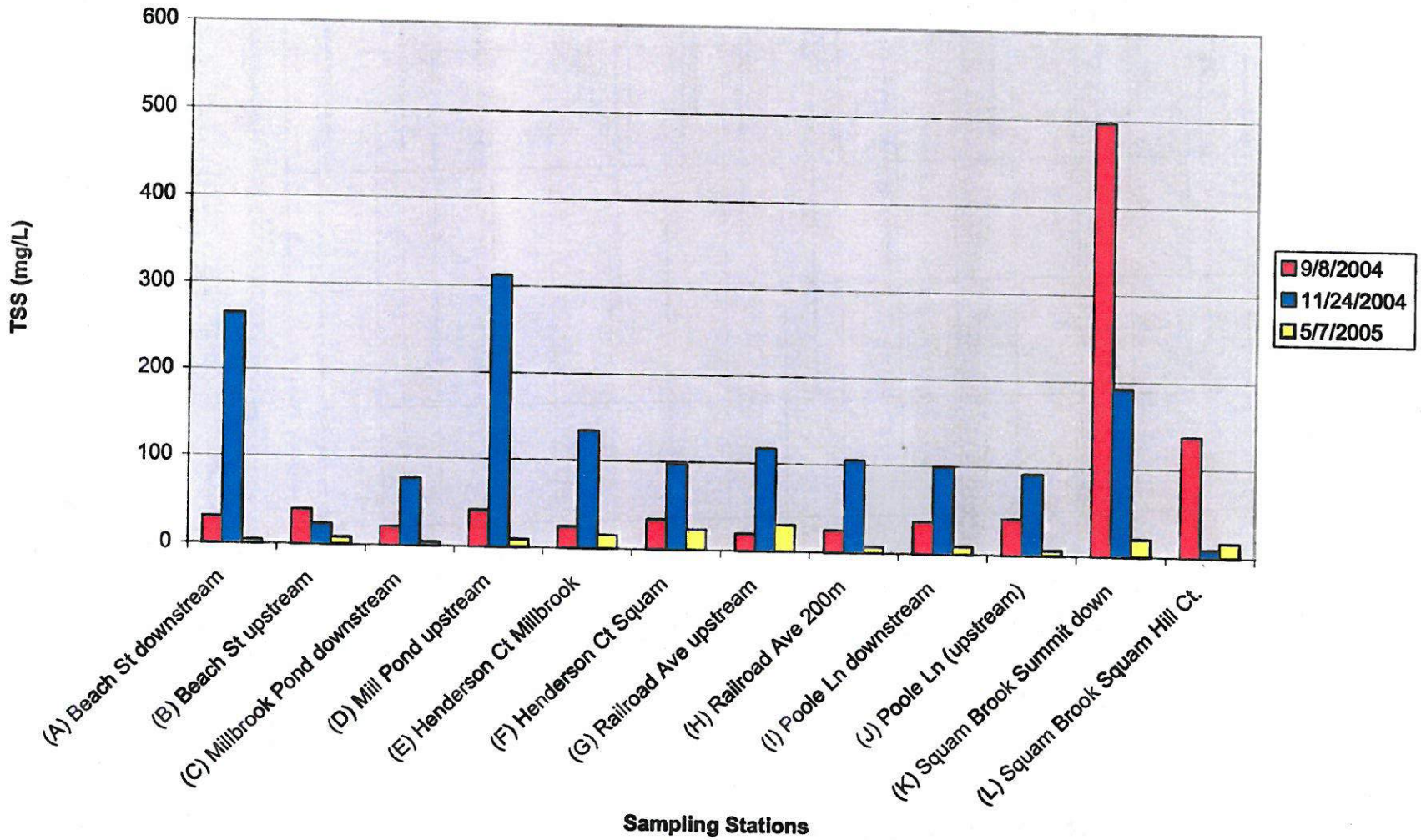


Figure F-6: E Coli -v- Rainfall (Wet Weather Sampling)

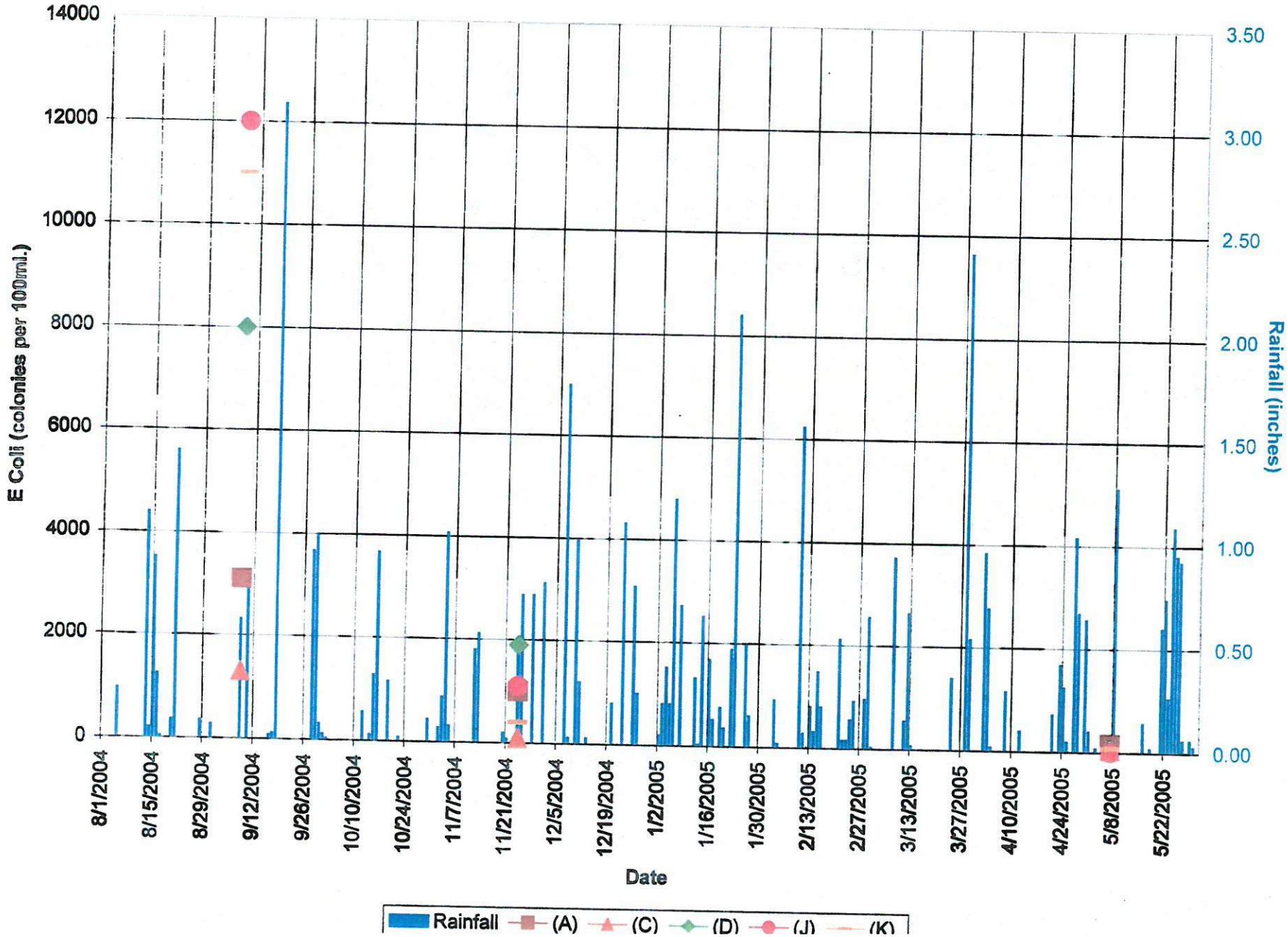
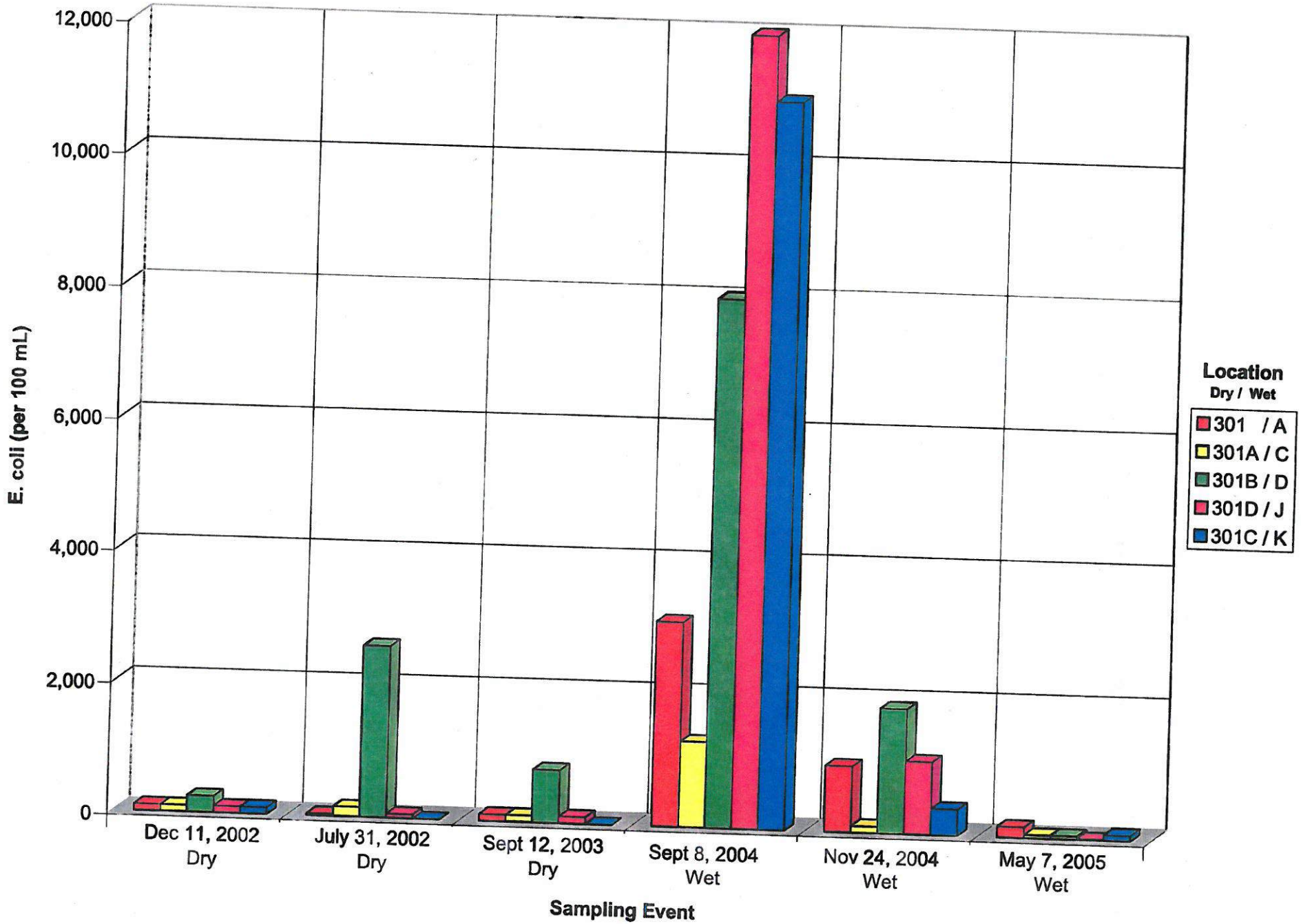


Figure F-7: Millbrook Sampling (E. coli)



Appendix G: Catch Basin Data

Appendix G: Catch Basin Cleaning Data

Catch Basin #	Location	Type	Depth (ft.)	Diameter (ft.)	No. Inlets	Volume (cu ft.)	October-04 Depth (ft.)	October-04 lume removed (cu.	October-04 % Full	May-05 Depth (ft.)	May-05 me removed (cu	May-05 % Full	Rate of Sediment buildup Oct. '04 - May, '05 (cu. ft. / day)
1	Beach St. (near restrooms)	Closed	5.50	4	1	69.12	1.00	12.57	18.18	1.00	12.57	18.18	0.06
2	Beach St. (near Mill Brook)	Open	2.00	4	0	25.13	0.80	10.05	40.00	0.50	6.28	25.00	0.03
3	Beach St.(near Mill Brook)	Open	4.00	4	2	50.27	0.50	6.28	12.50	1.00	12.57	25.00	0.06
4	Beach St.	Direct Discharge	6.00	4	0	75.40	NA		NA	NA		NA	NA
5	Beach St. x King St.	Closed	6.00	4	1	75.40	2.00	25.13	33.33	1.00	12.57	16.67	0.06
6	King St.	Closed	6.00	4	1	75.40	0.50	6.28	8.33	1.00	12.57	16.67	0.06
7	King St. (100' from Beach St.)	Closed	5.50	4	1	69.12	1.00	12.57	18.18	1.00	12.57	18.18	0.06
8	King St.	Closed	7.00	4	1	87.96	1.00	12.57	14.29	0.50	6.28	7.14	0.03
9	King St.	Closed	6.50	4	1	81.68	1.00	12.57	15.38	0.50	6.28	7.69	0.03
10	King St. x Forrest St.	Closed	6.50	4	1	81.68	0.50	6.28	7.69	0.00	0.00	0.00	0.00
11	King St. x Forrest St.	Closed	5.50	4	1	69.12	1.00	12.57	18.18	0.50	6.28	9.09	0.03
12	King St.	Closed	5.50	4	1	69.12	NA		NA	2.00	25.13	36.36	NA
13	King St	Closed	7.00	4	1	87.96	2.00	25.13	28.57	1.50	18.85	21.43	0.09
14	King St. x Holbrook Crt. Exit	Closed	6.00	4	2	75.40	2.50	31.42	41.67	1.50	18.85	25.00	0.09
15	King St x Holbrook Crt. Exit	Closed	6.00	4	1	75.40	1.50	18.85	25.00	1.00	12.57	16.67	0.06
16	Holbrook Crt	Closed	4.00	4	1	50.27	2.00	25.13	50.00	0.50	6.28	12.50	0.03
17	Holbrook Crt.	Open	4.00	4	0	50.27	0.50	6.28	12.50	0.50	6.28	12.50	0.03
18	Holbrook Crt.	Open	5.00	4	2	62.83	0.50	6.28	10.00	0.50	6.28	10.00	0.03
19	Holbrook Crt.	Open	4.00	4	0	50.27	0.50	6.28	12.50	0.50	6.28	12.50	0.03
20	Holbrook Crt.	Open	6.50	4	3	81.68	0.00	0.00	0.00	0.50	6.28	7.69	0.03
21	Holbrook Crt.	Open	4.50	4	0	56.55	0.30	3.77	6.67	0.00	0.00	0.00	0.00
22	Poole's Ln. (near IGA)	Closed	5.50	4	1	69.12	0.00	0.00	0.00	0.50	6.28	9.09	0.03
23	Poole's Ln. (near IGA)	Closed	4.00	4	1	50.27	0.00	0.00	0.00	1.00	12.57	25.00	0.06
24	Poole's Ln. (over brook)	Direct Discharge	4.00	4	3	50.27	NA		NA	NA		NA	NA
25	Poole's Ln. (over brook)	Direct Discharge	4.00	4	2	50.27	NA		NA	NA		NA	NA
26	Poole's Ln.	Closed	4.50	4	0	56.55	1.00	12.57	22.22	0.50	6.28	11.11	0.03
27	Poole's Ln.	Open	6.00	4	1	75.40	1.00	12.57	16.67	0.50	6.28	8.33	0.03
28	Poole's Ln. (across from Housing	Open	6.00	4	1	75.40	2.20	27.65	36.67	1.00	12.57	16.67	0.06
29	Poole's Ln. (across from Housing	Open	4.50	4	0	56.55	1.00	12.57	22.22	1.00	12.57	22.22	0.06
30	Maine St. x Railroad Ave.	Closed	4.00	4	0	50.27	1.00	12.57	25.00	1.00	12.57	25.00	0.06
31	Railroad Ave.	Open	5.00	4	0	62.83	0.50	6.28	10.00	1.00	12.57	20.00	0.06
32	Railroad Ave.	Closed	5.00	4	1	62.83	1.00	12.57	20.00	0.50	6.28	10.00	0.03
33	Railroad Ave. (over brook)	Direct Discharge	5.00	4	1	62.83	NA		NA	NA		NA	NA
34	Railroad Ave. (over brook)	Direct Discharge	5.00	4	1	62.83	NA		NA	NA		NA	NA
35	Railroad Ave. (near Dunkin Donut	Open	4.00	4	3	50.27	0.00	0.00	0.00	0.50	6.28	12.50	0.03
36	Railroad Ave.	Direct Discharge	5.20	4	1	65.35	1.20	15.08	23.08	0.00	0.00	0.00	0.00
37	Railroad Ave.	Open	3.00	4	1	37.70	0.00	0.00	0.00	0.00	0.00	0.00	0.00
38	Mill Lane	Closed	4.00	2	1	12.57	0.50	1.57	12.50	0.50	1.57	12.50	0.01
39	Mill Lane	Open	5.70	4	1	71.63	0.20	2.51	3.51	0.50	6.28	8.77	0.03
40	Mill Lane	Open	5.50	4	1	69.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00
41	Millbrook Meadow	Closed	NA	NA	NA	NA	NA		NA	NA		NA	NA
42	McKays Drive	Closed	5.00	4	0	62.83	0.50	6.28	10.00	0.50	6.28	10.00	0.03
43	McKays Drive	Closed	5.00	4	0	62.83	0.50	6.28	10.00	0.50	6.28	10.00	0.03
44	Railroad Station Parking Lot	NA	NA	NA	NA	NA	NA		NA	NA		NA	NA
45	Base of Summit Avenue	Direct Discharge	NA	NA	NA	NA	NA		NA	NA		NA	NA
	Total Volume Removed (cubic feet)							368.51			309.45		
	Total Volume Removed (cubic yards)							13.65			11.46		

Bold typeface used to highlight individual catch basins with the highest volume of sediments, highest percent full, or highest accumulation rates.

Appendix H: Catch Basin Cleaning Plan

TOWN OF ROCKPORT

DEPARTMENT OF PUBLIC WORKS

CATCH BASIN CLEANING PLAN

FOR THE MILL BROOK WATERSHED AND OTHER CRITICAL WATERSHEDS THAT AFFECT VALUABLE WATER SUPPLY, FRESHWATER AND COASTAL WETLAND RESOURCES

Prepared as part of the
Lower Mill Brook Coastal Nonpoint Source Pollution Assessment

June 2005

Town of Rockport
Catch Basin Cleaning Plan

Purpose of the Plan

It is known that the efficiency and effectiveness of sediment removal by catch basin systems declines as their capacity is reduced by sediment accumulation. One study (Pitt, 1985) concluded that catch basins can capture sediments up to approximately 60% of the sump volume. When sediment fills greater than 60% of their volume, catch basins reach steady state. At that point, catch basins can lose as much sediment to re-suspension and washout as they capture. Storm flows may then bypass treatment as well as re-suspend sediments trapped in the catch basin. Frequent clean-out can therefore retain the volume in the catch basin sump available for treatment of stormwater flows.

The purpose of this Catch Basin Cleaning Plan is to act as guidance for the town of Rockport Dept. of Public Works in its maintenance of the town's municipal catch basin system. Use of this guidance will maximize the efficiency of sediment and pollutant removal by the municipal catch basin system in an efficient and cost-effective manner for the DPW and its staff.

Reasons for Cleaning Catch Basins

There are many reasons for cleaning municipal catch basins, including:

- To prevent debris from blocking or impeding flow of drainage waters
- To keep drainage water from backing up onto streets and parking areas
- To maximize the efficiency of sediment collection and keep sediment from being sent downstream into wetland and coastal resource areas
- To prevent sediments from being re-suspended during larger precipitation events and washed downstream into wetland and coastal resource areas
- To remove pollutants from the drainage system
- To remove organic matter that can create breeding habitat for mosquitoes that spread harmful diseases such as West Nile virus
- To allow for periodic inspection for structural deficiencies in catch basin and the pipes that run into and out of them.

Catch Basin Cleaning Schedule

Typical maintenance of catch basins includes trash removal if a screen or other debris capturing device is used, and removal of sediment using a vactor truck. Operators need to be properly trained in catch basin maintenance.

Maintenance should include keeping a log of the amount and type of sediment collected and the date of removal. A template reporting form is attached to this Catch Basin Cleaning Plan that was developed and used during the Lower Mill Brook Coastal Nonpoint Source Pollution Assessment in 2005.

As reported above, at least one study (Pitt, 1985) concluded that catch basins can capture sediments up to approximately 60% of the sump volume. ***When sediment fills greater than 60% of their volume, catch basins no longer reliably capture sediment from storm flows.*** They can lose as much sediment to re-suspension and washout as they capture. Storm flows may then bypass treatment as well as re-suspend sediments trapped in the catch basin. Frequent clean-outs can help retain adequate volume in the catch basin sump for treatment of stormwater flows.

At a minimum, catch basins should be cleaned once or twice per year. MA DEP's Stormwater Policy dictates that inlets should be cleaned a minimum of 4 times a year and inspected monthly. USDA Natural Resource Conservation Service (NRCS) recommends cleanings up to fives times a year, citing a Wisconsin study that demonstrated that five cleanings removed a much greater amount of sediment from the system than fewer cleanings do (Marc MacQueen, Pers. Com.). Studies have suggested that increasing the frequency of maintenance can improve the performance of catch basins, particularly in industrial or commercial areas. One study of sixty catch basins in Alameda County, California, found that increasing the maintenance frequency from once per year to twice per year could increase the total sediment removed by catch basins by 30 percent on an annual basis. Increasing the maintenance frequency from once per year to monthly (twelve times per year), could increase the total sediment removed by over 200 percent on an annual basis. (Mineart and Singh, 1994). Annual sediment removed per inlet was 54 pounds for annual cleaning, 70 pounds for semi-annual and quarterly cleaning, and 160 pounds for monthly cleaning.

Monthly cleanings will maximize the annual sediment volume removed. For catch basins draining industrial uses, monthly cleaning increased total annual sediment collected to six times the amount collected by annual cleaning (180 lbs. versus 30 lbs.). These results suggest that, at least for industrial uses, more frequent cleaning of catch basins may improve removal efficiency. While Rockport is mostly residential, industrial areas like the commuter railroad station and commercial areas like the Whistlestop Mall, Isinglass Place and other facilities with large amounts of impervious surfaces can produce large quantities of runoff similar to a more-industrial situation. While these studies were conducted in more industrial settings, it follows that a similar pattern would be found in residential areas as well.

In particular, those catch basins should be cleaned more frequently that:

- ***Have shallower sumps (less than four-foot depth)***
- ***Receive higher levels of runoff from a larger drainage area or an area with higher levels of impervious surface***
- ***Receive higher levels of sedimentation from high levels of road sanding or from unvegetated, unstabilized land uses such as "soft" shouldered roads with no curbing or unpaved roads or parking areas or active or unstabilized construction sites.***
- ***Tend to reach the 60% of capacity level more quickly***
- ***Release their flow to more critical areas such as outstanding resource waters as defined by Mass. DEP, coastal streams or wetlands or are within a watershed of drinking water supply or coastal waters.***

More frequent street sweeping may reduce the number of annual catch basin cleanings needed to maintain optimal catch basin treatment capacity

Handling and Disposal

Catch basin cleanings – solid materials such as leaves, sand and twigs removed from storm water collection systems during cleaning operations -- are typically classified as a solid waste by the MA DEP. Catch basin cleanings must be handled and disposed in accordance with the agency's applicable regulations, policies and guidance for solid waste.

Except as explained below, catch basin cleanings from storm water-only drainage systems may be disposed at any landfill that is permitted by MA DEP to accept solid waste. DEP does not routinely require storm water only catch basin cleanings to be tested before disposal, unless there is evidence that they have been contaminated by a spill or some other means. Contaminated catch basin cleanings must be evaluated in accordance with the Hazardous Waste Regulations, 310 CMR 30.000, and handled as Hazardous Waste. Use or disposal of catch basin cleanings in any other way than in a permitted solid waste landfill requires prior approval from MA DEP under the Beneficial Use provisions of the *Solid Waste Management Facility Regulations* at 310 CMR 19.060.

Reporting Sheet

The attached Reporting Sheet template should be used and filled out whenever municipal catch basin systems are cleaned out. ***Filling out one Reporting Sheet per catch basin, including an identifying number or location for each catch basin and information on the date of cleaning, depth of material and***

an estimate of the amount of material removed will allow the town to keep track of which basins fill more quickly and which may need to be cleaned more often. This will allow for more efficiency, so that specific basins might be able to be targeted for a more intensive cleaning schedule (every month), while others may operate adequately on a less intensive cleaning schedule (once or twice annually). Recording diameter, depth, and type (open or closed bottom, etc.) and mapping of the inlets and outlets of each catch basin should be done once and then saved on the reporting sheet and used to verify the identify of each catch basin for future cleanings.

Some cities and towns have incorporated the use of GIS (Geographic Information System) systems to track sediment collection, and to optimize future catch basin cleaning efforts. At some point in the future if Rockport adopts a GIS-based mapping system, the information collected on the Reporting Sheets may be able to be incorporated into a town-wide GIS system to track sediment collection on a town-wide basis.

References:

Massachusetts Dept. of Environmental Protection (MA DEP), "Fact Sheet: Management of Catch Basin Cleanings", April 2004

Massachusetts Dept. of Environmental Protection (MA DEP), Stormwater Policy Handbook, Vol. 1: Stormwater Management, 1997

Mineart, P. and S. Singh. 1994. *Storm Inlet Pilot Study*. Woodward-Clyde Consultants. Alameda County Urban Runoff Clean Water Program. Oakland, CA

Pitt, R. 1985. *Bellevue Urban Runoff Project*. Final Report

Stormwater Manager's Resource Center, "Pollution Prevention Fact Sheets: Catch Basins"

TOWN OF ROCKPORT
Department of Public Works
Stormwater Catch Basin Cleanout Reporting Sheet

Date of Cleaning: _____

Catch Basin ID#: _____

Location of Catch Basin: _____

Description of Catch Basin

Depth of CB: _____

Depth of Sump: _____

Diameter of CB: _____

Number of Inlets: _____

Type of Catch Basin

Open Bottom: ☐ **Closed:** ☐ **Direct Discharge** ☐

Drainage System Connections

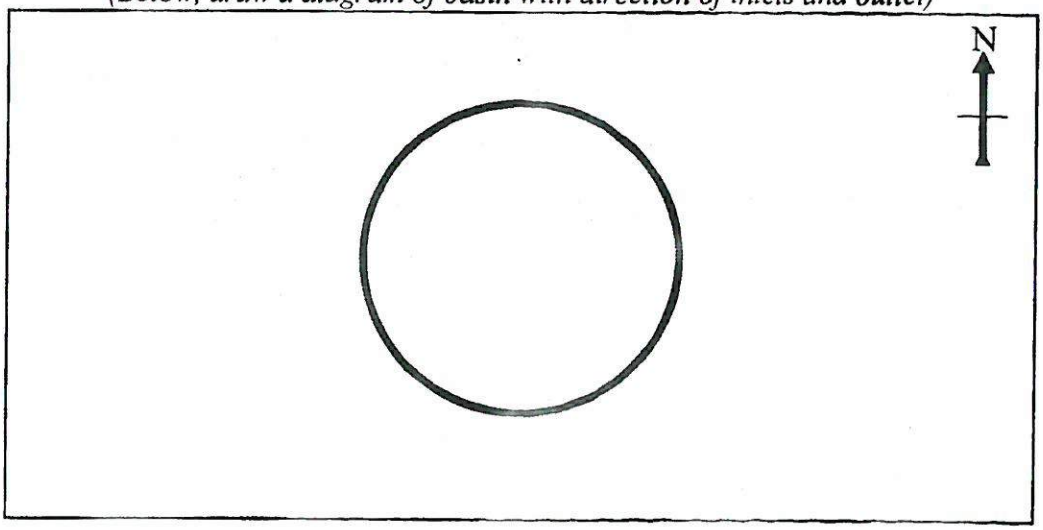
Basin-to-Basin: ☐ **Basin-to-Manhole:** ☐

Assessment of Stormwater Sediments

Depth at Deepest Point: _____

Volume of Material Removed: _____

(Below, draw a diagram of basin with direction of inlets and outlet)



Other Observations:

_____

_____

_____

_____

_____

Appendix I: Street Sweeping Plan

TOWN OF ROCKPORT

DEPARTMENT OF PUBLIC WORKS

STREET SWEEPING PLAN

FOR THE MILL BROOK WATERSHED AND OTHER CRITICAL WATERSHEDS THAT AFFECT VALUABLE WATER SUPPLY, FRESHWATER AND COASTAL WETLAND RESOURCES

Prepared as part of the
Lower Mill Brook Coastal Nonpoint Source Pollution Assessment

June 2005

Town of Rockport
Street Sweeping Plan

Purpose of the Plan

The purpose of this Street Sweeping Plan is to act as guidance for the town of Rockport Dept. of Public Works to maximize the efficiency and effectiveness of sediment and pollutant removal in its cleaning of the town's municipal streets and roadways.

Reasons for Sweeping Streets

It is important to sweep municipal streets and roadways for the following reasons:

- To keep sand, leaves, debris and other pollutants from building up on roadways
- To keep windborne roadway sands and silts from impacting air quality
- To keep sands and other roadway debris from clogging municipal drainage catch basins
- To keep sands and other roadway debris from washing into streams and wetlands
- To reduce the frequency that municipal catch basins must be cleaned to maintain optimal treatment capacity
- To minimize the current sediment build-up in Mill Pond and Frog Pond

Street Sweeping Schedule

Guidance from the USDA Natural Resources Conservation Service (NRCS) (Marc MacQueen, Pers. Com.) is that ***streets should be cleaned once per month unless there is reason to do them more often*** because of excessive volumes of debris or windborne sediment. Obviously for New England cities and towns, this excludes winter months when snow, ice and freezing temperatures may prevent year-round street sweeping.

Computer modeling in the Pacific Northwest suggest that from the standpoint of pollutant removal, the optimum sweeping frequency appears to be once every week or two (Claytor, 1999). More frequent sweeping operations yielded only a small increment in additional removal. The model also suggests that somewhat higher removal could be obtained on residential streets as opposed to more heavily traveled arterial roads.

More frequent street sweeping may reduce the number of annual catch basin cleanings needed to maintain optimal catch basin treatment capacity.

Disposal

Street sweepings are solid waste subject to the Massachusetts solid waste regulations. Street sweepings are not exempt from the Hazardous Waste Regulations, 310 CMR 30.000, and must be handled as hazardous waste when they exhibit any of the characteristics of a hazardous waste. If there is no evidence of unusual contamination, street sweepings do not have to be routinely tested, but, as is the case with any waste, the generator, in this case the Dept. of Public Works conducting the street sweeping, has the ultimate responsibility for determining whether the waste is a hazardous waste.

The options for disposing of or re-using street sweepings are as follows as outlined by MA DEP:

- **Disposal** Street sweepings may be disposed in either lined or unlined permitted solid waste landfills
- **Use at Landfills** Street sweepings may be used for daily cover at lined or unlined permitted solid waste landfills
- **Use as Fill or as an Additive to Restricted Use Compost** Street sweepings may be used for fill or as an additive to "restricted use" compost without prior approval from MA DEP only when the sweepings:
  - have not been collected from Urban Center Roads (see MA DEP for definition);
  - used as fill or compost ***only in public ways***
  - are used under the road surface or as fill along the side of the road within the public way;
  - are ***not used in residential areas***;
  - are kept above the level of the groundwater;
  - are not used in designated "No Salt Areas";
  - are ***not used within the 100 foot buffer zone of a wetland or within wetland resource areas*** including bordering vegetative wetlands and riverfront areas;
  - are ***not used within 500 feet of a ground or surface drinking water supply***.
- **Other Uses** Any use not pre-approved (outlined above) requires prior approval from MA DEP under the Beneficial Use provisions of the *Solid Waste Management Facility Regulations* at 310 CMR 19.060. To receive a "Beneficial Use Determination" or BUD, an application characterizing the waste and describing the proposed beneficial use must be submitted to MA DEP.

Reporting Sheet

It is important to keep a log of municipal street sweeping. The attached Reporting Sheet template or other form of log book should be used and filled out whenever municipal streets are swept.

References:

Claytor, R. 1999. "New Developments in Street Sweeper Technology." Center for Watershed Protection. Ellicott City, MD. *Watershed Protection Techniques*. 3(1).

Massachusetts Dept. of Environmental Protection (MA DEP) Bureau of Waste Prevention, Reuse and Disposal of Street Sweepings, Final Policy # BWP-94.092, 1994

Stormwater Manager's Resource Center, "Pollution Prevention Fact Sheet: Parking Lot and Street Cleaning"

TOWN OF ROCKPORT
Department of Public works
Street Sweeping Log Sheet

DATE	OPERATOR	START MILEAGE	END MILEAGE	STREETS SWEPT (including partial streets)	VOLUME SWEPT (est. in cu. Ft.)

Appendix J: Erosion and Sedimentation Control Model By-law

Model Stormwater Management Ordinance

CHAPTER __

Erosion and Sediment Control For Stormwater Management

SECTION _-1. PURPOSE AND AUTHORITY

1. Purpose

A. The purpose of this ordinance is to better manage land development in order to protect, maintain, and enhance the public health, safety, and general welfare of the citizens of (Town Name) by establishing minimum requirements and procedures to control the adverse impacts associated with stormwater runoff.

B. The proper management of stormwater runoff will meet the following objectives:

Reduce the adverse water quality impacts of stormwater discharges to rivers, lakes, reservoirs and streams in order to attain federal water quality standards;
Prevent the discharge of pollutants, including hazardous chemicals, into stormwater runoff;
Minimize the volume and rate of stormwater which is discharged, to rivers, streams, reservoirs, lakes and combined sewers that flows from any site during and following development;
Prevent erosion and sedimentation from land development, and reduce stream channel erosion caused by increased runoff;
Provide for the recharge of groundwater aquifers and maintain the base flow of streams;
Provide stormwater facilities that are attractive, maintain the natural integrity of the environment, and are designed to protect public safety;
Maintain or reduce pre-development runoff characteristics after development to the extent feasible;
Minimize damage to public and private property from flooding;
Ensure that these management controls are properly maintained.

2. Authority

The Department of Public Works shall administer, implement and enforce this ordinance. Any powers granted to or duties imposed upon the Department of Public Works may be delegated in writing by the Board of Public Works to employees or agents of the Department of Public Works.

SECTION _-2. DEFINITIONS

The following definitions describe the meaning of the terms used in this Ordinance:

Authorized Enforcement Agency: The Department of Public Works, its employees or agents designated to enforce this ordinance.

"Adverse impact" means any deleterious effect on waters or wetlands, including their quality, quantity, surface area, species composition, aesthetics or usefulness for human or natural uses which are or may potentially be harmful or injurious to human health, welfare, safety or property, to biological productivity, diversity, or stability or which unreasonably interfere with the enjoyment of life or property, including outdoor recreation.

"Best management practices (BMP)" are structural or biological devices that temporarily store or treat urban stormwater runoff to reduce flooding, remove pollutants, and provide other amenities. They can also be non-structural practices that reduce pollutants at their source. BMPs are described in a stormwater design manual, Stormwater Management, Volume Two: Stormwater Technical Handbook (March, 1997, Mass. Department of Environmental Protection, as updated or amended).

"Construction activity" is disturbance of the ground by removal of vegetative surface cover or topsoil, grading, excavation, clearing or filling.

"Design storm" is a rainfall event of specified size and return frequency that is used to calculate the runoff volume and peak discharge rate to a BMP.

"Detention" is the temporary storage of storm runoff in a BMP, which is used to control the" peak discharge rates, and which provides gravity settling of pollutants.
extended detention

"Disturbance" is any land clearing, grading, bulldozing, digging or similar activities.

"Drainage area" means that area contributing runoff to a single point measured in a horizontal plane, which is enclosed by a ridgeline.

"Drywell" is similar to an infiltration trench but smaller with inflow from a pipe; commonly covered with soil and used for drainage areas of less than 1 acre such as roadside inlets and rooftops runoff.

"Easement" means a grant or reservation by the owner of land for the use of such land by others for a specific purpose or purposes, and which must be included in the conveyance of land affected by such easement.

"Flow attenuation" means prolonging the flow time of runoff to reduce the peak discharge.

"Hydrology model" may include one of the following:
TR-20, a watershed hydrology model developed by the Natural Resources Conservation Service act that is used to route a design storm hydrograph through a pond;
TR 55, or Technical Release 55, "Urban Hydrology for Small Watersheds" is a publication developed by the Natural Resources Conservation Service to calculate stormwater runoff and an aid in designing detention basins;
Hydrocad.

"Impervious surfaces" are areas, such as pavement or rooftops, which prevent the infiltration of water into the soil.

"Infiltration" is the downward movement of water from the surface to the subsoil.

"Infiltration trench"" is a stormwater management excavation filled with aggregate which removes both soluble and particulate pollutants. Trenches are not intended to trap coarse sediments.

"Outfall" is the terminus of a storm drain or other stormwater structure where the contents are released.

"Peak discharge" is the maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event

"Permeable soils" are soil materials with a sufficiently rapid infiltration rate so as to greatly reduce or eliminate surface and stormwater runoff. These soils are generally classified as NRCS hydrologic soil types A and B.

"Person" is any individual, group of individuals, association, partnership, corporation, company, business, organization, trust, estate, administrative agency, public or quasi-public corporation or body, the Commonwealth or political subdivision thereof.

"Retention" is the holding of runoff in a basin without release except by means of evaporation, infiltration, or emergency bypass.

"Start of construction" is the first land-disturbing activity associated with a development, including land preparation such as: clearing, grading and filling; installation of streets and walkways; excavation for basements; footings, piers or foundations; erection of temporary forms; and installation of accessory buildings such as garages.

"Swale" is a natural depression or wide shallow ditch used to temporarily store, route, or filter runoff.

SECTION _-3. APPLICABILITY

1. Applicability

Prior to the issuance of any site plan approval or development permit for any proposed development listed below, a stormwater management permit, or a waiver of the requirement for a stormwater management permit, must be approved by the applicable Special Permit Granting Authority. No person shall, on or after the effective date of the ordinance, initiate any land clearing, land grading, earth moving or development activities without first complying with this ordinance. The following uses and activities shall be required to submit drainage reports, plans, construction drawings, specifications and as-constructed information in conformance with the requirements of this ordinance:

Multi-family residential developments involving four or more units;
Any new commercial, industrial, and institutional structures under the same ownership, with at least 5,000 square feet of gross floor area, 10,000 square feet of impervious surface, or that require 10 or more parking spaces.
Redevelopment or additions to existing commercial, industrial, and institutional uses which result in an additional impervious surface area or gross floor area of greater than 5,000 square feet, or which results in an increase of 10 or more parking spaces.
Subdivisions and construction activities of any kind disturbing greater than 5,000 square feet.
Development or redevelopment involving multiple separate activities in discontinuous locations or on different schedules if the activities are part of a larger common plan of development that all together disturbs one or more acres.

Exemptions

To prevent the adverse impacts of stormwater runoff, the stormwater performance standards in Section 29-6 must be met at new development sites. These standards apply to construction activities as described under Section 29-3.1. The following activities are exempt from the requirements for submittal and approval of a stormwater management plan under Section 29-4, but must comply with the stormwater performance standards in Section 29-6:
Any agricultural activity which is consistent with an approved soil conservation plan prepared or approved by the Natural Resource Conservation Service;

Any logging which is consistent with a timber management plan approved under the Forest Cutting Practices Act by Massachusetts Department of Environmental Management;
Additions or modifications to existing single family structures;
Developments that do not disturb more than 5,000 square feet of land, provided that they are not part of a larger common development plan;
Repairs to any stormwater treatment system deemed necessary by the (Town Name) Department of Public Works;
Any emergency activity that is immediately necessary for the protection of life, property or the environment, as determined by the Department of Public Works; and
Single family residential uses disturbing less than 5,000 square feet.

Stormwater Design Manual

A stormwater design manual, Stormwater Management, Volume Two: Stormwater Technical Handbook (March, 1997, Mass. Department of Environmental Protection, as updated or amended) is hereby incorporated by reference as part of this ordinance, and shall furnish additional policy, criteria and information including specifications and standards, for the proper implementation of the requirements of this ordinance.

This manual includes a list of acceptable stormwater treatment practices, including the specific design criteria for each stormwater practice. The manual may be updated and expanded from time to time, based on improvements in engineering, science, monitoring and local maintenance experience, at the discretion of the (Town Name) Department of Public Works or Massachusetts Department of Environmental Protection. Stormwater treatment practices that are designed and constructed in accordance with these design and sizing criteria will be presumed to meet the minimum water quality performance standards.

SECTION _-4 . PERMIT PROCEDURES AND REQUIREMENTS

1. Permit Required

No land owner or land operator shall receive any of the building, grading, or other land development permits required for land disturbance activities, and no land owner shall commence land disturbance activities, without approval of a Stormwater Management Permit from the Department of Public Works and meeting the requirements of this ordinance.

Application Requirements

Application for approval of a Stormwater Management Permit shall include the following:

A stormwater management plan or an application for waiver shall be submitted to the (Town Name) Department of Public Works for review and approval for any proposed development specified in Section 3.1. Three copies of the stormwater management plan shall be submitted, and clearly labeled, along with other documents required in this zoning ordinance for site plan review. The plan shall contain supporting computations, drawings, and sufficient information describing the manner, location, and type of measures in which stormwater runoff will be managed from the entire development. The plan shall serve as the basis for all subsequent construction.

An erosion and sediment control plan, which shall contain sufficient information to describe the nature and purpose of the proposed development.

ongoing maintenance agreement

non-refundable permit review fee

The applicant may request, and the (Town Name) Department of Public Works may grant, a waiver from any information requirements it judges to be unnecessary to the review of a particular plan.

3. Procedures for Review and Approval of Stormwater Permits

The procedures for review and approval of stormwater management permits shall be consistent with (review procedures of DPW), as appropriate to the use.

The Department of Public Works shall refer copies to the stormwater management permits to the Town Engineer for review, and shall consider any comments submitted by the Town Engineer during the review period.

The Department of Public Works shall hold a public hearing within twenty-one (21) days of the receipt of a complete application and shall take final action within twenty-one (21) days from the close of the hearing unless such time is extended by agreement between the applicant and [insert appropriate board or department]. Notice of the public hearing shall be given by publication in a local paper of general circulation, by posting and by first-class mailings to abutters at least seven (7) days prior to the hearing.

4. Criteria for Review of Stormwater Permits

In addition to other criteria used by the (Town Name) Department of Public Works in making permit decisions, for the uses specified in this ordinance, the Department of Public Works must also find that the Stormwater Management Plan submitted with the permit application meets the following criteria:

A. the Stormwater Management Plan and the Erosion and Sediment Control Plan are consistent with the Purposes and Objectives of this Bylaw in Section 29-1;

the Stormwater Management Plan meets the Performance Standards described in Section 29-6;

the Erosion and Sediment Control plan must meet the Design Requirements in Section 29-7.

DPW Action

The Department of Public Works' action, rendered in writing, shall consist of either:

Approval of the Stormwater Management Permit Application based upon determination that the proposed plan meets the purposes in Section 29-1 and the standards in Section 29-6 and will adequately protect the water resources of the community and is in compliance with the requirements set forth in this by-law;

Approval of the Stormwater Management Permit Application subject to any conditions, modifications or restrictions required by the Board which will ensure that the project meets the purposes in Section 29-1 and the standards in Section 29-6 and adequately protects water resources, set forth in this by-law;

Disapproval of the Stormwater Management Permit Application based upon a determination that the proposed plan, as submitted, does not meet the purposes in Section 29-1 and the standards in Section 29-6 or adequately protect water resources, as set forth in this by-law.

Failure of the Board to take final action upon an Application within the time specified above shall be deemed to be approval of said Application. Upon certification by the Town Clerk that the allowed time has passed without Board action, the Board must issue a Stormwater Management Permit.

6. Inspections

No Plan will be approved without adequate provision for inspection of the property before development activity commences. The applicant shall arrange with the DPW for scheduling the following inspections:

Initial inspection: prior to approval of any plan
Erosion Control Inspections: after site clearing, rough grading and final grading to ensure erosion control practices are in accord with the plan.
Bury inspection: prior to backfilling of any underground drainage or stormwater conveyance structures;
Final Inspection: when all work, including construction of stormwater management facilities and landscaping have been completed. Final inspection shall include a full, dated TV inspection of all stormwater pipes installed.

The Department of Public Works or its agent shall inspect the work and either approve it or notify the applicant in writing in what respects there has been a failure to comply with the requirements of the approved plan. Any portion of the work which does not comply shall be promptly corrected by the applicant or the applicant will be subject to the bonding provisions of Section 29-9 or the penalty provisions of Section 29-10. The Town may conduct random inspections to ensure effective control of erosion and sedimentation during all phases of construction.

Right-of-Entry for Inspection

When any new drainage control facility is installed on private property, or when any new connection is made between private property and a public drainage control system or sanitary sewer, the filing of an application shall be deemed as the property owner's permission to the (Town Name) Department of Public Works for the right to enter the property at reasonable times and in a reasonable manner for the purpose of the inspection. This includes the right to enter a property when it has a reasonable basis to believe that a violation of this ordinance is occurring or has occurred, and to enter when necessary for abatement of a public nuisance or correction of a violation of this ordinance.

Application Review Fees

The fee for review of any land development application shall be based on the amount of land to be disturbed at the site and the fee structure established by the (Town Name) Board of Public Works. All of the monetary contributions shall be credited to the utility enterprise fund, and shall be made prior to issuance of any building permit for development.

Section _-5. The Stormwater Management and Erosion Control Plan

Contents of the Stormwater Management and Erosion Control Plan

The application for a stormwater management permit shall consist of submittal of a stormwater management and erosion control plan, prepared by a professional engineer licensed by the Commonwealth of Massachusetts, which meets the design requirements provided by this

Ordinance. The plan shall include sufficient information to evaluate the environmental characteristics of the affected areas, the potential impacts of the proposed development on water resources; and the effectiveness and acceptability of measures proposed for managing stormwater runoff. The Plan must be designed to meet the Massachusetts Stormwater Management Standards as set forth in Section 29-6 of this ordinance and the DEP Stormwater Management Handbook Volumes I and II. The applicant shall certify on the drawings that all clearing, grading, drainage, construction, and development shall be conducted in strict accordance with the plan. The minimum information submitted for support of a stormwater management plan shall be as follows:

A. A locus map,
The existing zoning, and land use at the site,
The proposed land use,
The location(s) of existing and proposed easements,
The location of existing and proposed utilities,
The site's existing & proposed topography with contours at 2 foot intervals,
The existing site hydrology,
A description & delineation of existing stormwater conveyances, impoundments, and wetlands on or adjacent to the site or into which storm water flows.
A delineation of 100-year flood plains, if applicable
Estimated seasonal high groundwater elevation (November to April) in areas to be used for storm water retention, detention, or infiltration.
The existing and proposed vegetation and ground surfaces with runoff coefficient for each,
A drainage area map showing pre and post construction watershed boundaries, drainage area and storm water flow paths,
A description and drawings of all components of the proposed drainage system including:
locations, cross sections, and profiles of all brooks, streams, drainage swales and their method of stabilization,
all measures for the detention, retention or infiltration of water,
all measures for the protection of water quality,
the structural details for all components of the proposed drainage systems and storm water management facilities,
notes on drawings specifying materials to be used, construction specifications, and typicals, and expected hydrology with supporting calculations.
Proposed improvements including location of buildings or other structures, impervious surfaces, and drainage facilities, if applicable,
A description of construction and waste materials expected to be stored on-site, and a description of controls to reduce pollutants from these materials including storage practices to minimize exposure of the materials to storm water, and spill prevention and response.
Timing, schedules, and sequence of development including clearing, stripping, rough grading, construction, final grading, and vegetative stabilization, and
A maintenance schedule for the period of construction

Section _-6. Stormwater Management Performance Standards

1. Minimum Control Requirements

Projects must meet the Standards of the Massachusetts Stormwater Management Policy. These Standards are:

A. No new stormwater conveyances (e.g. outfalls) may discharge untreated stormwater directly to or cause erosion in wetlands or water of the Commonwealth.

B. Stormwater management systems must be designed so that post-development peak discharge rates do not exceed pre-development peak discharge rates.

C. Loss of annual recharge to groundwater should be minimized through the use of infiltration measures to the maximum extent practicable. The annual recharge from the post-development site should approximate the annual recharge rate from the pre-development or existing site conditions, based on soil types.

For new development, stormwater management systems must be designed to remove 80% of the average annual load (post development conditions) of Total Suspended Solids (TSS). It is presumed that this standard is met when:

(1) Suitable nonstructural practices for source control and pollution prevention and implemented;
(2) Stormwater management best management practices (BMPs) are sized to capture the prescribed runoff volume; and
(3) Stormwater management BMPs are maintained as designed.

E. Stormwater discharges from areas with higher potential pollutant loads require the use of specific stormwater management BMPs (see Stormwater Management Volume I: Stormwater Policy Handbook). The use of infiltration practices without pretreatment is prohibited.

F. Stormwater discharges to critical areas must utilize certain stormwater management BMPs approved for critical areas (see Stormwater Management Volume I: Stormwater Policy Handbook). Critical areas are Outstanding Resource Waters (ORWs), shellfish beds, swimming beaches, cold water fisheries and recharge areas for public water supplies.

G. Redevelopment of previously developed sites must meet the Stormwater Management Standards to the maximum extent practicable. However, if it is not practicable to meet all the Standards, new (retrofitted or expanded) stormwater management systems must be designed to improve existing conditions.

H. Erosion and sediment controls must be implemented to prevent impacts during disturbance and construction activities.

I. All stormwater management systems must have an operation and maintenance plan to ensure that systems function as designed.

When the proposed discharge may have an impact upon a sensitive receptor, including streams, storm sewers, and/or combined sewers, the DPW may require an increase in these minimum requirements, based on existing stormwater system capacity.

2. Stormwater Management Measures

A. Stormwater management measures shall be required to satisfy the minimum control requirements and shall be implemented in the following order of preference:

Infiltration, flow attenuation, and pollutant removal of runoff on-site to existing areas with grass, trees, and similar vegetation and through the use of open vegetated swales and natural depressions;
Use of stormwater on-site to replace water used in industrial processes or for irrigation;
Stormwater detention structures for the temporary storage of runoff which is designed so as not to create a permanent pool of water; and
Stormwater retention structures for the permanent storage of runoff by means of a permanent pool of water.

Retention and evaporation of stormwater on rooftops or in parking lots;

B. Infiltration practices shall be utilized to reduce runoff volume increases. A combination of successive practices may be used to achieve the applicable minimum control requirements. Justification shall be provided by the applicant for rejecting each practice based on site conditions.

C. Best Management Practices shall be employed to minimize pollutants in stormwater runoff prior to discharge into a separate storm drainage system or water body.

D. All stormwater management facilities shall be designed to provide an emergency overflow system, and incorporate measures to provide a non-erosive velocity of flow along its length and at any outfall.

E. The designed release rate of any stormwater structure shall be modified if any increase in flooding or stream channel erosion would result at a downstream dam, highway, structure, or normal point of restricted stream flow.

3. Specific Design Criteria

Additional policy, criteria, and information including specifications and design standards may be found in the Stormwater Design Manual.

Infiltration systems

Infiltration systems shall be equipped with clean stone and or filter fabric adjacent to the soil or other sediment removal mechanisms;
Infiltration systems greater than 3 feet deep shall be located at least l0 feet from basement walls;
Due to the potential for groundwater contamination from dry wells, they shall not be an acceptable method for management runoff containing pollutants;
Infiltration systems designed to handle runoff from commercial or industrial impervious parking areas shall be a minimum of 100 feet from any drinking water supply well;
Infiltration systems shall not be used as sediment control basins during construction unless specific plans are included to restore or improve the basin surface;
Infiltration basins shall be constructed with a three foot minimum separation between the bottom of the structure and the seasonal high groundwater elevation, as determined by a certified soil evaluator; and
Provisions shall be made for safe overflow passage, in the event of a storm which exceeds the capacity of an infiltration system.

Retention and detention ponds shall be designed and constructed in accordance with the criteria of the Stormwater Management, Volume Two: Stormwater Technical Handbook (March, 1997, Mass. Department of Environmental Protection, as updated or amended).

C. The applicant shall give consideration in any plan to incorporating the use of natural topography and land cover such as natural swales, and depressions as they exist prior to development to the degree that they can accommodate the additional flow of water.

D. The Department of Public Works shall give preference to the use of swales in place of the traditional use of curbs and gutters based on a case by case review of stormwater management plans by the Town Engineer and Department of Public Works.

E. The applicant shall consider public safety in the design of any stormwater facilities. The banks of detention, retention, and infiltration basins shall be sloped at a gentle grade into the

water as a safeguard against personal injury, to encourage the growth of vegetation and to allow the alternate flooding and exposure of areas along the shore. Basins shall have a 4:1 slope to a depth two feet below the control elevation. Side slopes must be stabilized and planted with vegetation to prevent erosion and provide pollutant removal The banks of detention and retention areas shall be designed with sinuous rather than straight shorelines so that the length of the shoreline is maximized, thus offering more space for the growth of vegetation;

F. Where a stormwater management plan involves direction of some or all runoff off of the site, it shall be the responsibility of the applicant to obtain from adjacent property owners any easements or other necessary property interests concerning flowage of water. Approval of a stormwater management plan does not create or affect any such rights.

G. All applicants for projects which involve the storage or use of hazardous chemicals shall incorporate handling and storage "best management practices" that prevent such chemicals from contaminating runoff discharged from a site into infiltration systems, receiving water bodies or storm drains, and shall include a list of such chemicals in the application.

H. Runoff from parking lots shall be treated by oil and water separators or other controls to remove oil and sediment;

I. The basic design criteria methodologies, and construction specifications, subject to the approval of the Department of Public Works and Town Engineer, shall be those generally found in the most current edition of the Stormwater Management, Volume Two: Stormwater Technical Handbook (March, 1997, Mass. Department of Environmental Protection, as updated or amended).

Section _-7. Design Requirements for Erosion and Sediment Control Plan

1. The design requirements of the Erosion and Sediment Control Plan are:

Minimize total area of disturbance

Sequence activities to minimize simultaneous areas of disturbance.

Minimize peak rate of runoff in accordance with the MA DEP Stormwater Policy.

Minimize soil erosion and control sedimentation during construction. Prevention of erosion is preferred over sedimentation control.

Divert uncontaminated water around disturbed areas

Maximize groundwater recharge.

Install, and maintain all Erosion and Sediment Control measures in accordance with the manufacturers specifications and good engineering practices

Prevent off-site transport of sediment.

Protect and manage on and off-site material storage areas (overburden and stockpiles of dirt, borrow areas, or other areas used solely by the permitted project are considered a part of the project).

Comply with applicable Federal, State and local laws and regulations including waste disposal,

sanitary sewer or septic system regulations, and air quality requirements, including dust control

Prevent adverse impact from the proposed activities to habitats mapped by the Massachusetts Natural Heritage & Endangered Species Program as Endangered, Threatened or of Special concern, Estimated Habitats of Rare Wildlife and Certified Vernal Pools, and Priority Habitats of Rare Species.

Institute interim and permanent stabilization measures. The measures shall be instituted on a disturbed area as soon as practicable but no more than 14 days after construction activity has temporarily or permanently ceased on that portion of the site.

Properly manage on-site construction and waste materials.

Prevent off-site vehicle tracking of sediments.

Section _-8 Maintenance

1. Operation, Maintenance and Inspection Agreement

Prior to issuance of any building permit for which stormwater management is required, the Department of Public Works shall require the applicant or owner to execute an operation, maintenance and inspection agreement binding on all subsequent owners of land served by the private stormwater management facility. The agreement shall be designed to ensure that water quality standards are met in all seasons and throughout the life of the system. Such agreement shall provide for access to the facility at reasonable times for regular inspections by the Town or its authorized representative and for regular or special assessments of property owners to ensure that the facility is maintained in proper working condition to meet design standards and any provision established. The agreement shall include:

The name(s) of the owner(s) for all components of the system.
Maintenance agreements that specify:
The names and addresses of the person(s) responsible for operation and maintenance.
The person(s) responsible for financing maintenance and emergency repairs.
A Maintenance Schedule for all drainage structures, including swales and ponds.
A list of easements with the purpose and location of each.
The signature(s) of the owner(s).
Stormwater management easements as necessary for:
Access for facility inspections and maintenance.
Preservation of stormwater runoff conveyance, infiltration, and detention areas and facilities, including flood routes for the 100-year storm event.
Direct maintenance access by heavy equipment to structures requiring regular cleanout.
(4) Stormwater management easement requirements:
The purpose of each easement shall be specified in the maintenance agreement signed by the property owner.
Stormwater management easements are required for all areas used for off-site stormwater control, unless a waiver is granted by the Town.
Easements shall be recorded with the Registry of Deeds prior to issuance of a Certificate of Completion.
(5) Changes to Operation and Maintenance Plans
The owner(s) of the stormwater management system must notify the Department of Public Works] of changes in ownership or assignment of financial responsibility.
The maintenance schedule in the Maintenance Agreement may be amended to achieve the purposes of this by-law by mutual agreement of the Department of Public Works] and the

Responsible Parties. Amendments must be in writing and signed by all Responsible Parties. Responsible Parties must include owner(s), persons with financial responsibility, and persons with operational responsibility.

B. The agreement shall be recorded by the applicant and/or owner in the land records of the Registry of Deeds.

C. The agreement shall also provide that, if after notice by the Town Engineer to correct a violation requiring maintenance work, satisfactory corrections are not made by the owner(s) within thirty days, the Department of Public Works may perform all necessary work to place the facility in proper working condition. The owner(s) of the facility shall be assessed the cost of the work and any penalties.

2. Maintenance Responsibility

A. The owner of the property on which work has been done pursuant to this Ordinance for private stormwater management facilities, or any other person or agent in control of such property, shall maintain in good condition and promptly repair and restore all grade surfaces, walls, drains, dams and structures, vegetation, erosion and sediment control measures and other protective devices. Such repairs or restoration and maintenance shall be in accordance with approved plans.

B. A maintenance schedule shall be developed for the life of any stormwater management facility and shall state the maintenance to be completed, the time period for completion, and who shall be legally responsible to perform the maintenance. This maintenance schedule shall be printed on the stormwater management plan.

C. Records of installation and maintenance

D. Failure to maintain practices

Section _-9. Performance Bond

The Department of Public Works shall require from the developer a surety or cash bond, irrevocable letter of credit, or other means of security acceptable to the Department of Public Works prior to the issuance of any building permit for the construction of a development requiring a stormwater management facility. The amount of the security shall not be less than the total estimated construction cost of the stormwater management facility. The bond so required in this section shall include provisions relative to forfeiture for failure to complete work specified in the approved stormwater management plan, compliance with all of the provisions of this Ordinance and other applicable laws and regulations, and any time limitations. The bond shall not be fully released without a final inspection of the completed work by the Town Engineer, submission of" As-built" plans, and certification of completion by the Department of Public Works of the stormwater management facilities being in compliance with the approved plan and the provisions of this Ordinance.

Section _-10. Enforcement and Penalties

1. Violations

Any development activity that has commenced or is conducted contrary to this Ordinance may be restrained by injunction or otherwise abated in a manner provided by law.

2. Notice of Violation

When the (Town Name) Department of Public Works determines that an activity is not being carried out in accordance with the requirements of this Ordinance, it shall issue a written notice of violation to the owner of the property. The notice of violation shall contain:

the name and address of the owner applicant;

the address when available or the description of the building, structure, or land upon which the violation is occurring;

a statement specifying the nature of the violation;

a description of the remedial measures necessary to bring the development activity into compliance with this Ordinance and a time schedule for the completion of such remedial action;

a statement of the penalty or penalties that shall or may be assessed against the person to whom the notice of violation is directed;

a statement that the determination of violation may be appealed to the municipality by filing a written notice of appeal within fifteen (15) days of service of notice of violation.

3. Stop Work Orders

Persons receiving a notice of violations will be required to halt all construction activities. This "stop work order" will be in effect until the (Town Name) Department of Public Works confirms that the development activity is in compliance and the violation has been satisfactorily addressed. Failure to address a notice of violation in a timely manner can result in civil, criminal, or monetary penalties in accordance with the enforcement measures authorized in this Ordinance.

4. Criminal and Civil Penalties

Any person who violates any provision of this ordinance, valid regulation, or the terms or conditions in any permit or order prescribed or issued thereunder, shall be subject to a fine not to exceed $300.00 for each day such violation occurs or continues or subject to a civil penalty, which may be assessed in an action brought on behalf of the Town in any court of competent jurisdiction.

5. Non-Criminal Disposition

As an alternative to criminal prosecution or civil action, the Town of (Town Name) may elect to utilize the non-criminal disposition procedure set forth in (Town Name) Town ordinances ~~G.L. Ch. 40, §21D~~. The Department of Public Works shall be the enforcing entity. The penalty for the 1st violation shall be up to $100. The penalty for the 2nd violation shall be up to $200. The penalty for the 3rd and subsequent violations shall be $ 300.00. Each day or part thereof that such violation occurs or continues shall constitute a separate offense.

6. Restoration of Lands

Any violator may be required to restore land to its undisturbed condition. In the event that restoration is not undertaken within a reasonable time after notice, the (Town Name) Department of Public Works may take necessary corrective action, the cost of which shall become a lien upon the property until paid.

7. Holds on Occupancy Permits

Occupation permits will not be granted until corrections to all stormwater practices have been made and accepted by the (Town Name) Department of Public Works.

Section _-11. Severability

The invalidity of any section or provision of this Ordinance shall not invalidate any other section or provision thereof.

Appendix K: Post-Construction Stormwater Management Model By-law

MODEL STORMWATER BYLAW
Duxbury, Marshfield, and Plymouth
December 31, 2004

Introduction

It is hereby determined that:

Land development projects and other land use conversions, and their associated changes to land cover, permanently alter the hydrologic response of local watersheds and increase stormwater runoff rates and volumes, which in turn increase flooding, stream channel erosion, and sediment transport and deposition, and decrease groundwater recharge;

Land development projects and other land use conversions also contribute to increased nonpoint source pollution and degradation of receiving waters;

The impacts of post-development stormwater runoff quantity and quality can adversely affect public safety, public and private property, surface water drinking water supplies, groundwater resources, drinking water supplies, recreation, aquatic habitats, fish and other aquatic life, property values and other uses of lands and waters;

These adverse impacts can be controlled and minimized through the regulation of stormwater runoff quantity and quality from new development and redevelopment, by the use of both structural and nonstructural Best Management Practices;

Localities in the Commonwealth of Massachusetts are required to comply with a number of both State and Federal laws, regulations and permits which require a locality to address the impacts of post-development stormwater runoff quality and nonpoint source pollution.

Therefore, the *[Stormwater Authority]* has established this stormwater management bylaw to provide reasonable guidance for the regulation of post-development stormwater runoff for the purpose of protecting local water resources from degradation. This bylaw regulates the post-construction stormwater controls for both new and re-development projects.

It has been determined that it is in the public interest to regulate post-development stormwater runoff discharges in order to control and minimize increases in stormwater runoff rates and volumes, post-construction soil erosion and sedimentation, stream channel erosion, and nonpoint source pollution associated with post-development stormwater runoff.

1.0 PURPOSE

A) The purpose of this Bylaw is to protect, maintain and enhance the public health, safety, environment and general welfare by establishing minimum requirements and procedures to control the adverse effects of increased post-development stormwater runoff and nonpoint source pollution associated with new development and redevelopment. It has been determined that proper management of post-development stormwater runoff will minimize damage to public and private property and infrastructure, safeguard the public health, safety, environment and general welfare of the public, protect water and aquatic resources, and promote groundwater recharge to protect surface and groundwater drinking supplies. This Bylaw seeks to meet that purpose through the following objectives:

1. Establish decision-making processes surrounding land development activities that protect the

integrity of the watershed and preserve the health of water resources;

2. Require that new development, redevelopment and all land conversion activities maintain the after-development runoff characteristics as equal to or less than the pre-development runoff characteristics in order to reduce flooding, stream bank erosion, siltation, nonpoint source pollution, property damage, and to maintain the integrity of stream channels and aquatic habitats;

3. Establish minimum post-development stormwater management standards and design criteria for the regulation and control of stormwater runoff quantity and quality; Establish minimum design criteria for the protection of properties and aquatic resources downstream from land development and land conversion activities from damages due to increases in volume, velocity, frequency, duration, and peak flow rate of storm water runoff; Establish minimum design criteria for measures to minimize nonpoint source pollution from stormwater runoff which would otherwise degrade water quality;

4. Establish design and application criteria for the construction and use of structural stormwater control facilities that can be used to meet the minimum post-development stormwater management standards;

5. Encourage the use of nonstructural stormwater management, stormwater better site design practices or "low-impact development practices", such as reducing impervious cover and the preservation of greenspace and other natural areas, to the maximum extent practicable; Coordinate site design plans, which include greenspace, with the Town's greenspace protection plan;

6. Establish provisions for the long-term responsibility for and maintenance of structural stormwater control facilities and nonstructural stormwater management practices to ensure that they continue to function as designed, are maintained, and pose no threat to public safety;

7. Establish provisions to ensure there is an adequate funding mechanism, including surety, for the proper review, inspection and long-term maintenance of stormwater facilities implemented as part of this Bylaw;

8. Establish administrative procedures for the submission, review, approval or disapproval of stormwater management plans, and for the inspection of approved active projects, and long-term follow up; Establish certain administrative procedures and fees for the submission, review, approval, or disapproval of stormwater plans, and the inspection of approved projects.

B) Nothing in this Bylaw is intended to replace the requirements of either, the Town of [_____] Flood Plain Zoning Bylaw, the Town of [_____] General Wetlands Protection Bylaw, or any other Bylaw that may be adopted by the Town of [_____]. Any activity subject to the provisions of the above-cited Bylaws must comply with the specifications of each.

2.0 DEFINITIONS

The following definitions shall apply in the interpretation and implementation of this Bylaw. Additional definitions may be adopted by separate regulation:

ALTER: Any activity, which will measurably change the ability of a ground surface area to absorb water or will change existing surface drainage patterns. Alter may be similarly represented as "alteration of drainage characteristics," and "conducting land disturbance activities."

BEST MANAGEMENT PRACTICE (BMP): Structural, non-structural and managerial techniques that are recognized to be the most effective and practical means to prevent and/or reduce increases in stormwater volumes and flows, reduce point source and nonpoint source pollution, and promote stormwater quality and protection of the environment. "Structural" BMPs are devices that are engineered and constructed to provide temporary storage and treatment of stormwater runoff. "Nonstructural" BMPs use natural measures to reduce pollution levels, do not require extensive construction efforts, and/or promote pollutant reduction by eliminating the pollutant source.

BETTER SITE DESIGN: Site design approaches and techniques that can reduce a site's impact on the watershed through the use of nonstructural stormwater management practices. Better site design includes conserving and protecting natural areas and greenspace, reducing impervious cover, and using natural features for stormwater management.

GENERAL STORMWATER MANAGEMENT PERMIT (GSMP): A permit issued for an application that meets a set of pre-determined standards outlined in the Regulations to be adopted by the *[Stormwater Authority]* under Section 4 of this Bylaw. By meeting these pre-determined standards, the proposed project will be presumed to meet the requirements and intent of this Bylaw.

HOTSPOT: Land uses or activities with higher potential pollutant loadings, such as auto salvage yards, auto fueling facilities, fleet storage yards, commercial parking lots with high intensity use, road salt storage areas, commercial nurseries and landscaping, outdoor storage and loading areas of hazardous substances, or marinas.

MASSACHUSETTS STORMWATER MANAGEMENT POLICY: The Policy issued by the Department of Environmental Protection, and as amended, that coordinates the requirements prescribed by state regulations promulgated under the authority of the Massachusetts Wetlands Protection Act G.L. c. 131 § 40 and Massachusetts Clean Waters Act G.L. c. 21, §. 23-56. The Policy addresses stormwater impacts through implementation of performance standards to reduce or prevent pollutants from reaching water bodies and control the quantity of runoff from a site.

NEW DEVELOPMENT: Any construction or land disturbance of a parcel of land that is currently in a natural vegetated state and does not contain alteration by man-made activities.

NONPOINT SOURCE POLLUTION: Pollution from many diffuse sources caused by rainfall or snowmelt moving over and through the ground. As the runoff moves, it picks up and carries away natural and human-made pollutants, finally depositing them into water resource areas.

PERSON: Any individual, group of individuals, association, partnership, corporation, company, business organization, trust, estate, the Commonwealth or political subdivision thereof to the extent subject to Town Bylaws, administrative agency, public or quasi-public corporation or body, the Town of *[_____]*, and any other legal entity, its legal representatives, agents, or assigns.

PRE-DEVELOPMENT: The conditions that exist at the time that plans for the land development of a tract of land are submitted to the *[Stormwater Authority]*. Where phased development or plan approval occurs (preliminary grading, roads and utilities, etc.), the existing conditions at the time prior to the first plan submission shall establish pre-development conditions.

POST-DEVELOPMENT: The conditions that reasonably may be expected or anticipated to exist after completion of the land development activity on a specific site or tract of land. Post-

development refers to the phase of a new development or redevelopment project after completion, and does not refer to the construction phase of a project.

RECHARGE: The replenishment of underground water reserves.

REDEVELOPMENT: Any construction, alteration, or improvement exceeding land disturbance of *[5,000]* square feet, where the existing land use is commercial, industrial, institutional, or multi-family residential.

STORMWATER AUTHORITY: the Town *of [_____] [Planning Board, Conservation Commission, Board of Health, or other specifically authorized Dept. or entity the Town decides is appropriate to administer, implement and enforce this bylaw, OR its authorized agent(s)]*. The *[Stormwater Authority]* is responsible for coordinating the review, approval and permit process as defined in this Bylaw. Other Boards and/or departments participate in the review process as defined in the Stormwater Regulations adopted by the *[Boards, Commissions and/or Departments of the Town of_____]*.

STORMWATER CREDITS: A form of incentive for developers to promote conservation of natural and open space areas. Projects that comply with prescribed requirements are allowed reductions in stormwater management requirements when they use techniques to reduce stormwater runoff at the site.

STORMWATER MANAGEMENT PERMIT (SMP): A permit issued by the *[Stormwater Authority]*, after review of an application, plans, calculations, and other supporting documents, which is designed to protect the environment of the Town from the deleterious affects of uncontrolled and untreated stormwater runoff.

STORMWATER UTILITY: A special assessment district set up to generate funding specifically for stormwater management. Users within the district pay a stormwater fee, and the revenue thus generated directly supports maintenance and upgrade of existing storm drain systems; development of drainage plans, flood control measures, and water-quality programs; administrative costs; and sometimes construction of major capital improvements.

3.0 AUTHORITY

This Bylaw is adopted under authority granted by the Home Rule Amendment of the Massachusetts Constitution, the Home Rule statutes, and pursuant to the regulations of the federal Clean Water Act found at 40 CFR 122.34, and as authorized by the residents of the Town of *[_____]* at Town Meeting, dated *[_____]*.

4.0 ADMINISTRATION

A) The *[Stormwater Authority]*, shall administer, implement and enforce this Bylaw. Any powers granted to or duties imposed upon the *[Stormwater Authority]* may be delegated in writing by the *[Stormwater Authority]* to its employees or agents.

B) Stormwater Regulations. The *[Stormwater Authority]* may adopt, and periodically amend, rules and regulations relating to the terms, conditions, definitions, enforcement, fees (including application, inspection, and/or consultant fees), procedures and administration of this Stormwater Management Bylaw by majority vote of the *[Stormwater Authority]*, after conducting a public hearing to receive comments on any proposed revisions. Such hearing dates shall be advertised in a newspaper of general local circulation, at least seven (7) days prior to the hearing date. After public notice and public hearing, the *[Stormwater Authority]* may promulgate rules and regulations

to effectuate the purposes of this Bylaw. Failure by the *[Stormwater Authority]* to promulgate such rules and regulations or a legal declaration of their invalidity by a court shall not act to suspend or invalidate the effect of this Bylaw.

C) Stormwater Management Manual. The *[Stormwater Authority]* will utilize the policy, criteria and information including specifications and standards of the latest edition of the Massachusetts Stormwater Management Policy, *[or approved local equivalent]*, for execution of the provisions of this Bylaw. This Policy includes a list of acceptable stormwater treatment practices, including the specific design criteria for each stormwater practice. The Policy may be updated and expanded periodically, based on improvements in engineering, science, monitoring, and local maintenance experience. Unless specifically altered in the Stormwater Regulations, stormwater management practices that are designed, constructed, and maintained in accordance with these design and sizing criteria will be presumed to be protective of Massachusetts water quality standards.

D) General Permit. The *[Stormwater Authority]* shall have the authority to develop a General Stormwater Management Permit (GSMP) for specific types of projects, such as, without limitation Construction of a *[Deck, Patio, Retaining Wall, Existing Driveway Expansion, Shed, Swimming Pool, Tennis or Basketball Court]*. Any such General Stormwater Management Permit Requirements shall be defined and included as part of any Stormwater Regulations promulgated as a result of this Bylaw.

E) Actions by the *[Stormwater Authority]*. The *[Stormwater Authority]* may take any of the following actions as a result of an application for a Stormwater Management Permit as more specifically defined as part of Stormwater Regulations promulgated as a result of this Bylaw: Approval, Approval with Conditions, Disapproval, or Disapproval without Prejudice.

F) Appeals of Action by the *[Stormwater Authority]*. A decision of the *[Stormwater Authority]* shall be final. Further relief of a decision by the *[Stormwater Authority]* made under this Bylaw shall be reviewable in the Superior Court in and action filed within *[60 days]* thereof, in accordance with M.G.L. Ch 249 § 4.

G) Stormwater Credit System. The *[Stormwater Authority]* may adopt, through the Regulations authorized by this Stormwater Management Bylaw, a Stormwater Credit System. This credit system will allow applicants the option, if approved by the *[Stormwater Authority]*, to take credit for the use of stormwater better site design practices to reduce some of the requirements specified in the criteria section of the Regulations. Failure by the *[Stormwater Authority]* to promulgate such a credit system through its Regulations or a legal declaration of its invalidity by a court shall not act to suspend or invalidate the effect of this Bylaw.

H) Stormwater Utility. The *[Stormwater Authority]* may adopt, through the Regulations authorized by this Stormwater Management Bylaw, a Stormwater Utility pursuant to M.G.L. Chapter 83 Section 16 and Chapter 40 Section 1A. The *[Stormwater Authority]* shall administer, implement and enforce this Utility. Failure by the *[Stormwater Authority]* to promulgate such a Stormwater Utility through its Regulations or a legal declaration of its invalidity by a court shall not act to suspend or invalidate the effect of this Bylaw.

5.0 APPLICABILITY

A) This bylaw shall be applicable to all new development and redevelopment, including, but not limited to, site plan applications, subdivision applications, grading applications, land use conversion applications, any activity that will result in an increased amount of stormwater runoff or pollutants flowing from the a parcel of land, or any activity that will alter the drainage characteristics of a parcel of land, unless exempt pursuant to Section 5.B) of this Bylaw. All new development and redevelopment under the jurisdiction of this Bylaw as prescribed in this Bylaw

shall be required to obtain a Stormwater Management Permit.

B) Exemptions

No person shall alter land within the Town of *[_____]* without having obtained a Stormwater Management Permit (SMP) for the property with the following exceptions:

1. Any activity that will disturb an area less than *[5000]* square feet or less than *[25%]* of a contiguous property, whichever is less. This exception may not be applied for contiguous properties held in common ownership at the time of adoption of this Bylaw that may have been previously subdivided and/or are attributed to multiple separate owners;

   Another option could be based on impervious area such as "Any activity that will increase a contiguous impervious area of less than [5000] square feet.

2. Normal maintenance and improvement of land in agricultural use as defined by the Wetlands Protection Act regulation 310 CMR 10.04 and MGL Chapter 40A Section 3.

3. Maintenance of existing landscaping, gardens or lawn areas associated with a single family dwelling;

4. Repair or replacement of an existing roof of a single-family dwelling;

5. The construction of any fence that will not alter existing terrain or drainage patterns;

6. Construction of utilities (gas, water, electric, telephone, etc.) other than drainage, which will not alter terrain, ground cover, or drainage patterns;

7. Emergency repairs to any stormwater management facility or practice that poses a threat to public health or safety, or as deemed necessary by the *[Stormwater Authority]*;

8. Any work or projects for which all necessary approvals and permits have been issued before the effective date of this Bylaw;

9. Redevelopment projects are presumed to meet the specified stormwater management requirements described in the Stormwater Regulations of the Town of *[_____]* if the total impervious cover is reduced by *[40%]* from existing conditions. Where site conditions prevent the reduction in impervious cover, stormwater management practices shall be implemented to provide stormwater controls for at least *[40%]* of the site's impervious area. When a combination of impervious area reduction and stormwater management practice implementation is used for redevelopment projects, the combination of impervious area reduction and the area controlled by a stormwater management practice shall equal or exceed *[40%]*.

10. An alteration, redevelopment, or conversion of land use to a hotspot such as, without limitation: auto salvage yards, auto fueling facilities, fleet storage yards, commercial parking lots with high intensity use, road salt storage areas, commercial nurseries and landscaping, outdoor storage and loading areas of hazardous substances, or marinas, shall require a Stormwater Management Permit.

6.0 PROCEDURES

Permit Procedures and Requirements shall be defined and included as part of any rules and regulations promulgated as permitted under Section 4 of this Bylaw.

7.0 ENFORCEMENT

The *[Stormwater Authority],* or an authorized agent of the *[Stormwater Authority]* shall enforce this Bylaw, regulations, orders, violation notices, and enforcement orders, and may pursue all civil and criminal remedies for such violations. Enforcement shall be further defined and included as part of any Stormwater regulations promulgated as permitted under Section 4 of this Bylaw.

8.0 SEVERABILITY

The invalidity of any section, provision, paragraph, sentence, or clause of this Bylaw shall not invalidate any section, provision, paragraph, sentence, or clause thereof, nor shall it invalidate any permit or determination that previously has been issued.

Appendix L: Structural BMP Recommendation Letter

June 15, 2005

Ms. Julie McMahon
13 Applecart Road
Rockport, MA 01966

Re: Lower Mill Brook Nonpoint Source Assessment & Management Project
Structural BMP Implementation

Dear Ms. McMahon:

Based on the results of the Nonpoint Source Assessment (Task II) conducted by the Town of Rockport, as described in the above mentioned project Scope of Services, Woodard & Curran (W&C) is pleased to provide you with recommendations for structural BMPs (Best Management Practices) that will provide water quality improvements to the Lower Mill Brook. Attachment A contains the Structural BMPs Locations Map. W&C recommends a phased approach to implementing structural BMPs in the Lower Mill Brook watershed based on available funding and Town staffing resources. The phases are ordered from lowest to highest estimated cost and ease of implementation.

Phase I – Retrofit Existing Catch Basins with Hoods

W&C recommends that all catch basins in the Lower Mill Brook watershed area be equipped with hoods. A hood is an inverted elbow or similar device that is installed over the outlet pipe. It is designed to retain oils and other floatables within the catch basin sump to prevent them from flowing downstream to the storm drain outfall.

One option is to use SNOUT® by Best Management Products, Inc. The SNOUT® Oil & Debris Stop is a plastic composite hooded outlet cover that attaches to the wall of the catch basin over the outlet pipe. It is cost effective and easy to install and maintain. Refer to Attachment B for product literature and details.

Goal: To prevent floating debris and oil from exiting a catch basin.

Design Considerations:

- Sizes should be based on outlet pipe diameter. Refer to Attachment B for manufacturer recommended sizing.

Expected Efficiency:

- Stop up to 95% of floatables while offering significant reductions of free oils and suspended solids
- The average TSS removal rate for deep sump (minimum of 4ft) and hooded catch basins is 25% with cleanout per the Massachusetts Stormwater Policy Handbook (March 1997).

Estimated Unit Cost: Dependent on outlet pipe size ($220 and up). See Attachment B for retail prices on various sizes. *Additional costs may include design and installation by Town staff or a hired contractor.*

Phase II – Catch Basin Sumps Minimum of Four (4) Feet Deep

W&C recommends, at a minimum, that the sump of all catch basins in the Lower Mill Brook watershed be at least 4 feet deep. As part of the NPS Assessment, the total depth of the catch basins was measured, but not the depth of the sump. The sump is the distance from the invert of the outlet pipe to the bottom of the catch basin. The total depth recorded for the following catch basins was four feet or less and should be targeted for increased sump depth: CB-2, CB-3, CB-16, CB-17, CB-19, CB-23, CB-30, CB-35, CB-37, and CB-38. These catch basins are highlighted on the Structural BMPs Locations Map. However, since some of these catch basins may not be "true" catch basins (e.g., culverts or infiltrating catch basins), these structural improvements should be implemented with approval from the Rockport Department of Public Works (DPW).

Goal: To remove debris, sediment, and hydrocarbons from stormwater runoff throughout the watershed. Sedimentation is a significant concern at Mill Pond and at various locations along the Lower Mill Brook. Likewise, trash, debris, and evidence of hydrocarbons have been observed in the brook.

Design Considerations:

- Deep sump volume general rule = 4 x pipe diameter or 4.0' for pipes 18" or less
- The discharge point is located at least 4 feet below the inflow point.
- Guidance for Deep Sump Catch Basins can be found on the MA DEP website: http://www.mass.gov/dep/brp/stormwtr/files/swmpolv2.pdf.

Expected Efficiency:

- The average TSS removal rate for deep sump (minimum of 4ft) and hooded catch basins is 25% with cleanout per the Massachusetts *Stormwater Policy Handbook* (March 1997).

Estimated Unit Cost: Approximately $5,000 to furnish and install one (1) new precast concrete catch basin with a 4-foot sump, including removal and disposal of existing catch basins, excavation, and backfill. *This cost estimate is based on recent bids for similar work in Lowell, MA. Additional costs may include design and site survey.*

Phase III – Erosion Control as Needed from Pooles Lane to Railroad Avenue

W&C recommends implementing erosion control as needed along Mill Brook from Pooles Lane toward Railroad Avenue, as indicated on the Structural BMPs Locations Map. Mill Brook's natural streambanks have eroded in this area.

W&C recommends using "soil bioengineering" techniques, such as BioLog® or BioD-Roll™. These products provide an environmentally sound technique for the restoration of Mill Brook's streambanks. Natural biodegradable products are used to provide the needed protection until establishment of mature vegetation, which will resist erosion forces. These methods not only provide strong, sustainable, aesthetically pleasing water edges, but also provide habitats for aquatic & terrestrial wildlife and filter run off water.

Design and cost information provided in this report is based on RoLanka's BioD-Roll™, which is made of biodegradable coir (coconut fiber). Product literature is provided in Attachment C. More information is available at https://protected11.purehost.com/d30053026/index.asp?pg=coirroll.

Goal: Prevent erosion by stabilizing stream banks and establishing natural vegetation.

Design Considerations:

- Coir rolls are 10-feet long and easy to handle at restoration sites.
- Coir rolls can be joined at the site using coir twine.
- Dry coir rolls have more strength and durability than vegetated coir rolls. High cost of transportation and difficulty in handling of the vegetated coir rolls make restoration with vegetated coir rolls more expensive and less practical.
- High density coir rolls last longer providing erosion protection, thereby allowing sufficient time to establish sustainable vegetation in field conditions.
- Specifications and installation instructions are provided in Attachment C.

Expected Efficiency:

- The densely packed coir rolls last over 5 years providing erosion resistance and supporting establishment of sustainable vegetation.

Estimated Unit Cost: BioD-Roll 30: Coir rolls (set of three (3)) including shipping is $288.00.

BioD-Twine: Two (2) spools for $24.00.

Pine Wedges: Eighteen (18) wedges, 24 in. long for $22.00.

Additional costs may include installation by Town staff or a hired contractor. Please note that these are unit costs. Actual quantities should be determined as part of the design phase.

Phase IV – Drainage System Modifications where Catch Basin Sump is Open to Brook

The following five (5) "catch basins" do not have sumps, rather they discharge through the street directly into Mill Brook: CB-4, CB-24, CB-25, CB-33, and CB-34. These catch basins are shown on the Structural BMPs Locations Map. In some cases, inlet pipes were also observed in the catch basins, indicating that additional drainage discharges to the brook at this point. W&C recommends that these catch basins be relocated to one side of Mill Brook. These catch basins, once relocated, should meet the requirements of a hooded deep sump catch basin as described in this letter and in the Massachusetts *Stormwater Policy Handbook* (Marcy 1997). Please note that W&C can not make recommendations as to how these modifications will alter the drainage system due to limitations of the Nonpoint Source Assessment. For instance, topography of the drainage area and connectivity of the drainage system should be reviewed as part of the design phase in implementing these modifications.

Goal:	Provide treatment of surface runoff on Pooles Lane, Railroad Avenue, and Beach Street where previously stormwater runoff entered Mill Brook directly.
Design Considerations:	• Further investigation is needed to determine the drainage system connectivity surrounding these catch basins and to determine the appropriate number and location of new drainage structures.
Expected Efficiency:	• The average TSS removal rate for deep sump (minimum of 4ft) and hooded catch basins is 25% with cleanout per the Massachusetts *Stormwater Policy Handbook* (March 1997).
Estimated Unit Cost:	See costs associated with Phase I and Phase II.

Phase V – Vortechnics® Stormwater Treatment Systems

W&C recommends that a Vortechs® model stormwater treatment unit be installed in an off-line configuration on the culvert portions of the stream, as indicated on the Structural BMPs Locations Map. W&C chose Vortechnics® instead of other brands of water quality BMPs because the Town of Rockport has already installed Vortechs® units in another area of Town. Town staff is familiar with the operation and maintenance required for these BMPs.

This unit removes contaminated sediment, floating hydrocarbons, and debris from stormwater through the use of centrifugal force and gravity settling. It is also equipped with baffles for flow control to eliminate turbulence and provide positive removal efficiencies throughout the full range of operation. The Vortechs® unit can achieve up to 80% total suspended solids removal. Refer to Attachment D for product literature including details and maintenance. Information on the Vortechs can also be found on the EPA website at http://www.epa.gov/NE/assistance/ceitts/stormwater/techs/vortechs.html.

Goal:	Remove sediment, floating hydrocarbons, and debris from Mill Brook and its tributaries, which act as Rockport's MS4.
Design Considerations:	• Proper sizing calculations of the Vortechs® should be performed and coordinated with the manufacturer. • Off-line installations generally provide the most cost effective design.
Expected Efficiency:	• 80% TSS removal with proper sizing and maintenance.
Estimated Unit Cost:	Budget costs for the two largest models (Vortechs® 11,000 and 16,000) range from approximately **$40,000 to $50,000**. *Additional costs include design and installation by a hired contractor.*

Other Considerations

Many of the recommended structural BMPs may be considered a disturbance within the buffer zone of a resource area and may require filing a Notice of Intent (NOI) application with the Rockport Conservation Commission and the MA DEP in accordance with the Wetlands Protection Act. For example, the installation of a Vortechs unit on the stream culvert would be considered such a disturbance. At a minimum, a Request for Determination of Applicability (RDA) application should be submitted to the local Conservation Commission and the MA DEP when appropriate.

W&C recommends further investigation before implementing structural BMPs. It will likely be necessary to perform an existing conditions survey, including site topography and invert elevations. The connectivity of the drainage system within the Lower Mill Brook watershed should be delineated, including drainage manholes, pipe flow direction, and pipe sizes. The drainage sub-watersheds will need to be redelineated to perform the watershed analysis required for the design of structural BMPs.

The Town of Rockport, in addition to maintaining and improving the municipal drainage system, should work with the owners of private drainage structures to ensure adequate operation and maintenance. For example, there are a number of drainage structures on private property that discharge to Mill Brook between Pooles Lane and Railroad Avenue. This area of Mill Brook is especially prone to heavy sedimentation. As part of Rockport's Stormwater Management Plan written by W&C in July 2003 and as required by the National Pollution Discharge Elimination System (NPDES) Small MS4 General Permit, the Town should be in the process of developing a post-construction runoff bylaw. Such a bylaw would grant the Town authority to require and enforce maintenance of BMPs on private property and to develop an inspection schedule.

Please feel free to contact us at 978.557.8150 should you have any questions.

Very truly yours,
WOODARD & CURRAN INC.

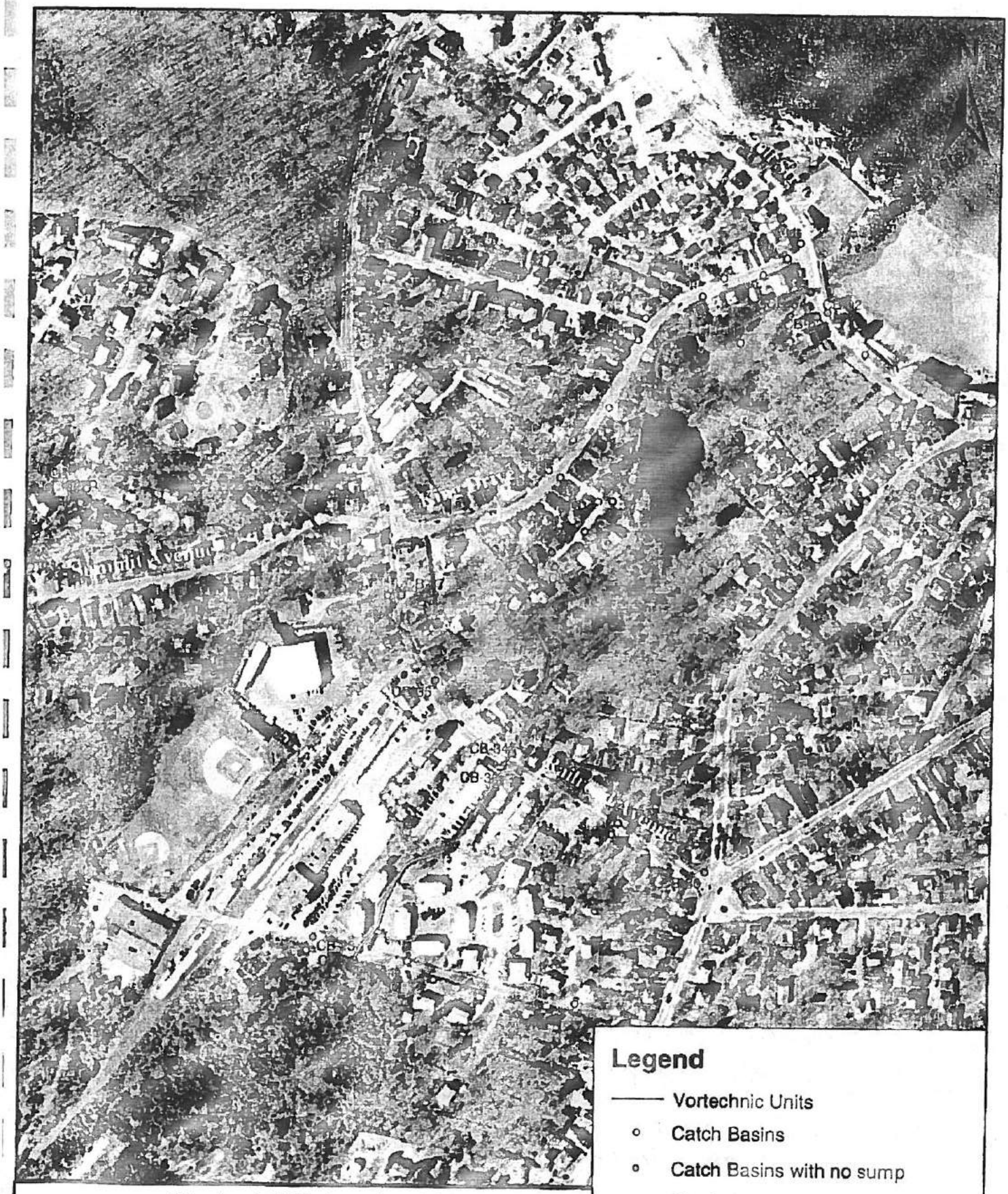
Ronald St. Michel, P.E.
Project Manager

Emily Ferrazza
Engineer

EJF/ejf
210095

Enclosures

cc: Ms. Julie Conroy, Massachusetts Office of Coastal Zone Management



Legend

- Vortechnic Units
- Catch Basins
- Catch Basins with no sump
- Catch Basins to deepen sumps
- Underground Stream
- Stream
- Streambank Stabilization (as needed)

Structural BMPs Locations Map
Lower Mill Brook NPS Assessment & Management Project
June 2005

WOODARD & CURRAN
Engineering · Science · Operations

GEI Dam Improvements

Adam Sea & Architect lead
413 241 6920

- Restore wetlands w/out losing open space
- Get rid of "sluiceway character of channel" - series of small ponds, more away from bldg
- Replace culvert?? We advise

1702 Grist
Lumber
Leather Sort
Isseneglass

Mill · Isseneglass for bleaching
· bldg empty @ 1930's
1830's — quarrying begins

- Parking lot drains to trail.
- "Swamp" by stream can be removed
- Remove great willows new shade?

Preservation act. Historic & Park.

MILONE & MAC BROOM, INC. Inter-Office Memo

TO: File

FROM: Jenabay Sezen

DATE: 7/11/2014

RE: 5220-02 Millbrook Pond and Mill Pond Tidal Elevations

The information below is a compilation of tidal data collected for the Rockport, Massachusetts area.

The tidal benchmarks in Table 1 were collected from the U.S. Department of Commerce National Oceanic and Atmospheric Administration, National Ocean Service (http://tidesandcurrents.noaa.gov/benchmarks.html?id=8441551). The information was provided in MLLW datum in meters and converted to NAVD88 in feet using the National Geodetic Survey elevation information for Station ID 8441551 (http://www.ngs.noaa.gov/Tidal_Elevation/diagram.jsp?PID=MY0231&EPOCH=1983-2001).

TABLE 1
TIDAL ELEVATIONS

		Elevation (Feet, NAVD88)	Elevation (Feet, MLLW)
(MHHW)	Mean Higher High Water	4.5	2.9
(MHW)	Mean High Water	4.1	2.8
(MSL)	Mean Sea Level	-0.2	1.4
(MTL)	Mean Tide Level	-0.3	1.4
(MLW)	Mean Low Water	-4.6	0.1
(MLLW)	Mean Lower Low Water	-4.9	0.0

The information in Table 2 was collected from the NOAA Tides and Currents website. Predicted tidal data for Station ID 8441551 was downloaded for all of 2014 and converted from MLLW datum in feet to NAVD88 datum in feet using the National Geodetic Survey information for Station ID 8441551. Minimum, maximum, and mean tidal elevations are shown in Table 2.

WWTP Precip Station Monthly Rain Fall Totals

DMR Reporting Month	Total Monthly Rain Fall
Jan-07	3.19
Feb-07	3.57
Mar-07	6.18
Apr-07	7.98
May-07	3.69
Jun-07	3.75
Jul-07	2.15
Aug-07	0.78
Sep-07	3.06
Oct-07	3.32
Nov-07	2.90
Dec-07	6.40
Jan-08	3.17
Feb-08	9.08
Mar-08	6.01
Apr-08	4.50
May-08	1.74
Jun-08	3.29
Jul-08	5.15
Aug-08	3.55
Sep-08	7.59
Oct-08	1.37
Nov-08	4.17
Dec-08	5.92
Jan-09	3.85
Feb-09	2.82
Mar-09	4.59
Apr-09	4.12
May-09	3.12
Jun-09	4.73
Jul-09	9.62
Aug-09	3.34
Sep-09	2.66
Oct-09	6.89
Nov-09	3.71
Dec-09	5.66
Jan-10	4.55
Feb-10	4.46
Mar-10	13.90
Apr-10	2.20
May-10	2.55
Jun-10	3.19

Jul-10	1.70
Aug-10	6.26
Sep-10	1.93
Oct-10	4.38
Nov-10	4.29
Dec-10	2.83
Jan-11	4.58
Feb-11	5.55
Mar-11	2.20
Apr-11	6.14
May-11	3.11
Jun-11	3.74
Jul-11	2.89
Aug-11	9.47
Sep-11	4.45
Oct-11	7.27
Nov-11	3.66
Dec-11	3.88
Jan-12	3.71
Feb-12	1.73
Mar-12	2.02
Apr-12	3.40
May-12	4.87
Jun-12	5.98
Jul-12	1.59
Aug-12	3.50
Sep-12	5.05
Oct-12	2.90
Nov-12	1.49
Dec-12	7.03
Jan-13	1.33
Feb-13	5.96
Mar-13	2.70
Apr-13	1.61
May-13	3.48
Jun-13	9.63
Jul-13	2.68
Aug-13	2.01
Sep-13	2.04
Oct-13	1.01
Nov-13	3.08
Dec-13	5.48
Jan-14	3.63
Feb-14	5.02
Mar-14	4.80

TABLE 2
2014 PREDICTED TIDAL ELEVATIONS

	Elevation (Feet, NAVD88)	Elevation (Feet, MLLW)
Min	-6.9	-2.0
Max	6.3	11.2
Mean	-0.2	4.7

Table 3 includes information from the FEMA Updated Tidal Profiles for the New England Coastline from March 2012 (http://www.fema.gov/media-library-data/1383243270931-eed96b0ca96641e6c0200ac996b63b7a/220602416_New_England_Tide_Report_2012_0326_FINAL.pdf). Tidal Profile 11 was used at mile 175 to find Mean Annual Max, 10% Annual chance, 2% Annual Chance, 1% Annual Chance, and 0.2% Annual Chance data in NAVD88 in feet.

TABLE 3
STORM EVENT ELEVATIONS FROM TIDAL PROFILE

	Elevation (Feet, NAVD88)
Mean Annual Max	7.5
10% Annual Chance	8.3
2% Annual Chance	9.3
1% Annual Chance	9.7
0.2% Annual Chance	11.2

Drive Joe: Cell 207-217-7411
7:00

Mill Pond and
Millbrook Meadow Restoration
Rockport, Massachusetts

Date	Time	Gage #1 Reading (ft)	Gage #1 Elevation (ft, NAVD88) (Reading + 7.8)	Gage #2 Reading (ft)	Gage #2 Elevation (ft, NAVD88) (Reading + 10.0)	Gage #3 Reading (ft)	Gage #3 Elevation (ft, NAVD88) (Reading + XX)	Name	Comments
4/28/14	12:15	1.55	9.35	1.94	11.94				
5/1/14	20:00	0.88	8.68	1.56	11.56				
6/4	9:15	1.64		1.84					7-9 am showers
6/6	8:30am	.88		1.58					overnight showers/rain

Gage #1 (Near Culvert) Invert Elevation = 7.8 feet NAVD88

Gage #2 (Near Footbridge) Invert Elevation = 10.0 feet NAVD88

Gage #3 (Mill Pond) Invert Elevation = XX feet NAVD88

MILONE & MACBROOM®

Receiving Stream: SANDY BAY
Plant ROCKPORT WWTP
Town or City ROCKPORT MASS
Month MARCH 2014
Chief Operator LARRY WINSON
Assistant Chief Operator RON HALE

DEPARTMENT OF ENVIRONMENTAL QUALITY ENGINEERING
DIVISION OF WATER POLLUTION CONTROL

MONTHLY OPERATION AND MAINTENANCE REPORT

Region ONE
County ESSEX
If Contract Operation, Company

Permit # State MA Fed. 0100145

Plant Design Flow 0.80 mgd.
Monthly Average Flow 0.81 mgd.
Monthly Average Flow Last Year 0.99 mgd.

Chief Operator Certification of Report Larry Winson Date 04/09/2014

COMPLETE STAFFING PLAN MUST BE SUBMITTED IN JUNE AND JANUARY OF EACH YEAR, OR WHEN ANY PERSONNEL CHANGES OCCUR.

This Plant was in Complete Permit Compliance this Month YES NO

	Weather			Sewage Flows (mgd)				grit & scr.	Disinfection		Settleable Solids				pH influent		pH effluent	
date	rain 1	temp 2	inf temp 3	max 3	min 4	total 6	bypass 7	cu.ft. 8	#/24 hrs 9	Cl_2 resid 10	raw 11	pri eff 12	sec eff 13	final 14	hi 15	low 16	hi 17	low 18
1	0	14		1.33	0.38	.760			8.0									
2	0	32		1.40	0.36	.704			8.0									
3	0	19	46	1.31	0.30	.726			8.0	.26	16.0			0.0	7.13		6.60	
4	0	14	45	1.90	0	.640			8.0	.25	10.5			0.0	7.11		6.59	
5	.07	26	46	1.89	0	.609		27.0	7.0	.24	12.0			0.0	7.09		6.60	
6	0	16	47	1.89	0	.595			8.0	.26	26.0			0.0	7.13		6.61	
7	0	14	47	1.12	0.30	.614			8.0	.26	23.0			0.0	7.10		6.60	
8	0	27		1.20	0.30	.595			7.0									
9	.05	26		1.28	0.24	.616			8.0									
10	.01	27	47	1.13	0.27	.584			8.0	.26	12.5			0.0	7.13		6.60	
11	0	34	48	1.13	0.29	.605			7.0	.26	16.0			0.0	7.15		6.63	
12	.71	38	48	1.43	0.49	.771		27.0	8.0	.26	28.0			0.0	7.15		6.60	
13	.04	23	46	1.40	0.48	.832			7.0	.25	19.0			0.0	7.14		6.59	
14	0	13	47	1.36	0.40	.735			8.0	.26	10.5			0.0	7.10		6.58	
15	.02	38		1.37	0.39	.769			8.0									
16	0	26		1.38	0.40	.743			7.0									
17	0	16	46	1.90	0	.677			9.0	.26	16.0			0.0	7.11		6.58	
18	0	24	47	1.91	0	.640			8.0	.26	29.0			0.0	7.13		6.62	
19	.36	30	47	1.88	0	.649		27.0	8.0	.26	16.0			0.0	7.15		6.66	
20	0	40	47	1.92	0	.725			8.0	.25	12.0			0.0	7.17		6.68	
21	0	33	47	1.88	0	.694			8.0	.25	23.0			0.0	7.15		6.60	
22	.01	27		1.90	0	.682			8.0									
23	0	33		1.93	0	.673			4.0									
24	0	16	47	1.90	0	.598			8.0	.24	17.0			0.0	7.11		6.59	
25	0	18	46	1.88	0	.575			8.0	.26	12.0			0.0	7.09		6.57	
26	0	30	46	1.40	0.10	.570			8.0	.26	14.5			0.0	7.11		6.59	
27	.01	22	47	1.13	0.23	.550			7.0	.25	29.0			0.0	7.13		6.60	
28	.03	37	48	1.09	0.20	.534			8.0	.25	12.0			0.0	7.14		6.63	
29	2.25	44		2.78	0.35	.847			8.0									
30	1.23	40		5.10	1.38	3.397			8.0									
31	.01	33	46	3.13	1.39	2.238			8.0	.06	1.0			0.0	7.09		6.99	
TOTAL	4.80					24.947		81.0	239.0									
AVE.	.15	27	47	1.75	0.27	.805		27.0	7.7	.25	16.9			0.0	7.12		6.62	

PRELIMINARY HYDROLOGIC AND HYDRAULIC ANALYSIS REPORT

MILLBROOK MEADOW
ROCKPORT, MASSACHUSETTS

September 2, 2014

MMI #5220-02-3

Prepared for:

Millbrook Meadow Committee

Prepared by:

MILONE & MACBROOM, INC.
99 Realty Drive
Cheshire, CT 06410
(203) 271-1773
www.miloneandmacbroom.com

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1.0 INTRODUCTION

Milone & MacBroom, Inc. has been engaged by the Millbrook Meadow Committee to evaluate restoration of Millbrook Meadow, a coastal town park located in Rockport, Massachusetts. A small perennial watercourse traverses the park and discharges to the Atlantic Ocean through a culvert located at the park entrance. Potential improvements to the channel and culvert will be investigated as part of an initial feasibility study to be submitted separately. This report presents a preliminary analysis of the hydrology and hydraulics related to Mill Brook and the Millbrook Meadow restoration.

Section 2.0 provides a description of the existing conditions of the site as they relate to the hydrologic and hydraulic aspects of meadow elements, including the Mill Pond and Mill Pond Dam, the Frog Pond weir, the granite-lined brook, the park, and the Beach Street culvert. Each of these plays a role in the behavior and movement of water through the Millbrook Meadow site.

Section 3.0 provides a discussion of site hydrology. Hydrology is the science concerned with the distribution and circulation of water. Hydrology tells us the volume of water delivered to the channel during a storm.

Section 4.0 is a discussion of the hydraulic condition of the site, as well as the predicted hydraulics associated with improvements to the site. Hydraulics is the science associated with the movement of water through the stream channel and culvert.

2.0 EXISTING CONDITIONS

The Millbrook Meadow includes a number of hydraulic features, including the Mill Pond and Mill Pond Dam, the Frog Pond weir, the granite-lined brook, the park, and the Beach Street culvert. Each of these plays a role in the behavior and movement of water through the site.

2.1 Mill Brook Watershed

The contributing watershed area to Mill Brook at the dam is approximately 0.89 square miles, as determined from United States Geological Survey (USGS) *StreamStats* and graphically depicted in Figure 1. The watershed is hilly with minor depressional storage, including Briar Swamp, a historic kettle pond glacial feature. Bedrock geology in the watershed is faulted intrusive rock from the Paleozoic era; the surficial geology consists of enormous intrusive granite either exposed at the surface or with a thin covering of glacial deposits.

The watershed is primarily forested, with a mixture of residential and commercial uses that increase in density moving downstream. See Section 3.0 for a discussion of watershed hydrology.

2.2 Mill Brook Channel

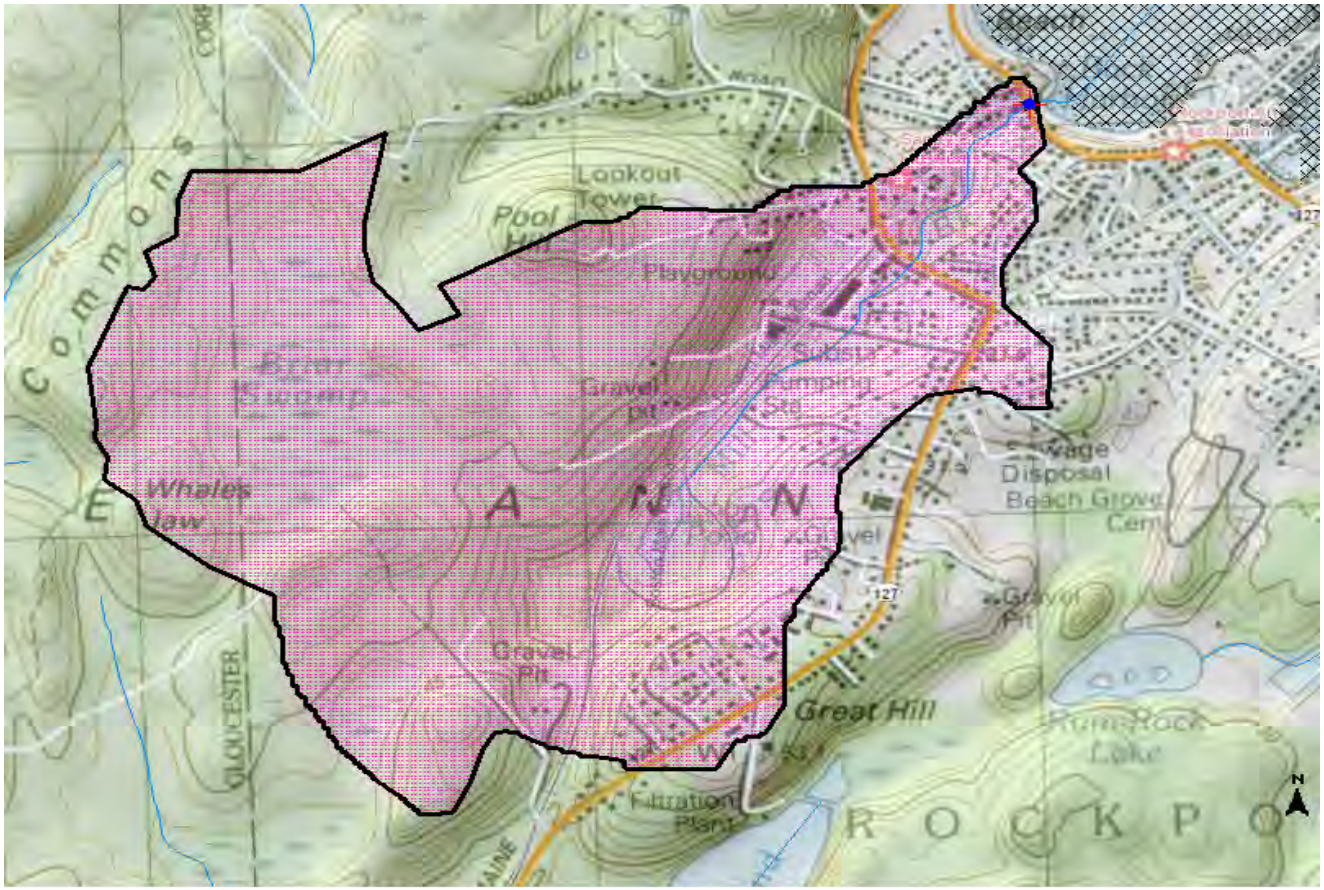
Mill Brook is a small perennial watercourse with its headwaters in Briar Swamp. In just under 1 mile, the channel descends from elevation 45 feet at Briar Swamp to elevation 7 feet at the inlet to the Beach Street culvert. The channel extends 900 feet through the project site, from just downstream of the Mill Brook Dam to the outlet of the Beach Street culvert at tidewater. The channel drops 4 feet in the first 100 feet downstream of the dam, at a slope of 4%. The channel through the remainder of the site is at a moderate average slope of 1%.

Mill Brook channel downstream of dam

Upstream of the Mill Pond, as Mill Brook flows onto the Millbrook Meadow property, the watercourse has a width of approximately 12 feet and a bed

USGS
Massachusetts StreamStats

StreamStats Print Page



Explanation

- NHDHGage
- NHDHDam
- Gaging Station, Continuous Record
- Low Flow, Partial Record
- Peak Flow, Partial Record
- Peak and Low Flow, Partial Record
- Stage Only
- Low Flow, Partial Record, Stage
- Miscellaneous Record
- Unknown
- GlobalWatershedPoint
- Dendritic Stream Network
- GlobalWatershed
- Excludepoly

0.3 0.15 0 0.3 Miles

9/2/2014 8:59:21 AM

substrate consisting of medium to coarse sands. The channel meanders as it discharges onto the Millbrook Meadow property and eventually flows north into Mill Pond. Large woody debris is present within the upper reaches of the brook. Small shallow pools are present. The brook has vegetated banks consisting of upland riparian zone vegetation and emergent marsh vegetation. The brook through this reach runs shallowly across a sandy delta deposit comprised of material transported from upstream, which is deposited as the swift channel flow meets the quiescent Mill Pond.

The channel downstream of the dam was heavily modified during the 2010 dam reconstruction. Large stone riprap has been placed to create a dissipation pool for flow over the spillway, as well as to armor the channel bed and banks. The riprap ranges in size from approximately 3 inches to over 24 inches and extends over 100 feet downstream of the dam. This reach of channel is lacking in fine substrate, with large open voids in the rock. During one site visit, water was observed passing under and through, instead of over, the riprap. This reach lacks suitable aquatic habitat, such as sufficient cover in pools and diversity in character of flow. Epifaunal substrate, which provides refuge, feeding, and spawning sites and nursery functions for aquatic microfauna and includes fallen trees, logs and branches, and undercut banks, is lacking. The coarse substrate precludes the establishment of appropriate bottom-dwelling organisms through this reach.

Looking upstream toward Frog Pond

As the channel begins to flow south, the now undersized channel is constrained by stacked stone and granite block walls and is generally only a few feet wide.

Downstream of the Frog Pond, the channel is almost entirely constrained by granite block walls and is generally 2.5 feet wide by 1 foot deep to the top of the granite block walls. The

cross section is homogenous and lacking in diversity. Channel substrate is silty sand. The presence of filamentous algae indicates a nutrient-rich system, perhaps as a result of nutrient input from fertilization.

Granite block has fallen into the channel, both individual blocks as well as entire sections of wall. The blocks constrict low flows in the channel. Based on anecdotal evidence, the channel banks along the failed wall sections become saturated and soggy following rain events.

The existing channel planform through the meadow is straight and channelized. A 1% slope channel would naturally be sinuous and have sand and gravel substrate with an assortment of habitats, including deeper pools with slow flow and sandy substrate, shallow swift flow over gravel and cobble riffles, long runs with swift flow and relatively coarse substrate, and shallow flow over point bars or sandy deposits.

Looking upstream from Beach Street

2.3 Mill Pond Dam

The Mill Pond Dam is a run-of-river structure that was rebuilt in 2010 following structural failure in 2006. The rebuilt dam consists of a concrete retaining wall with stone masonry facing on the upstream and downstream sides. The original dam had a clay core with hand-laid unmortared stone. The dam runs generally east to west and is approximately 110 feet long.

The crest of the dam is ± elevation 28 feet NAVD88. The downstream channel is at elevation 18 at the dam and 14.3 at the west abutment. The maximum structural height of the dam is 13.7 feet.

Under normal pool conditions, the elevation of the water surface in the impoundment is approximately 25.3 feet NAVD88, with an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Under design flood conditions (1% annual chance flood or 100-year flood), the surface area and volume are approximately 3.4 acres and 10 acre-feet, respectively.

The dam is classified as Small according to the Dam Safety Regulations (302 CMR 10.00) but is considered a Significant Hazard Class II due to proximity to residential and commercial properties, as well as the potential for impact to Beach Street.

The spillway consists of two inlet-controlled granite-block rectangular culverts. Each box is approximately 2.5'H x 2.5'W. The culverts have an invert elevation of 24.8 feet NAVD88. The existing spillway capacity is 130 cubic feet per second (cfs), significantly less than the 1% annual chance spillway design flood of 670 cfs.

The water level in the impoundment can be controlled by adjustment of weirboards at the face of the culverts. The weirboards are currently adjusted to optimize flow to the eel ladder.

TABLE 2-1
Mill Pond Dam

Feature	Detail
Type	Concrete core with stone masonry facing
Length	110 feet
Crest Elevation	±29 feet NAVD88
Spillway	Twin granite block culverts
Spillway Invert Elevation	24.8 feet NAVD88
Size Class[1]	Small
Hazard Class[1]	Significant / Class II
Impoundment Capacity[2]	
Normal Pool	3 acre-feet
Flood Pool	10 acre-feet

[1]According to Dam Safety Regulations (302 CMR 10.00)
[2]As computed by GEI Consultants, Inc.

Computations completed by GEI Consultants, Inc. predict that the dam will overtop by 2 feet for a duration of 13 hours during a 1% annual chance flood event. The dam was therefore designed to safely overtop during the design storm. The redesigned dam included provisions for a 20-foot-wide buffer between the downstream toe of the dam and any woody vegetation, in accordance with dam safety practice.

The dam is a run-of-river structure. Run-of-river dams typically have a small hydraulic head and storage volume, short residence times within the impoundment, and little or no control over the release of water from the impoundment. The storage capacity of Mill Pond during the 1% annual chance flood is approximately 7 acre-feet. This equates to approximately 0.5% of the 1% annual chance flood volume. The reservoir storage is therefore very small and has little to no effect on flooding conditions downstream of the dam.

The dam does not have an operational low-level outlet. A waiver of the requirement for a low-level outlet was submitted to the Office of Dam Safety as part of the dam reconstruction permitting process. The dam is fitted with an eel ladder attached to the downstream face of the dam to allow for passage of American eel.

2.4 Beach Street Culvert

Two primary elements of evaluation are the ability of the culvert to convey appropriate flows through the park, as well as the means by which debris clogging of the culvert can be minimized or avoided.

The Beach Street culvert is constructed of stacked granite block. The downstream face is mortared and appears to have been reconstructed in conjunction with the seawall waterward of Beach Street. The upstream face of the culvert is primarily constructed of unmortared stone. The parapet wall along the bridge is a separate installation of mortared stone.

The culvert has a span of ±5 feet, much narrower than the approach channel upstream of Mill Pond but larger than the block-lined channel immediately upstream of the culvert. The culvert rise is 4.5 feet on the upstream side and 5.5 feet on the downstream side. The total culvert length under Beach Street is 50 feet. The upstream invert elevation is 6.9 feet NAVD88; the downstream invert elevation is 5.5 feet NAVD88. Critical elevations at the Beach Street culvert are provided in Table 2-2.

TABLE 2-2
Beach Street Culvert
Critical Elevations

Location	Elevation (feet, NAVD88)
Beach Street (at culvert)	14.0
Culvert invert	6.9 (upstream)
	5.5 (downstream)
Culvert low chord	11.1 (southwest)
	11.5 (northwest)
	10.7 (southeast)
	11.0 (northeast)

The culvert becomes partially and/or fully clogged with debris during storm events, reducing culvert discharge capacity. Clogging of the culvert is exacerbated by fallen granite block at the face of the culvert, numerous utility crossings within the culvert, and the horizontally irregular stacking of the granite blocks from which the culvert is constructed. Excessive debris clogging is often the result of an undersized culvert.

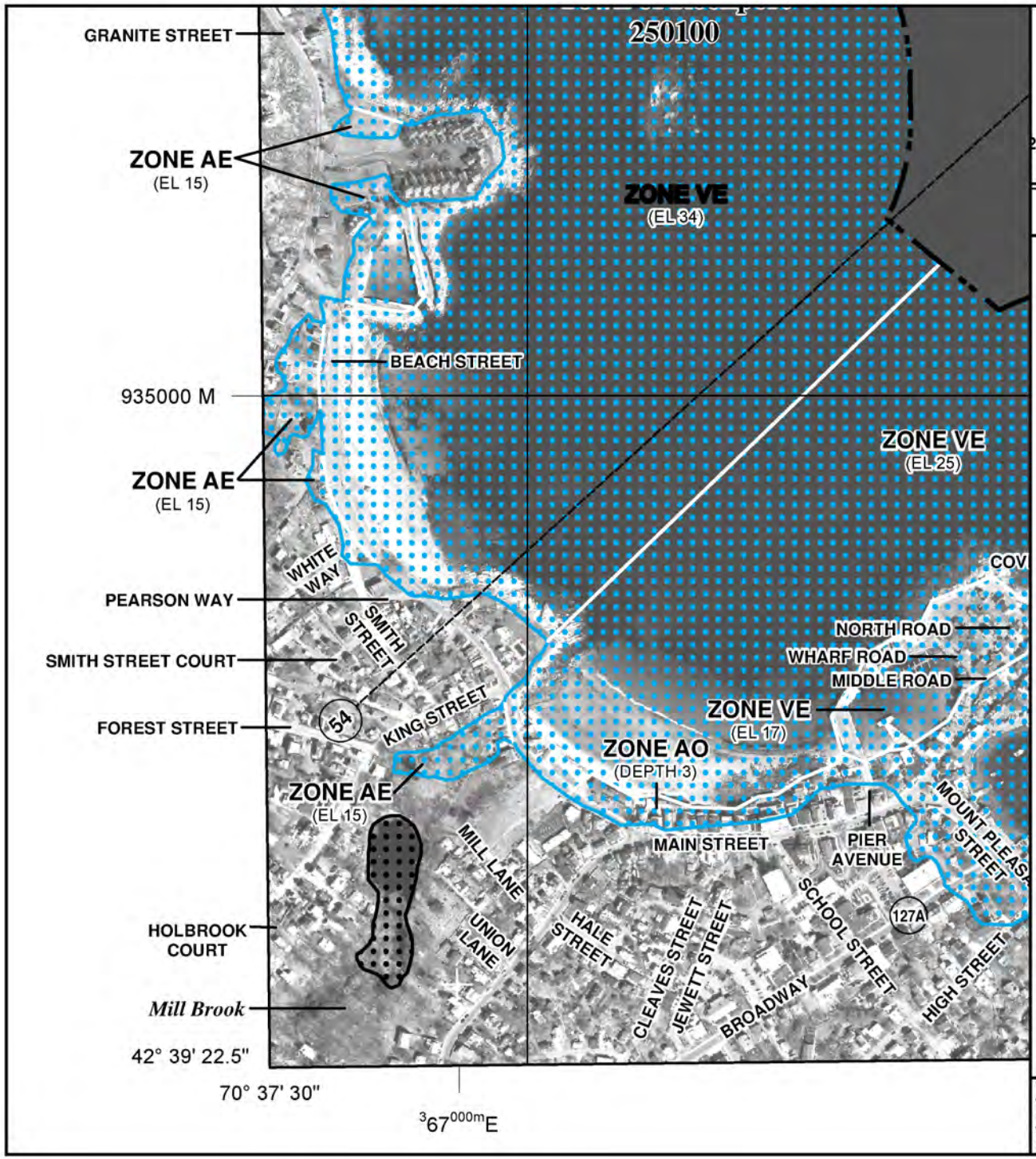
A preliminary structural inspection of the culvert was completed and will be submitted separately. The culvert was determined to be in poor condition.

2.5 FEMA

A Federal Emergency Management Agency (FEMA) Flood Insurance Study (FIS) for the Town of Rockport, Massachusetts was issued December 19, 1984. Tidal flooding and shallow flooding along the shoreline were studied by detailed methods to determine Base Flood elevations in Special Flood Hazard Areas. The study results from this FIS were reissued as part of the Essex County FIS dated July 3, 2012. Coastal hydrologic and hydraulic analyses for the Essex County coastline, including the town of Rockport, were revised and updated in the Essex County FIS issued July 16, 2014.

The Flood Insurance Rate Maps (FIRMs) for Essex County were reissued on July 16, 2014. A Firmette of the project area is included as Figure 2. Frog Pond and much of Millbrook Meadow between Frog Pond and Main Street are designated Zone AE with a Base Flood Stillwater Elevation of 15 feet. A description of FEMA Special Flood Hazard Zones is provided in Table 2-2. Mill Pond and the upstream wetland area were determined to be within FEMA Zone X, the 0.2% annual chance flood zone.

The 1% annual chance flood, also known as the Base Flood, is the flood that has a 1% chance of occurring in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. The Base Flood Elevation is the water surface elevation of the 1% annual chance flood.



MAP SCALE 1" = 500'

250 0 500 1000 FEET METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0336G

FIRM

FLOOD INSURANCE RATE MAP

ESSEX COUNTY, MASSACHUSETTS

(ALL JURISDICTIONS)

PANEL 336 OF 600

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
ROCKPORT, TOWN OF	250100	0336	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
25009C0336G

MAP REVISED
JULY 16, 2014

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

TABLE 2-3
FEMA Flood Hazard Zones

FEMA Zone	Description
AE	Base Flood Elevations determined
AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
VE	Coast flood zone with velocity hazard (wave action); Base Flood Elevations determined.
X	Areas of 2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood

2.6 Tide Data

Existing tidal elevation data available from the National Oceanic and Atmospheric Administration (NOAA), the National Geodetic Survey, and FEMA was reviewed and compiled. The tidal benchmarks in Table 2-4 were collected from the U.S. Department of Commerce NOAA, National Ocean Service (http://tidesandcurrents.noaa.gov/benchmarks.html?id=8441551). The information was provided in Mean Lower Low Water (MLLW) datum in meters and converted to NAVD88 in feet using the National Geodetic Survey elevation information for Station ID 8441551 (http://www.ngs.noaa.gov/Tidal_Elevation/diagram.jsp?PID=MY0231&EPOCH=1983-2001).

TABLE 2-4
Tidal Elevations
Compared to Culvert Elevations

		Elevation (Feet, NAVD88)	Elevation (Feet, MLLW)
(MHHW)	Mean Higher High Water	4.5	2.9
(MHW)	Mean High Water	4.1	2.8
(MSL)	Mean Sea Level	-0.2	1.4
(MTL)	Mean Tide Level	-0.3	1.4
(MLW)	Mean Low Water	-4.6	0.1
(MLLW)	Mean Lower Low Water	-4.9	0.0
	Top of Road	14.0	N/A
	Culvert Invert	5.5 (D/S)	N/A

Notes: D/S = downstream, N/A = not applicable

The information in Table 2-4 was collected from the NOAA Tides and Currents website. Predicted tidal data for Station ID 8441551 was downloaded for all of 2014 and converted from MLLW datum in feet to NAVD88 datum in feet using the National Geodetic Survey information for Station ID 8441551. Minimum, maximum, and mean predicted tidal elevations are shown in Table 2-5.

TABLE 2-5
2014 Predicted Tidal Elevations

	Elevation (Feet, NAVD88)	Elevation (Feet, MLLW)
Minimum	-6.9	-2.0
Maximum	6.3	11.2
Mean	-0.2	4.7

Table 2-6 includes information from the FEMA Updated Tidal Profiles for the New England Coastline from March 2012 (http://www.fema.gov/media-library-data/1383243270931-eed96b0ca96641e6c0200ac996b63b7a/220602416_New_England_Tide_Report_2012_0326_FINAL.pdf). Tidal Profile 11 was used at mile 175 to find Mean Annual Maximum, 10%

Annual Chance, 2% Annual Chance, 1% Annual Chance, and 0.2% Annual Chance data in NAVD88 in feet.

TABLE 2-6
Storm Event Tidal Elevations
FEMA Updated Tidal Profiles
for the New England Coastline

Percent Annual Chance Event	Elevation (Feet, NAVD88)
Mean Annual Maximum	7.5
10% Annual Chance	8.3
2% Annual Chance	9.3
1% Annual Chance	9.7
0.2% Annual Chance	11.2

FEMA has determined stillwater elevations for the entire coastline of Rockport, as presented in the July 16, 2014 Essex County FIS. Table 2-7 provides the stillwater elevations associated with the 10-, 2-, 1-, and 0.2% chance events as documented in the FIS.

TABLE 2-7
Storm Event Tidal Elevations
Comparison of Essex County FIS Dated July 16, 2014 and
FEMA Updated Tidal Profiles
for the New England Coastline

Percent Annual Chance Event	Elevation (Feet, NAVD88)		
	Essex County FIS	FEMA Updated Tidal Profiles	Difference
10% Annual Chance	7.5	8.3	+0.8
2% Annual Chance	8.3	9.3	+1.0
1% Annual Chance	8.6	9.7	+1.1
0.2% Annual Chance	9.2	11.2	+2.0

Tidal water surface elevations for both analyses are based on a statistical frequency analysis of annual maximum tidal heights along the coastline of Essex County, supplemented with high water mark data from significant events in the record. The FEMA Updated Tidal Profiles reference data from 18 long-term stations, with data available through 2007. The FEMA FIS tidal elevations were first presented in the 1984 FIS for the Town of Rockport. Although it is not specifically defined, the FIS must only reference data through 1979, the year the study was completed. The tidal water surface elevations predicted in the FEMA Updated Tidal profiles are significantly higher than those presented in the FEMA FIS. Tidal elevations predicted by the FEMA Updated Tidal Profiles were used in the hydraulic analyses as these elevations are more conservative and are based on more current tidal data.

Tidal data loggers were installed by Milone & MacBroom, Inc. on July 18, 2014. The data loggers will collect water surface elevation in Rockport Harbor and the Mill Brook channel; the location of the data loggers is graphically depicted in Appendix B. The data loggers will be left in place through the fall.

It is apparent that the culvert invert is above the normal mean high water but that the highest tides and all flood tides will extend into and through the culvert. This is not unusual but reduces the culvert flow capacity during these events.

3.0 HYDROLOGY

Peak flow hydrology for the Mill Brook watershed was previously developed by GEI Consultants, Inc. in 2010 for reconstruction of the Mill Pond Dam. Peak flows were developed using TR-20. The results of its analysis are presented in Table 3-1.

TABLE 3-1
2010 Dam Design Flows[1]

Return Frequency, Years	Predicted Peak Flows, cfs[2]
2	233
5	316
10	405
25	520
50	574
100	670

1 As computed by GEI Consultants, 2010

2 cfs = cubic feet per second

The above predicted peak flood flows are high for a watershed of this size. This may be due to the low infiltration rates as a result of shallow bedrock in the watershed, or may be a computational error in the analysis. Seasonal and habitat-forming flows have not been previously developed for Mill Brook. A range of flows was included in the hydraulic model to analyze these lower flow conditions, from 5 cfs to 100 cfs.

Regional hydraulic geometry relationships were used to compute bankfull discharge as 34 cfs. Hydraulic geometry relationships are based on analysis of long-term United States Geological Survey (USGS) stream gaging stations in Massachusetts (Bent, 2013). Hydraulic geometry relationships are a series of regression equations developed from statistical analysis of the gage data that relate bankfull discharge, width, depth, and cross-sectional area to the contributing drainage area. The resulting discharge is normally comparable to or slightly below the 2-year flood frequency.

The above flows are used in the Hydrologic Engineering Center River Analysis System (HEC-RAS) water surface profile Existing and Proposed Conditions models to analyze the Mill Brook channel and Beach Street culvert.

In conclusion, the predicted 2010 design flows for the dam appear to be overly high and should be reconsidered.

4.0 HYDRAULIC ANALYSIS

Hydraulic analysis of Mill Brook was completed using the U.S. Army Corps of Engineers (USACE) HEC-RAS (river analysis software) program (version 4.1). The HEC-RAS computer program entitled *River Analysis System* was written by the USACE Hydrologic Engineering Center and is considered appropriate for riverine flood studies. The model is used to compute water surface profiles for one-dimensional, steady-state, and gradually varied flow. This system can accommodate a full network of channels, a dendritic system, or a single river reach. HEC-RAS is capable of modeling water surface profiles under subcritical, supercritical, and mixed-flow conditions. Appendix A includes a HEC-RAS cross-section location plan.

The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's Equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed-flow regime calculations, hydraulics of dams and bridges, and evaluating profiles at a river confluence.

4.1 Existing Conditions Hydraulic Analysis

Existing Conditions hydraulics were analyzed to determine the existing hydraulic condition within the park. A HEC-RAS model was developed based on topography and bathymetry collected for this project. The HEC-RAS model extends from the base of the dam to Front Beach. A HEC-RAS cross-section location plan is included as Figure 3. HEC-RAS output data is included in Appendix E.

Mill Brook Channel

The existing channel capacity was determined to be approximately 8 cfs. Above 8 cfs, flow will overtop the channel banks and spill out into the park. Figure 4 graphically depicts the predicted water surface profiles through the park under the Existing Condition for 5 cfs, bankfull, and the 20% and 1% annual chance storm events. Predicted velocities during the bankfull event are in the range of 2 to 7 feet per second, depending on location. Velocity markedly increases at the approach to the culvert due to constriction of the flow. Table 4-1 provides predicted water surface elevation and velocity through the project reach for Existing Conditions.

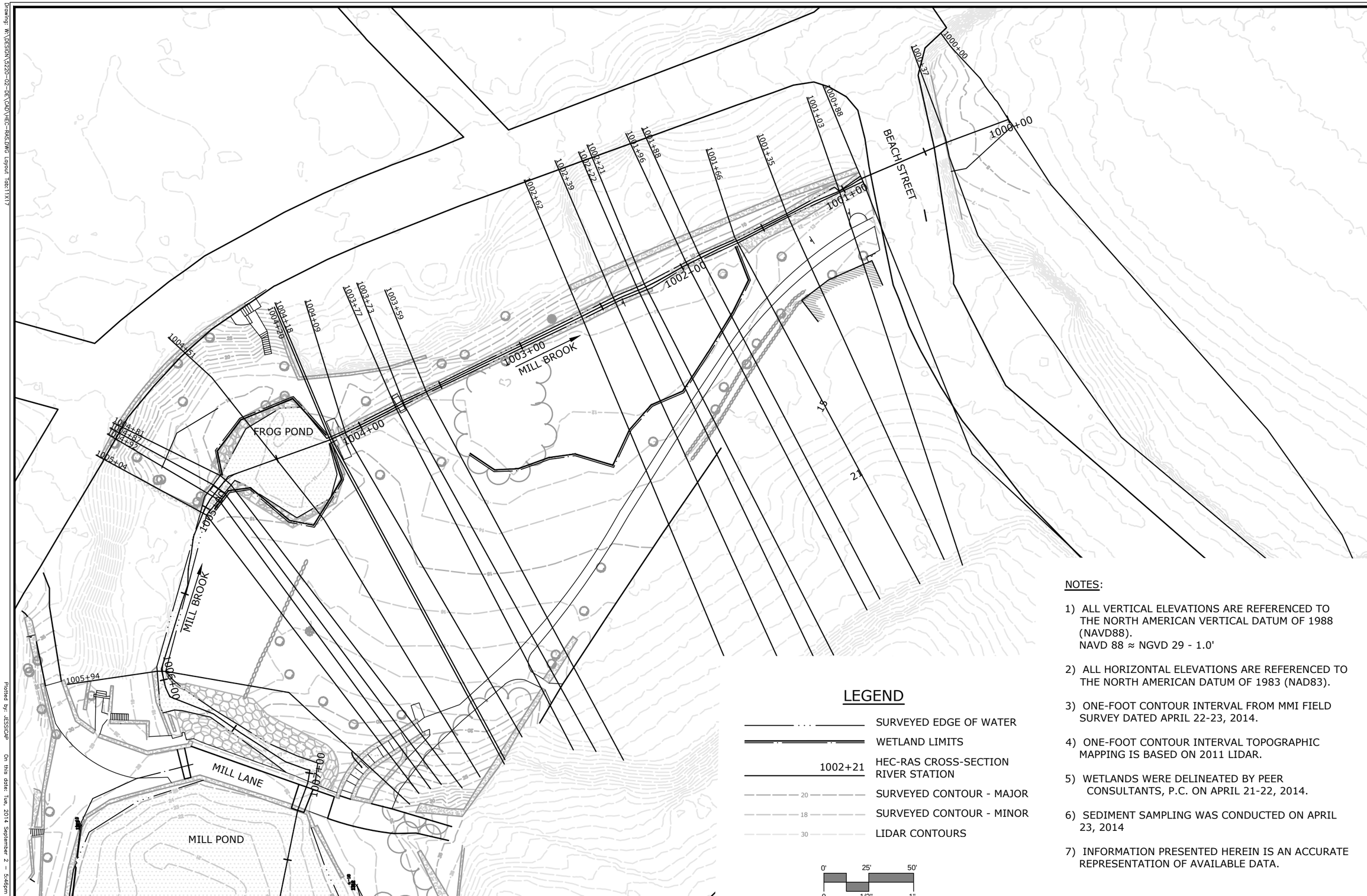
TABLE 4-1
Mill Brook Channel
Predicted Water Surface Elevation and Velocity
Bankfull Event (34 cfs)

River Station	Location	Water Surface Elevation (feet, NAVD88)	Velocity (feet per second)
1000+37	D/S face of Beach Street culvert	6.0	3.5
1000+88	U/S face of Beach Street culvert	7.8	6.6
1002+39	151' u/s of Beach Street	11.3	6.0
1003+59	271' u/s of Beach Street	12.0	2.1
1005+04	25' u/s of Frog Pond	13.9	13.3
1005+94	120' u/s of Frog Pond	15.2	7.8

Notes: d/s = downstream u/s = upstream

Frog Pond Bridge

Frog Pond is supported by a weir at the upstream face of the stone footbridge. The weir appears to be constructed of stacked granite block. The water level in Frog Pond was formerly adjustable through the use of weirboards, as evidenced by the presence of a deteriorated steel rail bolted to the south training wall and bolt holes in the north training wall.



LEGEND

	SURVEYED EDGE OF WATER
	WETLAND LIMITS
1002+21	HEC-RAS CROSS-SECTION RIVER STATION
20	SURVEYED CONTOUR - MAJOR
18	SURVEYED CONTOUR - MINOR
30	LIDAR CONTOURS

0' 25' 50'
0 1/2" 1"

NOTES:

1) ALL VERTICAL ELEVATIONS ARE REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
NAVD 88 ≈ NGVD 29 - 1.0'

2) ALL HORIZONTAL ELEVATIONS ARE REFERENCED TO THE NORTH AMERICAN DATUM OF 1983 (NAD83).

3) ONE-FOOT CONTOUR INTERVAL FROM MMI FIELD SURVEY DATED APRIL 22-23, 2014.

4) ONE-FOOT CONTOUR INTERVAL TOPOGRAPHIC MAPPING IS BASED ON 2011 LIDAR.

5) WETLANDS WERE DELINEATED BY PEER CONSULTANTS, P.C. ON APRIL 21-22, 2014.

6) SEDIMENT SAMPLING WAS CONDUCTED ON APRIL 23, 2014

7) INFORMATION PRESENTED HEREIN IS AN ACCURATE REPRESENTATION OF AVAILABLE DATA.

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REVISIONS

HEC-RAS SECTIONS PLAN - CROSS SECTION LOCATION

MILLBROOK MEADOW

ROCKPORT, MASSACHUSETTS

--	JEP	EEL
DESIGNED	DRAWN	CHECKED

SCALE 1"=50'

DATE SEPTEMBER 2, 2014

PROJECT NO. 5220-02

SHEET NO. FIG. 3

Copyright Milone & MacBroom, Inc - 2014

Drawing: W:\DESIGN\5220-02-DE\CAD\HEC-RAS.DWG Layout Tab:11X17
Plotted by: JESSICAP On this date: Tue, 2014 September 2 - 5:46pm

The existing weir is 3.3 feet wide, with a rise of 4.2 feet. The table crest of the weir is at elevation 11.7 feet; the low chord of the footbridge is at 15.9 feet at the highest point of the arch. The surrounding park grade is approximately 15 feet. The weir vertically constricts flow through the footbridge.

The footbridge capacity is computed to be 68 cfs. Above 68 cfs, flow is predicted to overtop the granite and stone walls of the pond and spill out into the park.

Beach Street Culvert

Beach Street is an urban local road, with a Recommended Design Flood Frequency of the 25-year storm according to the *2006 Massachusetts Highway Department Project Development and Design Guidelines*. The 25-year storm discharge predicted by GEI for Mill Brook is 520 cfs. The existing culvert was computed to have a capacity of 153 cfs during the 25-year event.

Massachusetts culvert design guidelines recommend a minimum desirable freeboard of 2 feet between the allowable headwater and the roadway overtopping elevation. Allowable headwater is determined by the following criteria:

- → Nondamaging to upstream property
- → Below the traffic lanes of interest or no higher than the shoulder or 1.6 feet below the edge of the shoulder
- → Equal to a headwater/depth no greater than 1.5
- → No greater than the low point in the road grade
- → Equal to the elevation where flow diverts around the culvert

Upstream properties include the Nate's at Front Beach restaurant to the south of the entrance to the park and the Peg Leg Inn to the north. Nate's at Front Beach has a surveyed elevation of 14.3; the Peg Leg Inn was not surveyed during this phase of

analysis. Visual observation confirms that the first floor elevation of the Peg Leg Inn is higher than Nate's at Front Beach.

Table 4-2 provides design elevations at the Beach Street culvert.

TABLE 4-2
Beach Street Culvert
Design Elevations

Location	Elevation (feet, NAVD88)
Beach Street (at culvert)	14.1
Low point in road grade[1]	14.0
Edge of shoulder	14.0
Upstream property[2]	14.3

[1]This is also the elevation where flow would divert around the culvert.
[2]Nate's at Front Beach

Residential structures along King Street and Mill Lane are generally above flood elevations in the park. The exception is 19 King Street, which sits at approximate elevation 19 just downstream of the dam. However, the controlling hydraulic feature at this location is the Frog Pond weir as opposed to the Beach Street culvert.

Under existing conditions, Beach Street is predicted to overtop during the 2-year storm and greater. The existing culvert does not meet the recommended guidelines of passing the 25-year storm. Additionally, even the 2-year storm event is predicted to impact upstream properties. The computed headwater to depth ratio for the 25-year design storm is 1.86, higher than the recommended value of 1.5.

4.2 Proposed Conditions

The primary concerns and challenges regarding the Mill Brook channel include:

→ Lack of channel capacity
→ Lack of habitat diversity and appropriate channel structure
→ Ability to absorb storm flows

The Beach Street culvert has its own challenges. These relate to the hydraulics of the culvert and include:

→ Lack of capacity
→ Debris clogging during storms

A hydraulic analysis was completed to determine the following:

- Appropriate channel dimension for the Mill Brook channel
- Impact on park hydraulics of increasing culvert capacity
- Impact of debris clogging on channel flooding

These analyses are discussed in the following sections.

Mill Brook Channel

Natural streams are known to tend toward a state called "dynamic equilibrium," in which the independent variables of discharge, sediment load, and valley slope influence the form of the stream. Dynamic equilibrium implies that the stream experiences change but will tend toward a stable state following disturbance. A stable channel is able to transport the discharge and sediment load provided by the watershed without adjustment to its channel form and dimension.

Hydraulic geometry relationships (HGRs) provide a means by which to predict stable channel geometry within a rural watershed. HGR computations predict a bankfull channel 15 feet wide.

The Copeland Stable Channel Design tool within HEC-RAS uses an analytical approach to achieve channel stability. The Copeland method predicts a stable channel width of 16 feet.

The effect of widening the channel on nuisance ponding was evaluated for the bankfull flow of 34 cfs. Preliminary hydraulic results indicate that widening the channel by as little as 5 feet can reduce ponding within the park.

Channel widths ranging from the existing 2.5 feet to 20 feet were evaluated to determine the relative benefit of successive widening of the channel. The following graph (Figure 5) depicts the effect of channel width on water surface elevation in the park just downstream of the dogwood grove, at River Station 1002+62 during the bankfull event. It is clear that widening the channel from its existing 2.5-foot width to a 5-foot width greatly reduces water surface elevations. Widening the channel beyond 5 feet provides additional reduction in water surface elevation, as well as ecological benefits such as increased flow diversity and additional ability to buffer storm surge.

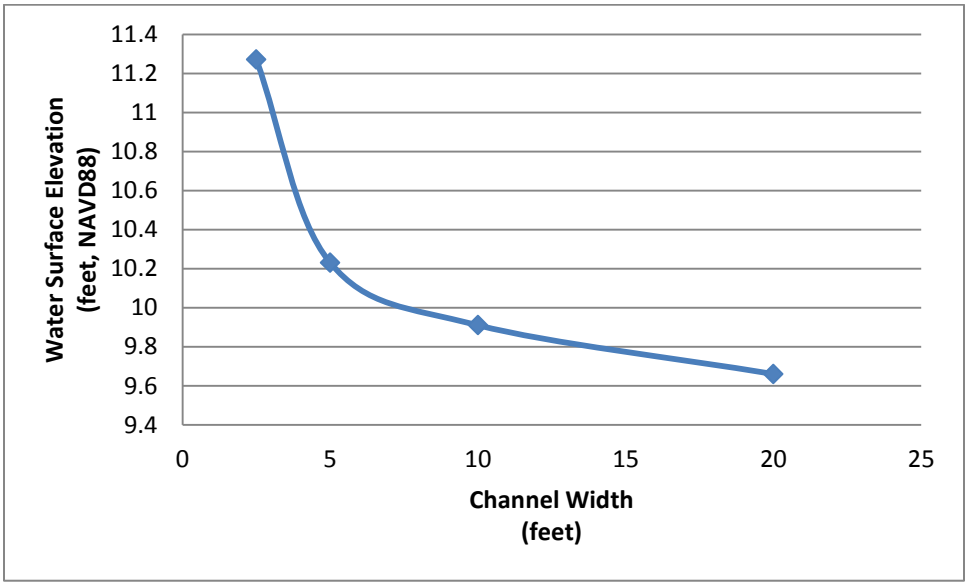


Figure 5: Water Surface Elevation Versus Channel Width in the Meadow, Just Downstream of Dogwood Grove (River Station 1002+62) for the Bankfull Event

Beach Street Culvert

The Beach Street culvert was assessed to determine if widening the existing culvert would have a positive impact on nuisance ponding within the park. The existing culvert has a capacity of approximately 100 cfs. This is greater than the bankfull event. Therefore, increasing the culvert span (width) would have little to no effect on reducing nuisance ponding, which occurs several times a year.

Multiple culvert widths were analyzed. A 10-foot-wide culvert has a capacity of approximately 150 cfs, an increase of 50 cfs over existing conditions. This increase in capacity reduces water surface elevations during the 2-year storm but has almost no effect on the 5-year storm discharge of 316 cfs and greater.

Widening the culvert span also reduces velocity through the culvert during storm events. This has the inverse effect of raising upstream water surface elevation as opposed to lowering it. Figures 6 and 7 graphically depict the effect of widening the culvert span on capacity (discharge) and water surface elevation, respectively.

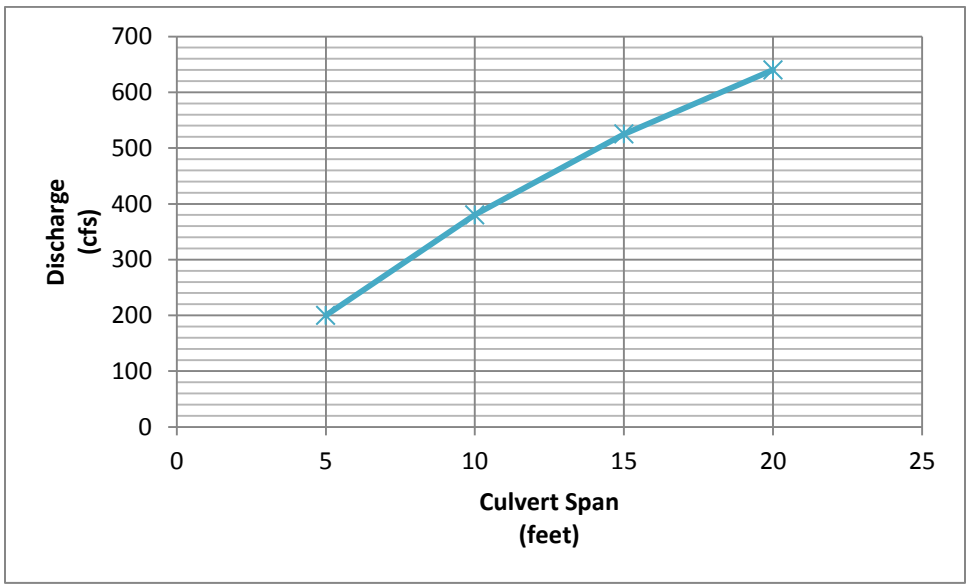


Figure 6: Culvert Span Versus Capacity

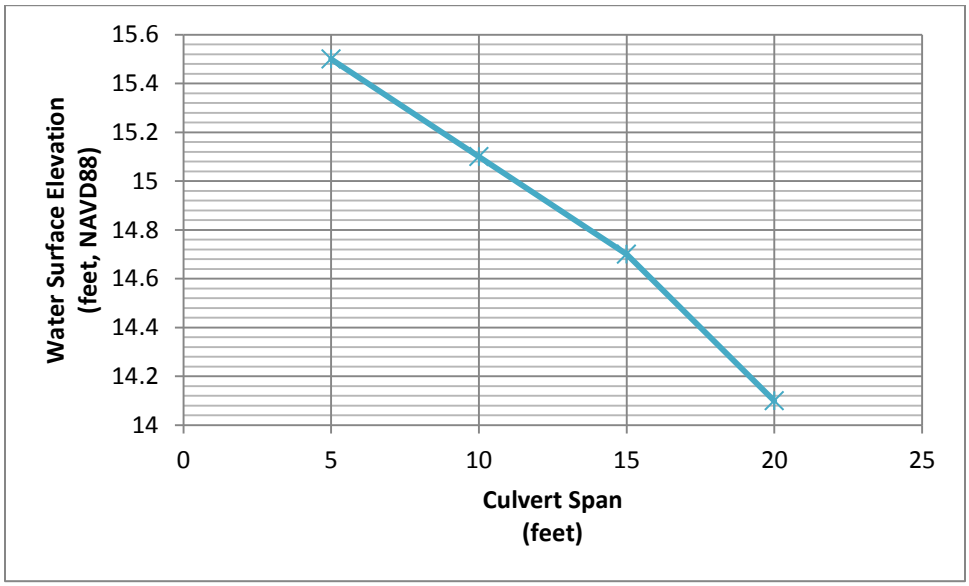


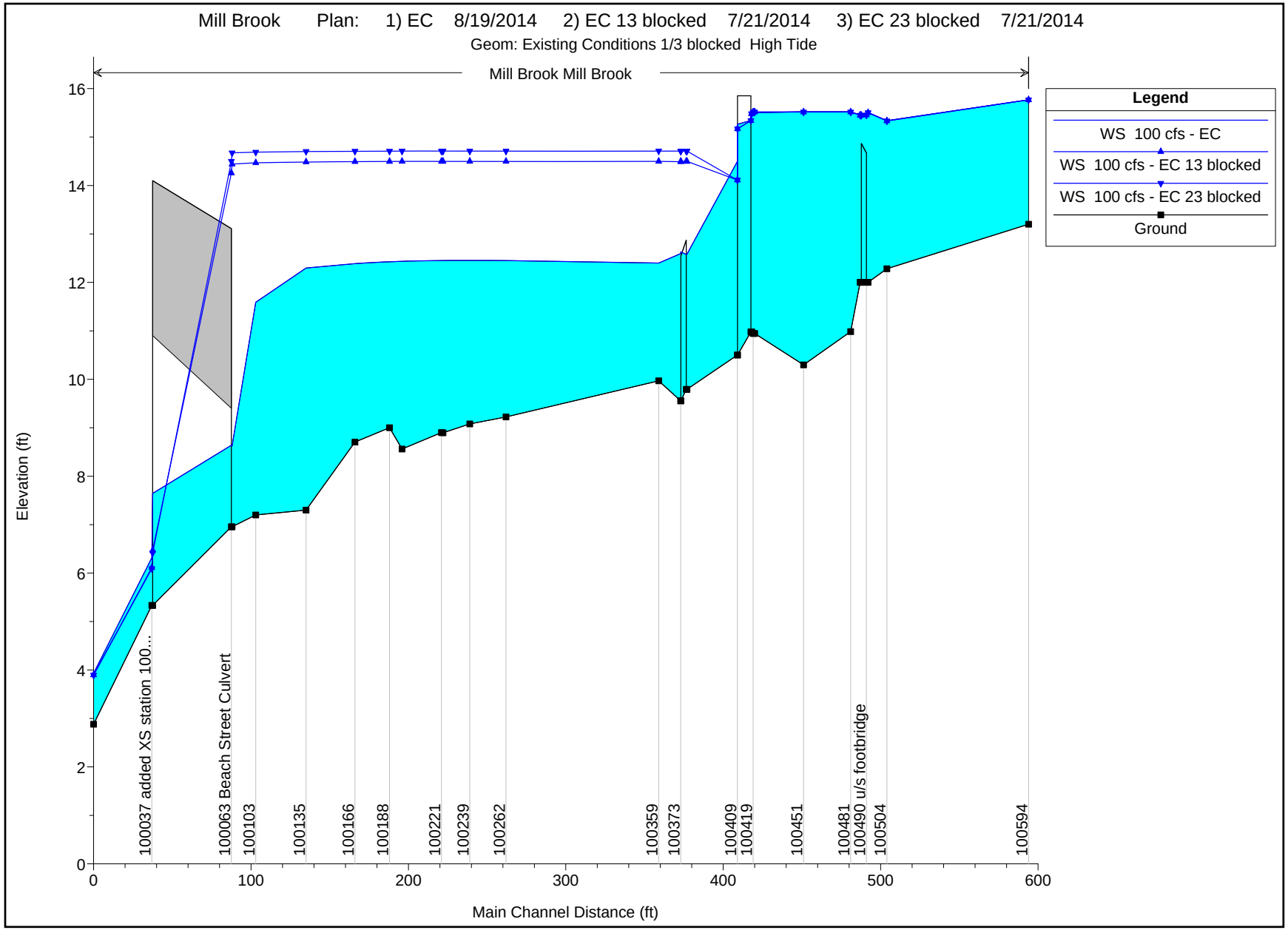
Figure 7: Culvert Span Versus Water Surface Elevation for the Bankfull Event

A 15-foot-wide culvert would be required to meet the Massachusetts recommended design criteria for the 25-year storm, using the potentially high GEI hydrology. Of great impact to culvert operations is reduction in debris clogging. Analysis of the culvert under two scenarios, 1/3 blocked and 2/3 blocked, was conducted for the 100 cfs flow. Figure 8 graphically depicts the impact of debris blockage on water surface elevations within the park.

It is clearly evident that even minor debris blockage of the culvert significantly raises water surface elevation.

5.0 ADDITIONAL DATA NEEDS AND NEXT STEPS

Several elements will benefit from additional analyses, as identified through this initial study. These are described as follows:


Mill Brook Plan: 1) EC 8/19/2014 2) EC 13 blocked 7/21/2014 3) EC 23 blocked 7/21/2014
Geom: Existing Conditions 1/3 blocked High Tide
Mill Brook Mill Brook
Legend
WS 100 cfs - EC
WS 100 cfs - EC 13 blocked
WS 100 cfs - EC 23 blocked
Ground
Elevation (ft)
Main Channel Distance (ft)
0
2
4
6
8
10
12
14
16
0
100
200
300
400
500
600
100037 added XS station 100...
100063 Beach Street Culvert
100103
100135
100166
100188
100221
100239
100262
100359
100373
100409
100419
100451
100481
100490 u/s footbridge
100504
100594

Hydrology – The bankfull channel discharge estimates are significantly lower than the predicted 2-year storm discharge. The HGR equations for discharge are based on measurements collected throughout Massachusetts and are not specifically related to a percent annual chance event. However, bankfull is often estimated as the 1.5-year event, such that the bankfull discharge should be of similar scale to the 2-year event. This discrepancy warrants additional hydrologic review, which should be completed during subsequent phases of design.

Mill Brook Channel – Channel dimension and planform will be adjusted to balance the channel footprint within the park with the increased hydraulic benefit of channel widening. Even minimal widening will improve the hydraulic function and habitat structure within the channel.

Beach Street Culvert – Initial analyses have demonstrated that debris clogging has a quantifiable effect on culvert discharge capacity. Eliminating existing blockages at the culvert, including fallen granite block and abandoned utility piping, will mitigate future debris clogging. Additionally, the means to safely and easily access the culvert to quickly remove debris from the culvert has been incorporated into the conceptual design.

Initial analyses also indicate that increasing the culvert span has the potential to reduce ponding in the park. Any modifications to the existing culvert will need to be closely coordinated with the Rockport Department of Public Works.

6.0 CONCLUSION

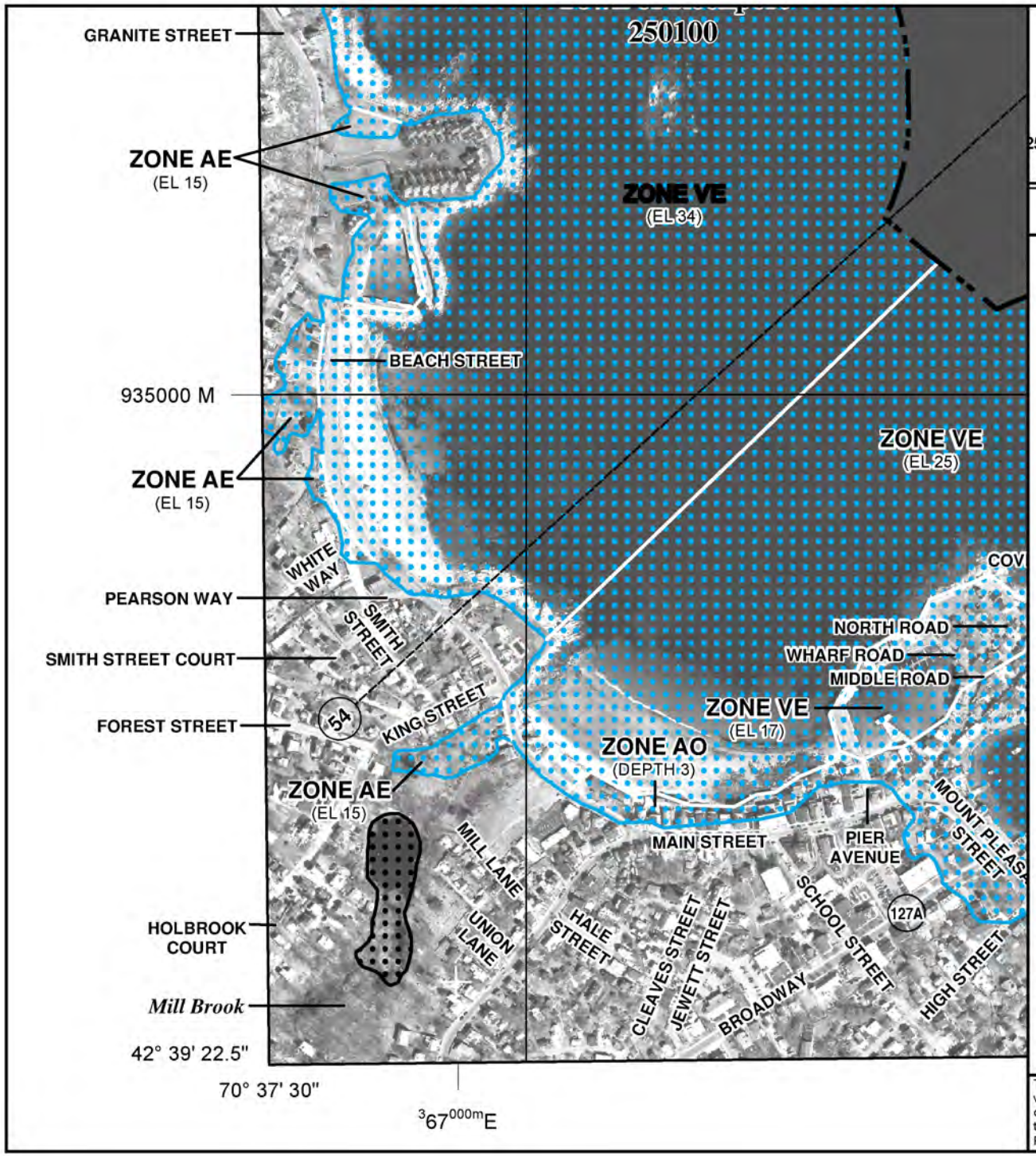
Preliminary findings indicate that increasing the width of the Mill Brook channel decreases nuisance flooding in the park. Increasing channel width will also allow for creation of increased habitat structures, including pools and riffles. The proposed channel modifications have been incorporated into the conceptual design of the channel.

Widening the span of the Beach Street culvert to 10 feet can alleviate frequent flooding but will not significantly impact larger storm events. A culvert span of 15 feet is required to achieve the Massachusetts recommended design guidelines for culverts, based on the 25-year storm event predicted by GEI. Reducing debris clogging of the culvert does have a significant impact on nuisance flooding within the park. Milone & MacBroom, Inc. will continue to work with the Town of Rockport Department of Public Works to determine the most cost-effective treatment of the culvert to reach project goals.

5220-02-3-s214-rpt

APPENDIX A

FEMA Documentation



MAP SCALE 1" = 500'

250 0 500 1000 FEET METERS

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0336G

FIRM

FLOOD INSURANCE RATE MAP

ESSEX COUNTY, MASSACHUSETTS

(ALL JURISDICTIONS)

PANEL 336 OF 600

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
ROCKPORT, TOWN OF	250100	0336	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
25009C0336G

MAP REVISED
JULY 16, 2014

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

FLOOD INSURANCE STUDY

ESSEX COUNTY, MASSACHUSETTS
(ALL JURISDICTIONS)
Volume 1 of 4

COMMUNITY NAME	COMMUNITY NUMBER
AMESBURY, CITY OF	250075
ANDOVER, TOWN OF	250076
BEVERLY, CITY OF	250077
BOXFORD, TOWN OF	250078
DANVERS, TOWN OF	250079
ESSEX. TOWN OF	250080
GEORGETOWN, TOWN OF	250081
GLOUCESTER, CITY OF	250082
GROVELAND, TOWN OF	250083
HAMILTON, TOWN OF	250084
HAVERHILL, CITY OF	250085
IPSWICH, TOWN OF	250086
LAWRENCE, CITY OF	250087
LYNN, CITY OF	250088
LYNNFILED, TOWN OF	250089
MANCHESTER BY THE SEA, TOWN OF	250090
MARBLEHEAD, TOWN OF	250091
MERRIMAC, TOWN OF	250092
METHUEN, CITY OF	250093
MIDDLETON, TOWN OF	250094
NAHANT, TOWN OF	250095
NEWBURY, TOWN OF	250096
NEWBURYPORT, CITY OF	250097
NORTH ANDOVER, TOWN OF	250098
PEABODY, CITY OF	250099
ROCKPORT, TOWN OF	250100
ROWLEY, TOWN OF	250101
SALEM, CITY OF	250102
SALISBURY, TOWN OF	250103
SAUGUS, TOWN OF	250104
SWAMPSCOTT, TOWN OF	250105
TOPSFIELD, TOWN OF	250106
WENHAM, TOWN OF	250107
WEST NEWBURY, TOWN OF	250108

REVISION DATE:
JULY 16, 2014

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
25009CV001B

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Selected Flood Insurance Rate Map panels for the community contain information that was previously shown separately on the corresponding Flood Boundary and Floodway Map panels (e.g., floodways, cross sections). In addition, former flood hazard zone designations have been changed as follows:

Old Zone	New Zone
A1 through A30	AE
V1 through V30	VE
B	X
C	X

Part or all of this Flood Insurance Study may be revised and republished at any time. In addition, part of this Flood Insurance Study may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the Flood Insurance Study. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current Flood Insurance Study components.

Initial Countywide FIS Effective Date: July 3, 2012

Revised Coastal FIS Effective Date: July 16, 2014

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Exhibit 1 - Flood Profiles – continued

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Exhibit 2 - Flood Insurance Rate Map Index
Flood Insurance Rate Map

FLOOD INSURANCE STUDY
ESSEX COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study (FIS) revises and updates information on the existence and severity of flood hazards in the geographic area of Essex County, including the Towns of Andover, Boxford, Danvers, Essex, Georgetown, Groveland, Hamilton, Ipswich, Lynnfield, Manchester by the Sea, Marblehead, Merrimac, Middleton, Nahant, Newbury, North Andover, Rockport, Rowley, Salisbury, Saugus, Swampscott, Topsfield, Wenham, and West Newbury; the Cities of Amesbury, Beverly, Gloucester, Haverhill, Lawrence, Lynn, Methuen, Newburyport, Peabody, and Salem (referred to collectively herein as Essex County), and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood-risk data for various areas of the community that will be used to establish actuarial flood insurance rates and to assist the community in its efforts to promote sound floodplain management. Minimum floodplain management requirements for participation in the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some states or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. These criteria take precedence over the minimum federal criteria for purposes of regulating development in the floodplain, as set forth in the Code of Federal Regulations at 44 CFR, 60.3. In such cases, the more restrictive criteria take precedence and the state (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for this FIS report are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

This FIS was prepared to incorporate all the communities within Essex County into a countywide format. Information on the authority and acknowledgements for each jurisdiction included in this countywide FIS, as compiled from their previously printed FIS reports, is shown below:

Amesbury, City of	For the original December 1979 study, the hydrologic and hydraulic analyses were prepared by the U.S. Army Corps of Engineers (USACE) for the Federal Emergency Management Agency (FEMA), under Inter-Agency Agreement No. IAA-H-7-76,

Newburyport, City of (continued)	under Contract No. H-4772. This work was completed in December 1983 (Reference 23).
North Andover, Town of	For the original December 15, 1982 study, the hydrologic and hydraulic analyses were prepared by Cullinan Engineering Co., Inc. for FEMA, under Contract No. H-4797. This work was completed in November 1980. For the June 2, 1993 revision, the hydrologic and hydraulic analyses were prepared by CDM. For FEMA, under Contract No. EMW-88-R-2627. This work was completed in October 1990 (Reference 24).
Peabody, City of	The hydrologic and hydraulic analyses for the November 1979 study were prepared by Anderson-Nichols and Company, Inc., for the FIA, under Contract No. H-4524. This work, which was completed in November 1978, covered all significant flooding sources in the City of Peabody (Reference 25).
Rockport, Town of	The hydrologic and hydraulic analyses for the December 19, 1984 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract NO. H-4772. This work was completed in September 1983 (Reference 26).
Rowley, Town of	The hydrologic and hydraulic analyses for the August 5, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in September 1980. The wave height and wave runup analyses were added by Dewberry & Davis using information supplied by Stone & Webster in the FIS's for the Towns of Ipswich and Newbury. The coastal analyses were completed in December 1983 (Reference 27).
Salem, City of	The hydrologic and hydraulic analyses in the February 5, 1985 study represents a revision of the original analyses by Anderson & Nichols, Inc., for FEMA, under Contract No. H-3715. The updated version was prepared by Stone& Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in July 1983. The hydrologic and hydraulic analyses in the

Wenham, Town of (continued)	original study was completed in December 1987. The hydrologic and hydraulic analyses for the Ipswich River in the 1991 revision were taken from the June 17, 1991 FIS for the Town of Topsfield (Reference 33).
West Newbury, Town of	The hydrologic and hydraulic analyses for the December 1978 study were performed by CDM for the FIA, under Contract No. H-3861. This work, which was completed in January 1977, covered all significant flooding sources affecting the Town of West Newbury (Reference 34).

Base map information shown on the Flood Insurance Map (FIRM) was derived from digital orthophotography. Base map files were provided in digital form by Massachusetts Geographic Information System (MassGIS). Ortho imagery was produced at a scale of 1:5,000. Aerial photography is dated April 2005. The projection used in the preparation of this map was Massachusetts State Plane mainland zone (FIPSZONE2001). The horizontal datum was NAD83, GRS1980 spheroid.

Additionally for the countywide revision, the Shawsheen River was restudied from its confluence with Merrimack River to the county boundary with Middlesex County. Revised hydrologic and hydraulic analyses were prepared by URS for FEMA. The funding for the study was provided by the under the Hazard Mitigation and Technical Assistance Contract Number HSFEHQ-06-D-0162, Task Order 042. That study was completed in May 2008.

For Shawsheen River, LiDAR data were collected in 2006/2007 by URS Group, Inc., and its subconsultant, EarthData. The vertical and horizontal accuracy of the LiDAR data are summarized in a May 29, 2007, report entitled Final LiDAR Report, Shawsheen River, Middlesex/ Essex Counties (URS Group, Inc., 2007).

The digital base map information for the Shawsheen River was provided by MassGIS. This information was derived from 15 centimeter (cm) and 30 cm digital orthophotos from aerial photography dated April 2008.

The coastal hydrologic and hydraulic analyses for this revised coastal study, was performed by Strategic Alliance for Risk Reduction (STARR) for FEMA under Contract No. HSFEHQ-09-D-0370. This study was completed May 7, 2013. This new study resulted in revisions to the Special Flood hazard Areas (SFHAs) within the coastal communities of the Towns of Danvers, Essex, Ipswich, Manchester by the Sea, Marblehead, Nahant, Newbury, Rockport, Rowley, Salisbury, Saugus, Swampscott, and Wenham; the Cities of Beverly, Gloucester, Lynn, Newburyport, Peabody, and Salem.

In 2011, STARR collected Light Detection and Ranging (LiDAR) covering 20.3 square miles of the Essex County coastline. The LiDAR was captured to the 'highest' vertical accuracy requirement which is the equivalent of a 2-foot contour accuracy. A 2 meter Digital Elevation Model (DEM) was derived from the LiDAR data. The DEM was projected in State Plane Massachusetts Mainland FIPS 2001 NAD 1983 US foot and used as the basis for coastal analysis and floodplain boundary delineation.

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Upper Artichoke Reservoir	On the Town of West Newbury/City of Newburyport Corporate Limits from its confluence with Merrimack River to its confluence with North Tributary Brook
Shallow Flooding	Sluice Pond, Flax Pond, and Cedar Pond in the Town of Lynn
Waters River	In the Town of Danvers, from its confluence with Danvers River to approximately 4600 feet upstream from State Highway 35 (Water Street)

For the countywide revision, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester by the Sea, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott.

Detailed study flooding sources that were not restudied as part of this revision may include a profile baseline on the FIRM. The profile baselines for these flooding sources were based on the best available data at the time of their study and are depicted as they were on the previous FIRMs. In some cases the transferred profile baseline may deviate significantly from the channel or may be outside of the floodplain.

Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon, by FEMA and the individual communities within Essex County. For this countywide revision, no new approximate studies were executed.

All or portions of the flooding sources listed in Table 3, "Flooding Sources Studied by Approximate Methods," were studied by approximate methods in the pre-countywide FISs.

TABLE 3 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS-continued

Flooding Source Name	Community (ies)
Upper Artichoke Reservoir	West Newbury
Upper Millpond	Rowley
Walden Pond	Lynn, Saugus
Waters River	Danvers
Wenham Lake	Beverly, Wenham
West Meadow River	Haverhill
Wetlands	Groveland, Haverhill
Wheeler Brook	Georgetown
Wilson Pond	Rowley

This FIS also incorporates the determinations of letters issued by FEMA resulting in map changes (Letter of Map Revision [LOMR], Letter of Map Revision - based on Fill [LOMR-F], and Letter of Map Amendment [LOMA]), as shown in Table 4, "Letters of Map Change."

TABLE 4 – LETTERS OF MAP CHANGE

Community	Case Number	Flooding Source	Letter Date
Andover, Town of	04-01-007P	Skug River	03/16/2004
Beverly, City of	08-01-0002P	Massachusetts Bay	08/01/2008
Boxford, Town of	96-01-031P	Upper and Lower Kimball Pond	8/8/1996
Wenham, Town of	06-01-B791P	Pond 1 and Pond 2	02/15/2007

For this 2013 coastal revision, the coastal analysis establishes the flood elevations for selected recurrence intervals primarily in the coastal communities of the Towns of Essex, Danvers, Ipswich, Manchester by the Sea, Marblehead, Nanhunt, Newbury, Rockport, Rowley, Saugus, and Swampscott; the Cities of Beverly, Gloucester, Newburyport, Lynn, Peabody, and Salem, There were no new LOMR determinations that resulted in FIRM revisions. No new riverine or approximate studies were performed as part of this coastal revision.

2.2 Community Description

Essex County is located in northeastern Massachusetts. In Essex County, there are twenty-six (26) towns and eight (8) cities. The Towns of Georgetown, Groveland, Manchester, Merrimac, Middleton, Newbury, Rockport, Salisbury, West Newbury and the Cities of Amesbury, Gloucester, Haverhill, Methuen, Newburyport, and Salem are located in northern Essex County. The Towns of

TABLE 5 – POPULATION AND TOTAL AREA BY COMMUNITY-continued

Community	Total Area (sq. mi)[1]	Population[1]
Newbury, Town of	26.5	6,666
Newburyport, City of	10.6	17,416
N. Andover, Town of	27.8	28,352
Peabody, City of	16.9	51,251
Rockport, Town of	17.6	6,952
Rowley, Town of	20.6	5,856
Salem, City of	18.1	41,340
Salisbury, Town of	17.9	8,283
Saugus, Town of	11.8	26,628
Swampscott, Town of	6.7	13,787
Topsfield, Town of	12.8	6,085
Wenham, Town of	8.1	4,875
W. Newbury, Town of	14.6	4,235

[1]U.S Census Bureau (Reference 35)

2.3 Principal Flood Problems

Past flooding on the streams within Essex County indicates that flooding can occur during any season of the year. Most major floods have occurred during February, March, and April and are usually the result of spring rains and/or snowmelt. Floods occurring during the midsummer and late summer are often associated with tropical storms moving up the Atlantic coastline. Severe flooding in Essex County generally occurs as a result of hurricanes or melting snows and spring rains, with more localized flooding caused by summer thunder-storms.

Trees, brush, and other vegetation growing along stream banks impede flood flows during high waters, thus creating backwater and increasing flood heights. Furthermore, trees, ice, and other debris may be washed away and carried downstream to collect on bridges and other obstructions. As the flood flow and debris surges downstream until another obstruction is encountered. Debris may collect against a bridge or culvert until the load exceeds the structural capacity, causing its destruction. It is difficult to predict the degree to which, or the location where, debris may accumulate. Therefore, in the development of the flood profiles it has been necessary to assume no accumulation of debris or obstruction of flow.

The flood problems for the communities within Essex County have been compiled and are described below, this information may not include the latest flood events:

Historical records indicate that since December 1740, numerous storms and floods have occurred in the Merrimack River Basin. The two latest and the most

The most significant recorded floods were those occurring in March 1936 and in September 1938. The 1936 flood was the result of a combination of heavy rains and extensive snow melt, while the 1938 flood was caused by heavy precipitation over the Merrimack River basin at a time when the ground was saturated from an earlier storm. The USACE estimated the 1936 flood to have a recurrence interval in excess of the 1-percent-annual-chance, while the 1938 flood had a recurrence interval of approximately the 2-percent-annual-chance (References 2 and 13).

Low-lying areas of Beverly, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Newburyport, Rockport, Rowley, Salem, Salisbury, Saugus, and Swampscott are subject to the periodic flooding and wave attack that accompany coastal storms, such as northeasters and hurricanes. The Town of Essex is protected from the heaviest wave attack by the barrier islands at Crane and Coffin Beaches. The majority of these storms cause damage only to low coastal roads, boats, beaches, and seawalls. Occasionally, a major storm accompanied by strong onshore winds and high tides results in surge and wave activity that causes extensive property damage and erosion. Some of the more significant storms in the area include those of December 1909 and 1959, November 1945, 1963, and 1968, and February 1972 and 1978. These storms damaged harbors and marinas and residential and commercial developments in the coastal areas.

Chebacco Lake occasionally floods in the spring but is not believed to threaten property in Essex.

Coastal flooding in Beverly has been particularly evident in the low-lying areas on the north end of Dane Street Beach. Riverine flooding has not generally been a serious problem in the city. The only significant problem reported is the frequent flooding of a portion of Cabot Street by North Beverly Drainage Ditch. This flooding is due primarily to an undersized culvert at Cabot Street and heavy siltation of the Boston and Maine rail road culverts just downstream of Cabot Street.

Few detailed records of flooding are available for the Town of Boxford or for the Town of Hamilton. Major storms in the region occurred in 1936, 1938, 1949, 1955, 1968, 1976, 1979 and 1987. Little information is available for any of these storms. In Boxford, the April 1987 storm caused overtopping of several roads located around local streams. During the April 1987 storm, the town suffered approximately $37,000 worth of flood damages, of which the town recovered approximately $14,000 of eligible damages from Federal Emergency Relief. Numerous residential and commercial basements were pumped by the Fire Department. At least 5 roads were overtopped by floodwaters. Most of these were caused by high backwater from downstream ponds, although in one case the culvert system serving as the Stiles Pond outlet was overtopped and required replacement. In Hamilton, the April 1987 storm caused overtopping of the approaches to the Winthrop Street Bridge by the Ipswich River. The bridge could not be crossed for several days until the floodwaters receded. The Winthrop Street

Newburyport Bridge has been caused by high tides or high river flows, separately or in coincidence. The high river flows have resulted from heavy rainfall or from a combination of rainfall and snowmelt. Since 1923, the USGS has continuously recorded Merrimack River flows at the gage 28 miles upstream of Newburyport in Lowell, Massachusetts. This station measures runoff from a 4,635 square mile drainage area, or 93 percent of the drainage area that contributes flow to the Merrimack River at Newburyport.

The major floods in Peabody have resulted from rainfall alone or in combination with snowmelt. Flooding on the Ipswich River has been documented by gage records, high-water marks, and personal accounts. Records at the Middleton and Ipswich gages on the Ipswich River give an indication of the magnitude of flood discharges in Peabody, which is located between the two gages. The Middleton gage began operation in 1938. The highest flood of record occurred in January 1979 and had a measured peak flow of 835 cfs. Other major floods, in order of decreasing peak discharge, are as follows: March 1968, October: 1962, March 1969, January 1958, March 1948, July 1938, December 1969, and March 1954. The gage at Ipswich began recording flows in 1931. Its highest flood of record also occurred in 1968, but a flood that occurred in March 1936, showed a peak discharge of nearly the same magnitude (References 40 and 41). Based upon accounts of Ipswich residents, a flood that occurred in February 1886 reached a higher elevation than that for the 1936 flood, however, no measurements of this 1886 flood are available (Reference 39). The 1936 flood had a flood height of 51.2 feet on the Ipswich River at Boston Street. The Ipswich River had 1968 flood heights of 52.8 and 51.3 feet at Boston Street and the USGS gaging station at South Middleton, respectively (References 38 and 42). Gage records and published high-water marks are not available to document floods on the Goldthwaite Brook, Proctor Brook, North River, Strongwater Brook, and Tapley Brook. The hydrologic conditions that cause major flooding on the Ipswich River can result in flooding on these smaller streams.

During the February 6-7, 1978 blizzard in Rockport, damage occurred to coastal areas around Penzance Road, Old Harbor, and Pigeon Cove.

The area of the City of Salem that has been consistently the most heavily damaged is Salem Willows. Also subject to damage are areas within Salem Harbor, such as Derby Wharf, Palmerus Cove, and Forest River Park.

Continuing erosion in Salisbury associated with severe storms also acts to reduce beach and dune width to below protective and recreational use requirements (Reference 43).

River flooding in Saugus has not been a serious problem in the tidal area. The non-tidal flooding problems are primarily due to flooding' along the Saugus River and its tributaries.

There are four completed reservoirs located upstream of the Merrimack River in New Hampshire. These projects are operated in conjunction with each other to reduce flooding on the upstream tributaries and main stem of the Merrimack River. As a result of these projects, flood discharges along the Merrimack River in Essex County have been significantly reduced. In addition to the upstream reservoirs, the USACE has also completed five local protection projects, but they are not designed to affect flooding in Essex County. These structures are the Franklin Falls Dam on the Pemigewasset River, the Edward McDowell Dam on Nubanusit Brook, the Blackwater Dam on the Blackwater River; and two dams that control Hopkinton Lake, the Everett Dam on the Piscataquog River, and the Hopkinton Dam on the Contoocook River. The USACE studies indicate that the five completed projects would have reduced the peak discharge of the 1936 flood on the Merrimack River at Andover by approximately 35 percent. No flood control reservoirs have been built in the Massachusetts portion of the Merrimack River basin, and none are contemplated. In part, this is because of the relatively flat topography, which does not permit storage of large volumes of floodwaters behind a single large dam without flooding adjacent developed areas. There are also 14 non-federal reservoir or lake systems existing in the Merrimack River Basin with usable storage in excess of 4,000 acre-feet. These reservoirs have no storage specifically allocated for flood control; however, they are drawn down during the winter months and are capable of storing significant amounts of runoff during the spring snowmelt period. Dams located at Tuxbury Pond, Lake Gardner, and immediately downstream of Pond Street on the Powwow River do not provide flood protection measures.

The State of Massachusetts provides concrete seawalls and stone revetments to protect coastal highways in Essex County. Other protective structures were generally constructed and are maintained by the communities and private property owners to satisfy their individual requirements and financial capabilities. These structures include such backshore protection as timber and steel sheet piles, bulkheads, stone revetments, concrete seawalls, and pre-cast concrete units (Reference 43).

The principal protection along the Essex County coastline consists of a system of concrete and stone seawalls, approximately half of which are maintained by the communities and the remaining half by private owners.

At Kettle Cove in Manchester, each house has its own seawall, some of which were rebuilt after the February 1978 storm.

There is a partially submerged breakwater approximately 2 miles offshore of Rockport which affords some protection to the general area of Sandy Bay. At Rockport Harbor, there is a shore attached breakwater.

Regulation of the outlet structures of the major ponds in the Town of Boxford provide a limited means of controlling flood levels both upstream and

Engineering Corporation for FEMA (Reference 91). The NAS methodology is based on three major concepts.

First, a storm surge on the open coast is accompanied by waves. The maximum height of these waves is related to the depth of water by the following equation:

$$H_b = 0.78d$$

where H_b is the crest to trough height of the maximum or breaking wave and d is the stillwater depth. The elevation of the crest of an unimpeded wave is determined using the equation:

$$Z_w = S_* + 0.7H_* = S_* + 0.55d$$

where Z_w is the wave crest elevation, S_* is the stillwater elevation at the site, and H_* is the wave height at the site. The 0.7 coefficient is the portion of the wave height which reaches above the stillwater elevation. H_b is the upper limit for H_*.

The second major concept is that the breaking wave height may be diminished by dissipation of energy by natural or man-made obstructions. The wave height transmitted past a given obstruction is determined by the following equation:

$$H_t = BH_i$$

where H_t is the transmitted wave height, H_i is the incident wave height, and B is a transmission coefficient ranging from 0.0 to 1.0. The coefficient is a function of the physical characteristics of the obstruction. Equations have been developed by NAS to determine B for vegetation, buildings, natural barriers such as dunes, and man-made barriers such as breakwaters and seawalls (Reference 89).

The third concept deals with unimpeded reaches between obstructions. New wave generation can result from wind action. This added energy is related to distance and mean depth over the unimpeded reach.

As part of this countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott. A description of the revised analyses is presented in the Countywide section below.

For each coastal community within Essex County that has been studied prior to this countywide update, the coastal analyses described in the previous FIS reports have been compiled and are summarized below.

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS-continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1] 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ATLANTIC OCEAN – cont'd				
Entire coastline within Newbury	7.4	8.1	8.4	9.0
Entire coastline of Rockport	7.5	8.3	8.6	9.2
Shoreline of Massachusetts Bay, Salem Harbor, and Beverly Harbor In Salem	7.7	8.5	8.8	9.6
Entire shoreline within Community of Salisbury	7.4	8.1	8.3	9
Saugus tidal area	8.0	8.8	9.2	10
Entire coastline of Swampscott	7.6	8.4	8.7	9.5
At the Rowley/Newbury Corporate limits	7.5	8.2	8.5	9.2
At the Ipswich/Rowley Corporate limits	7.7	8.4	8.7	9.4
BEVERLY HARBOR				
Waters River, Porter River, and Crane River In Danvers	8.4	9.5	10.0	11.6
Waters River Within Peabody Limits	8.4	9.5	10.0	11.6
At the Danvers River In Beverly	7.5	8.3	8.7	9.4

[1]North American Vertical Datum of 1988

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS-continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1] 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
POND 2				
Approximately 700 feet north of intersection of Pleasant and Cherry Streets	*	*	53.6	*
PILLINGS POND				
At Lynnfield	97.4	98.1	98.3	98.8
SLUICE POND				
In Lynn	63.8	*	64.2	*
UPPER ARTICHOKE RESERVOIR				
Upstream of the Dam in Newburyport	12.6	12.9	13	13.1

* Data not available

[1]North American Vertical Datum of 1988

Countywide Analyses

As part of the countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott. Redelineation of coastal flood hazards is defined as applying the results of previous coastal analyses to new or more detailed topographic data. Provided below is a summary of the analyses performed. All revised coastal analyses and redelineation of coastal flood hazards were performed in accordance with Appendix D "Guidance for Coastal Flooding Analyses and Mapping," (Reference 96) of the Guidelines and Specifications (G&S), as well as, the "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update" (Reference 97).

For the communities with revised coastal analyses, published values in the Tidal Flood Survey (Reference 98) were used to estimate the stillwater elevations for the 10-, 2-, and 1-percent-annual-chance floods for open water flooding sources.

Figure 1 - Transect Location Map

Figure 2, "Transect Schematic" represents s sample transect that illustrates the relationship between stillwater elevation, the wave crest elevation, the ground elevation profile and the location of the V/A zone boundary.

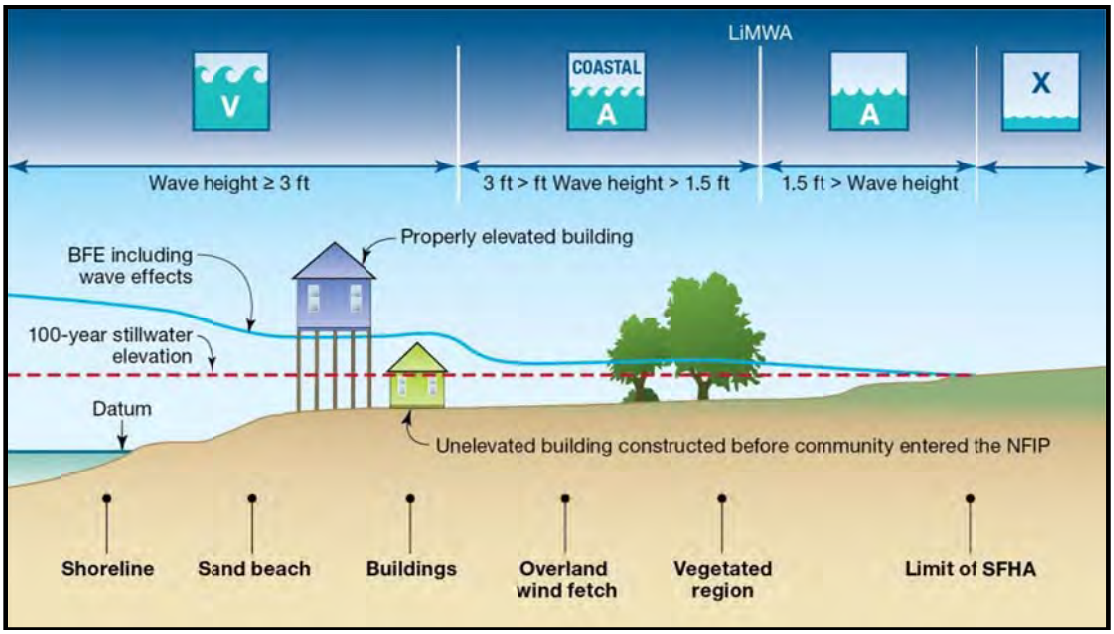

Figure 2. Transect Schematic

3.4 Vertical Datum

All FIS reports and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum used for newly created or revised FIS reports and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the completion of the North American Vertical Datum of 1988 (NAVD 88), many FIS reports and FIRMs are now prepared using NAVD 88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to the NAVD 88. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. Ground, structure, and flood elevations may be compared and/or referenced to NGVD 29 by applying a standard conversion factor. **The conversion factor from NGVD 29 to NAVD 88 is -0.8, and from NAVD 88 to NGVD 29 is +0.8.**

For information regarding conversion between the NGVD 29 and NAVD 88, visit the National Geodetic Survey website at www.ngs.noaa.gov, or contact the National Geodetic Survey at the following address:

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (Exhibit 2). On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, AO, and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations, but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM.

Redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott by applying the results of previous coastal analyses to new or updated topographic data.

For the countywide revision, the following communities provided high resolution topographic data. For these communities, this data was used for redelineation of the detailed and approximate study reaches.

- City of Beverly – Town Provided, Contour Interval 2 feet
- City of Methuen – Town Provided, Contour Interval 2 feet
- Town of Topsfield – James W. Sewall Company, Contour Interval 2 feet

For unrevised flooding sources in Essex County, data was taken from previously printed FISs for each individual community and are compiled below.

For the streams studied in detail, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. The boundaries were interpolated between cross sections using topographic maps at a scale of 1:4,800 with a contour interval of 5 feet in Andover (Reference 108); using photogrammetric maps in Beverly (Reference 109); at a scale of 1:4,800 with a contour interval of 4 feet in Boxford (Reference 110); at a scale of 1:2,400 and 1: 4,800 with contours intervals of 5 feet in Danvers, Lawrence, and Haverhill, Manchester, Merrimac, Methuen, Middleton, Salem, and Saugus (Reference 111 and 112); at a scale of 1:24,000 with a contour interval of 10 feet in Amesbury, Groveland, Hamilton, Topsfield, Wenham, West Newbury, and Georgetown (Reference 112); using topographic maps in Ipswich, Lynn, Gloucester, and Essex, Newbury, Newburyport, Rowley, Salisbury and Swampscott (Reference 113); at scales of 1:24,000 and 1:2,400 with contour intervals of 10 and 5 feet in Lynnfield (References 114 and 115); at a scale of

APPENDIX B

TIDAL DATA DOCUMENTATION

NOAA Tidal Benchmark Sheet
NGS Tidal Elevation Data Sheet
NOAA Tides and Currents 2014 Predicted Tidal Data
Excerpt from the FEMA Updated Tidal Profiles for the New England Coastline

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

Datums Page

Station ID: 8441551 **PUBLICATION DATE: 04/21/2003**
Name: ROCKPORT HARBOR MASSACHUSETTS
NOAA Chart: 13279 **Latitude: 42° 39.5' N**
USGS Quad: ROCKPORT **Longitude: 70° 36.9' W**

To reach the tidal bench marks from the intersection of State Routes 127 and 127A, proceed NE on State Route 127A (Broadway) for 0.6 km (0.4 mi) to Mount Pleasant Street, then continue NE past Mount Pleasant Street for 0.2 km (0.1 mi) to the parking lot for the T-wharf. The bench marks are in the vicinity of the T-wharf, along Broadway, and on Bearskin Neck. The tide gauge and staff were on the Sandy Bay Yacht Club pier.

T I D A L B E N C H M A R K S

PRIMARY BENCH MARK STAMPING: NO 10 1928
DESIGNATION: TIDAL 10 STA 8

MONUMENTATION: Tidal Station disk VM#: 11826
AGENCY: US Coast and Geodetic Survey (USC&GS) PID#: MY0231
SETTING CLASSIFICATION: Granite sidewalk curb

The primary bench mark is a disk set in a granite curb near the intersection of Broadway and Mount Pleasant Street, 24 m (79 ft) NNE of the north corner of Rockport Baptist Church, 17.28 m (56.7 ft) WNW of power pole #1468, and 7.56 m (24.8 ft) south of the centerline of Broad Street.

BENCH MARK STAMPING: NO 11 1928
DESIGNATION: TIDAL 11 STA 8

MONUMENTATION: Tidal Station disk VM#: 11827
AGENCY: US Coast and Geodetic Survey (USC&GS) PID:
SETTING CLASSIFICATION: Granite sidewalk curb

The bench mark is a disk set in a granite curb in front of the brick School Administration building on the north side of Broadway, 21.95 m (72.0 ft) SW of the SE corner of the building, 16.18 m (53.1 ft) east of the SW corner of the building, and 5.39 m (17.7 ft) north of the centerline of Broadway. Note: The high point of the mark is along the west edge of the mark.

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

Station ID: 8441551 **PUBLICATION DATE: 04/21/2003**
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NOAA Chart: 13279 **Latitude: 42° 39.5' N**
USGS Quad: ROCKPORT **Longitude: 70° 36.9' W**

T I D A L B E N C H M A R K S

BENCH MARK STAMPING: 1551 A 1992
DESIGNATION: 844 1551 A

MONUMENTATION: Tidal Station disk VM#: 11828
AGENCY: National Ocean Service (NOS) PID:
SETTING CLASSIFICATION: Concrete retaining wall

The bench mark is a disk set in a 0.30 m x 0.40 m (1.0 ft x 1.3 ft) concrete base along the north edge of the T-wharf, 25.30 m (83.0 ft) NW of the entrance to Sandy Bay Yacht Club, 17.01 m (55.8 ft) WSW of the NE interior corner of the T-wharf, 0.61 m (2.0 ft) NW of telephone pole #189, and 0.15 m (0.5 ft) above the asphalt pavement.

BENCH MARK STAMPING: 1551 B 1992
DESIGNATION: 844 1551 B

MONUMENTATION: Tidal Station disk VM#: 11829
AGENCY: National Ocean Service (NOS) PID:
SETTING CLASSIFICATION: Concrete base

The bench mark is a disk set in the NW corner of a 0.40 m (1.3 ft) square concrete base for the street sign at the intersection of South Street and Donovans Corner on Bearskin Neck, 12.68 m (41.6 ft) SSW of telephone pole #1484, 7.50 m (24.6 ft) NE of a 0.61 m (2.0 ft) diameter granite post, and 0.76 m (2.5 ft) SW of the NW corner of the Lobster Shack.

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

Station ID: 8441551 **PUBLICATION DATE: 04/21/2003**
Name: ROCKPORT HARBOR MASSACHUSETTS
NOAA Chart: 13279 **Latitude: 42° 39.5' N**
USGS Quad: ROCKPORT **Longitude: 70° 36.9' W**

T I D A L B E N C H M A R K S

BENCH MARK STAMPING: MOE 78
DESIGNATION: 841 1551 MOE

MONUMENTATION: Survey disk VM#: 11830
AGENCY: US Army Corps of Engineers (USE) PID:
SETTING CLASSIFICATION: Granite rock

The bench mark is a disk set in a granite rock along the south side of a rotary at the end of Bearskin Neck, 29 m (94 ft) SW of the center of the rock located in the middle of the rotary at the end of Bearskin Neck, 7.44 m (24.4 ft) NE of the NW corner of Old Stone Fort, 1.46 m (4.8 ft) SSE of the inner edge of the curb, and 0.30 m (1.0 ft) above the road.

BENCH MARK STAMPING:
DESIGNATION: 844 1551 T 25

MONUMENTATION: Not specified (see descriptive text) VM#: 11831
AGENCY: Unknown PID:
SETTING CLASSIFICATION: Rock outcrop

The bench mark is a copper plug in a rock outcrop behind the New England Telephone Company building at the intersection of Main and Hale Streets in Rockport, 21 m (70 ft) south of the intersection, 10 m (33 ft) SW of the centerline of Hale Street, and 5 m (17 ft) SE of the east corner of the building.

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

Station ID: 8441551 **PUBLICATION DATE: 04/21/2003**
Name: ROCKPORT HARBOR MASSACHUSETTS
NOAA Chart: 13279 **Latitude: 42° 39.5' N**
USGS Quad: ROCKPORT **Longitude: 70° 36.9' W**

T I D A L D A T U M S

Tidal datums at ROCKPORT HARBOR based on:

LENGTH OF SERIES: 2 Months
TIME PERIOD: August 1992 - September 1992
TIDAL EPOCH: 1983-2001
CONTROL TIDE STATION: 8443970 BOSTON, BOSTON HARBOR

Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

MEAN HIGHER HIGH WATER	MHHW	=	2.883
MEAN HIGH WATER	MHW	=	2.752
MEAN SEA LEVEL	MSL	=	1.436
MEAN TIDE LEVEL	MTL	=	1.426
MEAN LOW WATER	MLW	=	0.099
MEAN LOWER LOW WATER	MLLW	=	0.000

North American Vertical Datum (NAVD88)

Bench Mark Elevation Information In METERS above:

Stamping or Designation	MLLW	MHW
NO 10 1928	5.495	2.743
NO 11 1928	6.517	3.765
1551 A 1992	4.709	1.957
1551 B 1992	5.083	2.331
MOE 78	8.817	6.065
844 1551 T 25	20.455	17.703

U.S. DEPARTMENT OF COMMERCE
National Oceanic and Atmospheric Administration
National Ocean Service

Station ID:	**8441551**	**PUBLICATION DATE:**	**04/21/2003**
Name:	**ROCKPORT HARBOR MASSACHUSETTS**		
NOAA Chart:	**13279**	**Latitude:**	**42° 39.5' N**
USGS Quad:	**ROCKPORT**	**Longitude:**	**70° 36.9' W**

D E F I N I T I O N S

Mean Sea Level (MSL) is a tidal datum determined over a 19-year National Tidal Datum Epoch. It pertains to local mean sea level and should not be confused with the fixed datums of North American Vertical Datum of 1988 (NAVD88).

NAVD88 is a fixed datum derived from a simultaneous, least squares, minimum constraint adjustment of Canadian/Mexican/United States leveling observations. Local mean sea level observed at Father Point/Rimouski, Canada was held fixed as the single initial constraint. NAVD88 replaces NGVD29 as the national standard geodetic reference for heights. Bench mark elevations relative to NAVD88 are available from NGS through the World Wide Web at National Geodetic Survey.

NGVD29 is a fixed datum adopted as a national standard geodetic reference for heights but is now considered superseded. NGVD29 is sometimes referred to as Sea Level Datum of 1929 or as Mean Sea Level on some early issues of Geological Survey Topographic Quads. NGVD29 was originally derived from a general adjustment of the first-order leveling networks of the U.S. and Canada after holding mean sea level observed at 26 long term tide stations as fixed. Numerous local and wide-spread adjustments have been made since establishment in 1929. Bench mark elevations relative to NGVD29 are available from the National Geodetic Survey (NGS) data base via the World Wide Web at National Geodetic Survey.

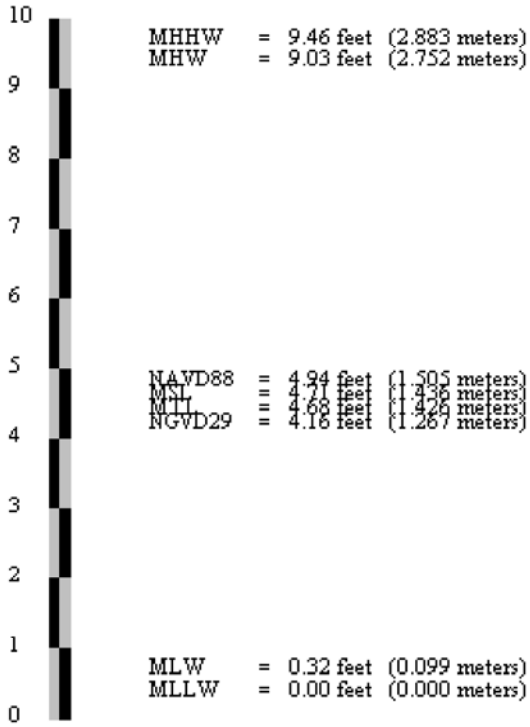
NAVD88 and NGVD29 are fixed geodetic datums whose elevation relationships to local MSL and other tidal datums may not be consistent from one location to another.

The Vertical Mark Number (VM#) and PID# shown on the bench mark sheet are unique identifiers for bench marks in the tidal and geodetic databases, respectively. Each bench mark in either database has a single, unique VM# and/or PID# assigned. Where both VM# and PID# are indicated, both tidal and geodetic elevations are available for the bench mark listed.

The NAVD88 elevation is shown on the Elevations of Tidal Datums Table Referred to MLLW only when two or more of the bench marks listed have NAVD88 elevations. The NAVD88 elevation relationship shown in the table is derived from an average of several bench mark elevations relative to tide station datum. As a result of this averaging, NAVD88 bench mark elevations computed indirectly from the tidal datums elevation table may differ slightly from NAVD88 elevations listed for each bench mark in the NGS database.

ELEVATION INFORMATION

PID:	MY0231
VM:	11826
STATION ID:	8441551
EPOCH:	1983-2001
DATE:	Tuesday, September 2, 2014 9:25:59 AM EST



Updated Tidal Profiles for the New England Coastline

March 2012

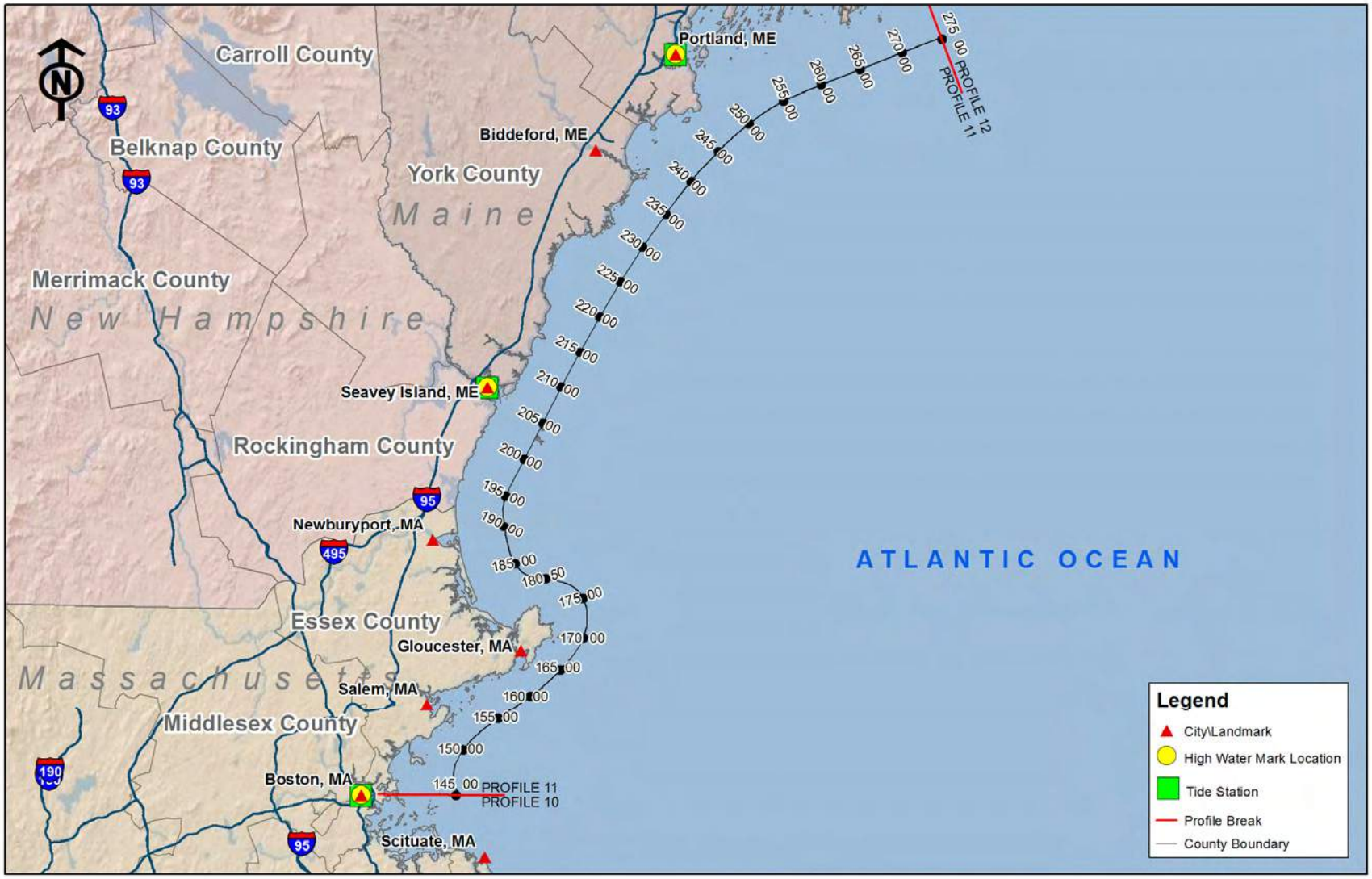


Figure B13. Base Map for Profile 11 from Boston, MA to mile 275

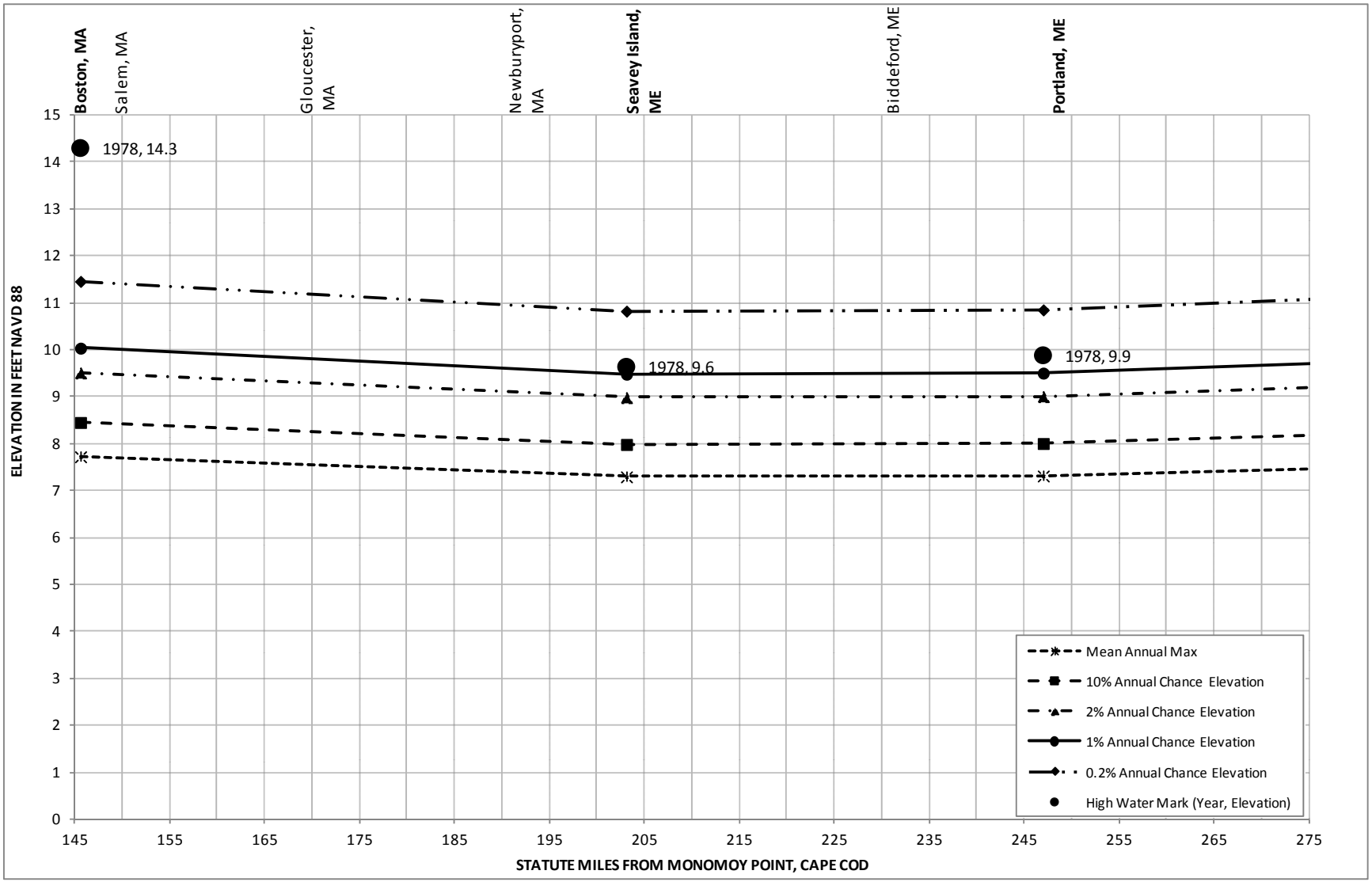


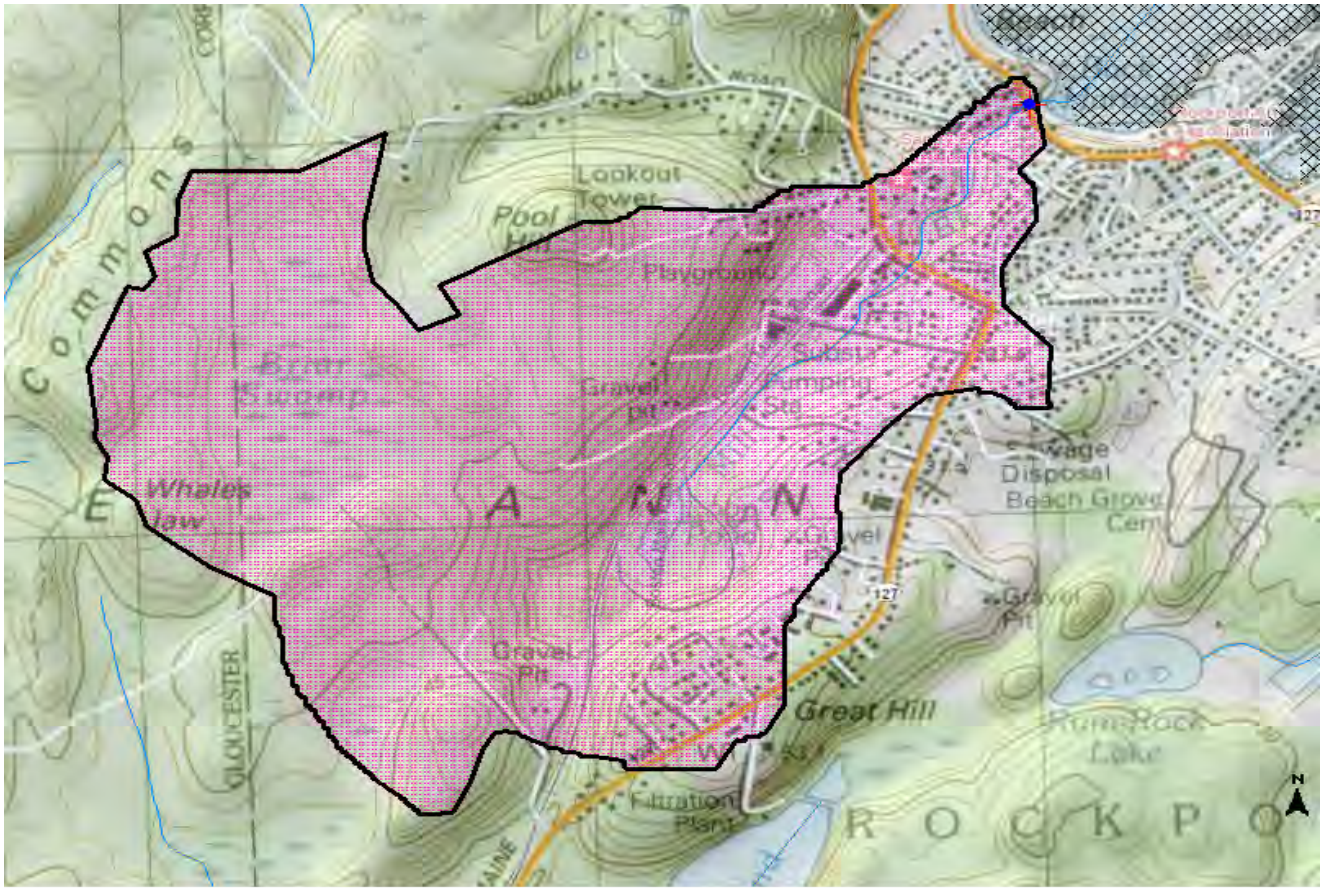
Figure C13. Tidal Flood Profile 11 from Boston, MA to mile 275

APPENDIX C

HYDROLOGIC DOCUMENTATION

USGS
Massachusetts StreamStats

StreamStats Print Page



Explanation

- NHDHGage
- NHDHDam
- Gaging Station, Continuous Record
- Low Flow, Partial Record
- Peak Flow, Partial Record
- Peak and Low Flow, Partial Record
- Stage Only
- Low Flow, Partial Record, Stage
- Miscellaneous Record
- Unknown
- GlobalWatershedPoint
- Dendritic Stream Network
- GlobalWatershed
- Excludepoly

0.3 0.15 0 0.3 Miles

9/2/2014 8:59:21 AM

USGS Massachusetts StreamStats

Basin Characteristics Report

Date: Tue Sep 2 2014 08:55:52 Mountain Daylight Time
NAD27 Latitude: 42.6594 (42 39 34)
NAD27 Longitude: -70.6225 (-70 37 21)
NAD83 Latitude: 42.6595 (42 39 34)
NAD83 Longitude: -70.6220 (-70 37 19)
ReachCode: 01090001001032
Measure: 2.26

Basin has been edited

Parameter	Value
X coordinate of the outlet in Massachusetts State Plane (meters)	215225.0
Y coordinate of the outlet in Massachusetts State Plane (meters)	906975.0
X coordinate of the centroid in Massachusetts State Plane (meters)	270921.0
Y coordinate of the centroid in Massachusetts State Plane (meters)	933825.9
Area in square miles	0.89
Mean annual precipitation in the Conn River basin, in inches	0.000
Average area slope in percent	0
square mile area covered by stratified drift	
Total stream length in miles	0.85
stratified drift per unit stream lenth	
low flow region indicator for Massachusetts	0
Area of forest land (percent)	
Area of sand and gravel deposits (percent)	
Coarse-grained stratified drift - SYE	
Percentage of impervious area determined from NLCD 2001 impervious dataset	0
Percentage of urban land cover determined from NLCD 2001 land cover dataset	

USGS Massachusetts StreamStats

Streamstats Ungaged Site Report

Date: Tue Sep 2 2014 09:02:53 Mountain Daylight Time
Site Location: Massachusetts
NAD27 Latitude: 42.6594 (42 39 34)
NAD27 Longitude: -70.6225 (-70 37 21)
NAD83 Latitude: 42.6595 (42 39 34)
NAD83 Longitude: -70.6220 (-70 37 19)
ReachCode: 01090001001032
Measure: 2.26
Drainage Area: 0.89 mi2
Percent Urban: 24.3 %
Percent Impervious: 10.6 %

Basin has been edited

Low Flows Basin Characteristics			
100% Statewide Low Flow (0.89 mi2)			
Parameter	Value	Regression Equation Valid Range	
		Min	Max
Drainage Area (square miles)	0.89 (below min value 1.61)	1.61	149
Mean Basin Slope from 250K DEM (percent)	2.81	0.32	24.6
Stratified Drift per Stream Length (square mile per mile)	0.0189	0	1.29
Massachusetts Region (dimensionless)	0	0	1

Warning: Some parameters are outside the suggested range. Estimates will be extrapolations with unknown errors.

Basin has been edited

Probability of Perennial Flow Basin Characteristics			
100% Perennial Flow Probability (0.89 mi2)			
Parameter	Value	Regression Equation Valid Range	
		Min	Max
Drainage Area (square miles)	0.89	0.01	1.99
Percent Underlain By Sand And Gravel (percent)	1.80	0	100

934

Percent Forest (percent)	62.10	0	100
Massachusetts Region (dimensionless)	0	0	1

Low Flows Streamflow Statistics

Statistic	Flow (ft^3/s)	Prediction Error (percent)	Equivalent years of record	90-Percent Prediction Interval	
				Minimum	Maximum
D50	0.85				
D60	0.52				
D70	0.25				
D75	0.18				
D80	0.12				
D85	0.0855				
D90	0.0536				
D95	0.0287				
D98	0.0174				
D99	0.0119				
M7D2Y	0.0323				
AUGD50	0.0904				
M7D10Y	0.00962				

The equation for estimating the probability of perennial flow is applicable for most areas of Massachusetts except eastern Buzzards Bay, Cape Cod, and the Island regions. The estimate obtained from the equation assumes natural flow conditions at the site. The equation also is best used for sites with drainage areas between 0.01 to 1.99 mi2, as errors beyond for basins beyond these bounds are unknown.

Probability of Perennial Flow Statistics

Statistic	Value	Standard Error (percent)
PROBPEREN	0.78	0.3

APPENDIX D

HYDRAULIC GEOMETRY COMPUTATIONS

Prepared in cooperation with the
Massachusetts Department of Environmental Protection
Bureau of Resource Protection
Wetlands and Waterways Program and
Massachusetts Environmental Trust

Equations for Estimating Bankfull Channel Geometry and Discharge for Streams in Massachusetts

Scientific Investigations Report 2013–5155

U.S. Department of the Interior
U.S. Geological Survey

Table 6. Statistical summary of simple and multiple regression equations for estimating bankfull stream width, mean depth, cross-sectional area, and discharge for streams in and near Massachusetts.

[n, number; R^2, coefficient of determination; S_e, standard error of the estimate; S_p, standard error of the prediction; ft, foot; mi^2, square miles; ft^2, square foot; ft^3/s, cubic foot per second; log, base-10 logarithm; %, percent]

Equation number	Equation	n	R^2	Adjusted R^2	Predicted R^2	S_e (log)	S_e (%)	S_p (log)	S_p (%)
	Simple regression equation								
8	Bankfull width (ft) = 15.0418 [Drainage area (mi^2)]$^{0.4038}$	33	0.877	0.873	0.8635	0.0903	21.02	0.0950	22.14
9	Bankfull mean depth (ft) = 0.9502 [Drainage area (mi^2)]$^{0.2960}$	33	0.820	0.814	0.7995	0.0826	19.19	0.0873	20.30
10	Bankfull cross-sectional area (ft^2) = 14.1156 [Drainage area (mi^2)]$^{0.7026}$	33	0.911	0.908	0.9028	0.1308	30.81	0.1369	32.31
11	Bankfull discharge (ft^3/s) = 37.1364 [Drainage area (mi^2)]$^{0.7996}$	33	0.770	0.762	0.7428	0.2608	65.89	0.2757	70.43
	Multiple regression equation								
12	Bankfull width (ft) = 10.6640 [Drainage area (mi^2)]$^{0.3935}$ [Mean basin slope (%)]$^{0.1751}$	33	0.900	0.894	0.8780	0.0825	19.17	0.0913	21.26
13	Bankfull mean depth (ft) = 0.7295 [Drainage area (mi^2)]$^{0.2880}$ [Mean basin slope (%)]$^{0.1346}$	33	0.845	0.834	0.8159	0.0781	18.13	0.0850	19.76
14	Bankfull cross-sectional area (ft^2) = 7.6711 [Drainage area (mi^2)]$^{0.6842}$ [Mean basin slope (%)]$^{0.3105}$	33	0.937	0.932	0.9320	0.1123	26.30	0.1234	29.00
15	Bankfull discharge (ft^3/s) = 8.2490 [Drainage area (mi^2)]$^{0.7545}$ [Mean basin slope (%)]$^{0.7659}$	33	0.871	0.862	0.8366	0.1986	48.23	0.2234	55.02

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JOB Millbrook Meadow
SHEET NO. _____ OF _____
CALCULATED BY EL DATE _____
CHECKED BY _____ DATE _____
SCALE _____

Bankfull Discharge

$Q_b = 37.1364\ DA^{0.7996}$

Q_b = bankfull discharge

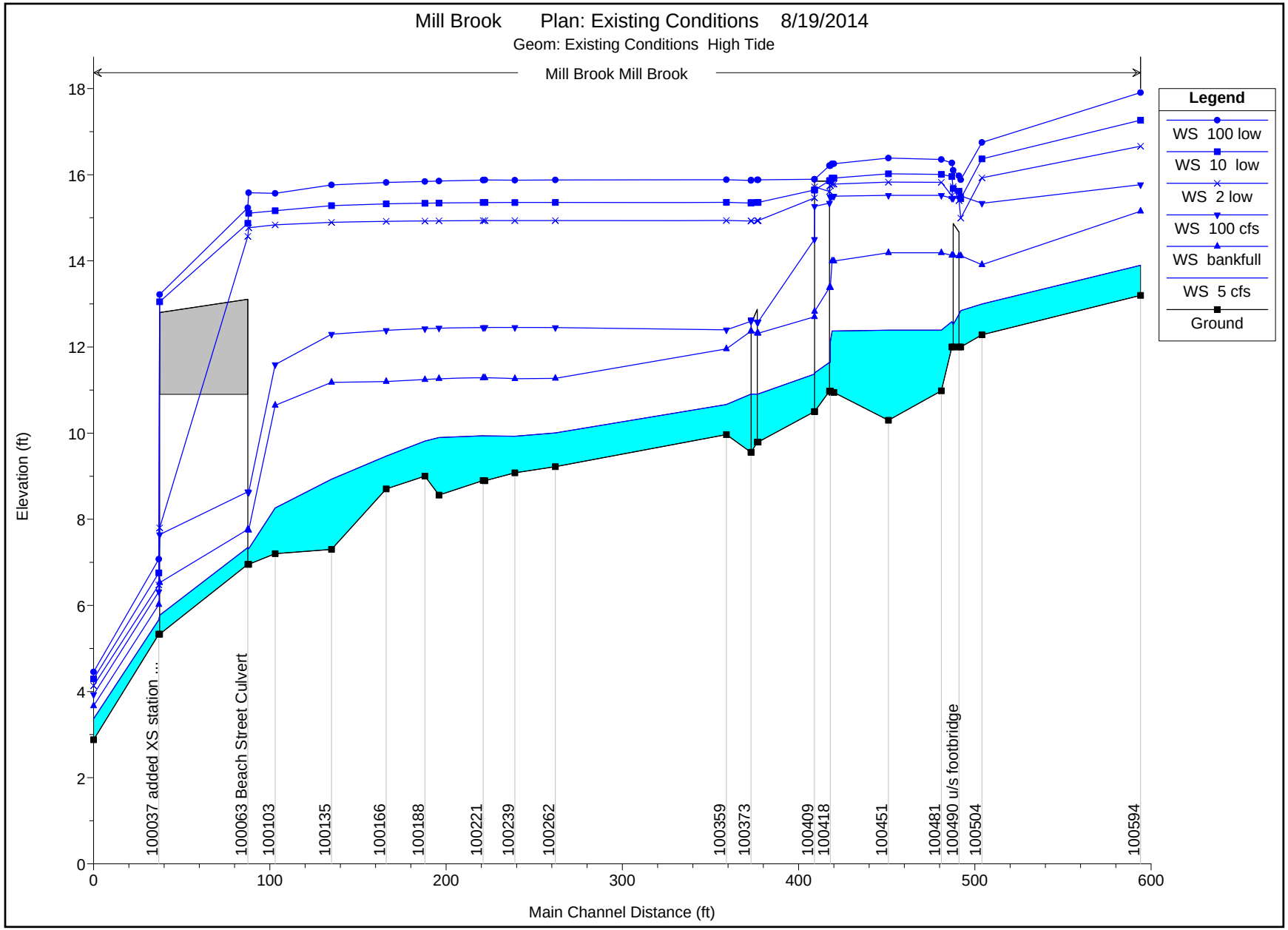
DA = Drainage Area
0.88 mi^2

$Q_b = 33.5$ cfs

$\boxed{\rightarrow 34 \text{ cfs}}$

APPENDIX E

HEC-RAS DATA


Mill Brook Plan: Existing Conditions 8/19/2014
Geom: Existing Conditions High Tide
Mill Brook Mill Brook
Legend
WS 100 low
WS 10 low
WS 2 low
WS 100 cfs
WS bankfull
WS 5 cfs
Ground
Elevation (ft)
Main Channel Distance (ft)
0
2
4
6
8
10
12
14
16
18
0
100
200
300
400
500
600
100037 added XS station ...
100063 Beach Street Culvert
100103
100135
100166
100188
100221
100239
100262
100359
100373
100409
100418
100451
100481
100490 u/s footbridge
100504
100594

HEC-RAS Plan: EC River: Mill Brook Reach: Mill Brook

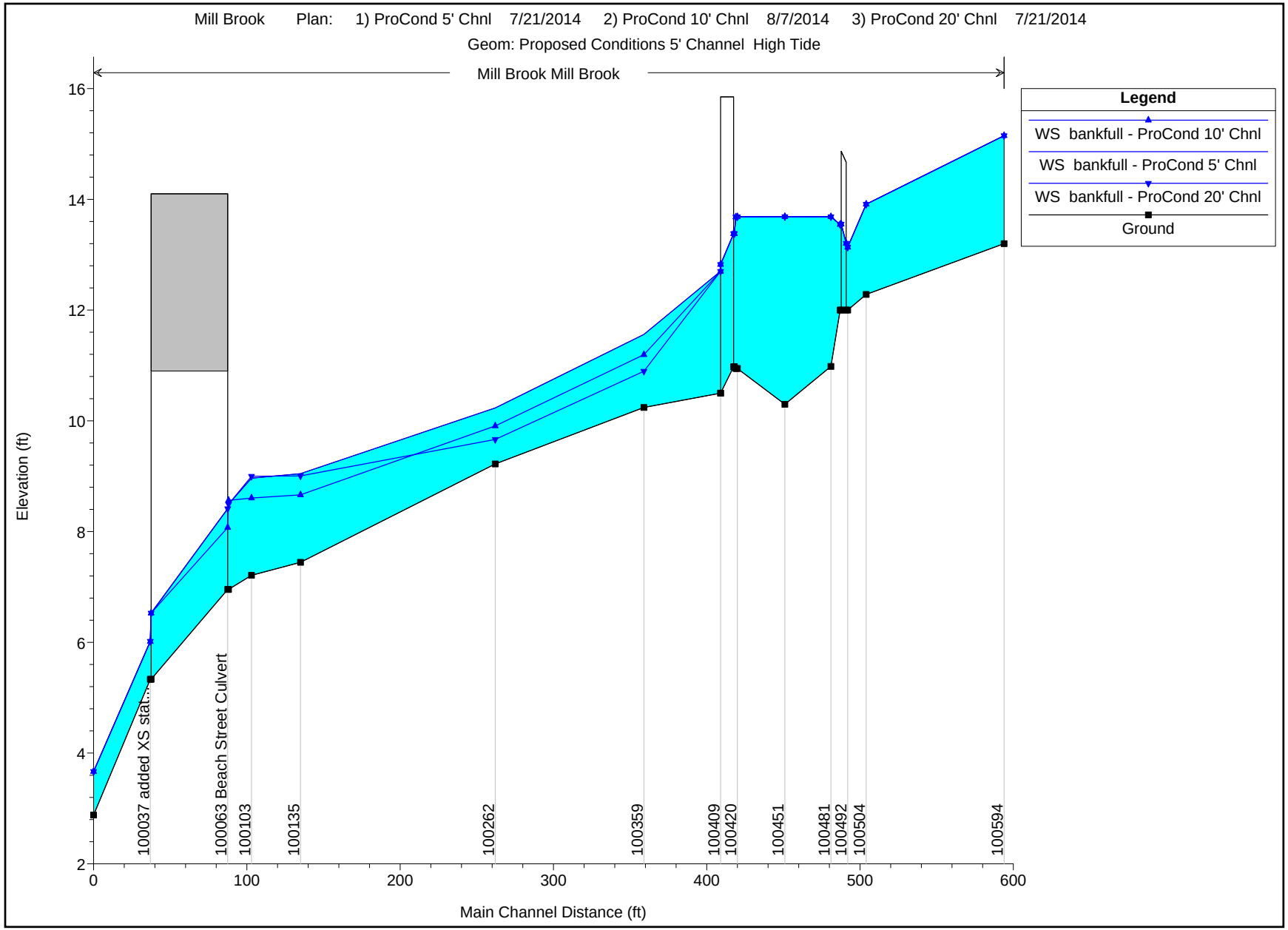
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100594	5 cfs	5.00	13.20	13.90	13.71	13.95	0.005642	1.88	2.66	5.61	0.48
Mill Brook	100594	bankfull	34.00	13.20	15.15	14.46	15.34	0.004025	3.49	9.73	5.66	0.47
Mill Brook	100594	100 cfs	100.00	13.20	15.77	15.85	16.59	0.012901	7.32	14.14	11.87	0.93
Mill Brook	100594	2 low	233.00	13.20	16.66	17.01	17.75	0.012906	9.23	32.75	29.74	0.97
Mill Brook	100594	10 low	405.00	13.20	17.27	17.71	18.45	0.012907	10.35	54.41	41.82	0.99
Mill Brook	100594	100 low	670.00	13.20	17.91	18.20	19.07	0.012905	10.71	85.89	59.92	1.14
Mill Brook	100504	5 cfs	5.00	12.28	13.00		13.14	0.016213	2.99	1.67	3.76	0.79
Mill Brook	100504	bankfull	34.00	12.28	13.91	13.91	14.59	0.020876	6.62	5.13	3.81	1.01
Mill Brook	100504	100 cfs	100.00	12.28	15.34	15.16	15.72	0.005316	5.27	20.69	18.36	0.67
Mill Brook	100504	2 low	233.00	12.28	15.93	16.17	16.72	0.008704	8.07	34.80	35.42	0.90
Mill Brook	100504	10 low	405.00	12.28	16.37	16.69	17.37	0.010127	9.69	56.37	65.31	0.99
Mill Brook	100504	100 low	670.00	12.28	16.75	17.11	17.95	0.011846	11.37	86.74	90.97	1.10
Mill Brook	100492	5 cfs	5.00	12.00	12.84	12.76	12.93	0.015949	2.33	2.15	6.36	0.71
Mill Brook	100492	bankfull	34.00	12.00	14.12	13.41	14.24	0.003797	2.84	11.95	7.82	0.41
Mill Brook	100492	100 cfs	100.00	12.00	15.51	14.30	15.60	0.001557	2.73	46.17	51.38	0.28
Mill Brook	100492	2 low	233.00	12.00	14.99	15.53	16.47	0.028947	10.34	25.52	28.90	1.18
Mill Brook	100492	10 low	405.00	12.00	15.45	15.99	17.11	0.029483	11.72	43.18	48.77	1.22
Mill Brook	100492	100 low	670.00	12.00	15.88	16.48	17.68	0.029119	12.81	68.59	67.82	1.24
Mill Brook	100490		Bridge									
Mill Brook	100487	5 cfs	5.00	12.00	12.59	12.59	12.72	0.029615	2.94	1.70	6.36	1.00
Mill Brook	100487	bankfull	34.00	12.00	14.13		14.22	0.001816	2.38	14.27	8.43	0.32
Mill Brook	100487	100 cfs	100.00	12.00	15.45		15.55	0.001244	2.85	47.47	50.39	0.29
Mill Brook	100487	2 low	233.00	12.00	15.51	15.50	15.98	0.005939	6.31	50.44	52.20	0.63
Mill Brook	100487	10 low	405.00	12.00	15.96	15.96	16.54	0.007006	7.50	76.76	66.15	0.70
Mill Brook	100487	100 low	670.00	12.00	16.28	16.52	17.18	0.010595	9.77	104.98	99.73	0.88
Mill Brook	100481	5 cfs	5.00	10.98	12.39	11.56	12.39	0.000042	0.19	26.86	48.00	0.04
Mill Brook	100481	bankfull	34.00	10.98	14.19		14.19	0.000016	0.29	116.38	52.73	0.03
Mill Brook	100481	100 cfs	100.00	10.98	15.52		15.52	0.000027	0.52	199.62	76.04	0.05
Mill Brook	100481	2 low	233.00	10.98	15.82	12.73	15.84	0.000107	1.08	223.54	81.58	0.10
Mill Brook	100481	10 low	405.00	10.98	16.01	13.11	16.05	0.000270	1.77	240.10	102.45	0.15
Mill Brook	100481	100 low	670.00	10.98	16.35	13.62	16.45	0.000530	2.61	280.18	128.33	0.22
Mill Brook	100451	5 cfs	5.00	10.30	12.39		12.39	0.000001	0.06	88.81	62.82	0.01
Mill Brook	100451	bankfull	34.00	10.30	14.19		14.19	0.000004	0.17	201.82	66.94	0.02
Mill Brook	100451	100 cfs	100.00	10.30	15.52		15.52	0.000009	0.33	311.37	133.65	0.03
Mill Brook	100451	2 low	233.00	10.30	15.83		15.84	0.000038	0.71	357.79	168.05	0.06
Mill Brook	100451	10 low	405.00	10.30	16.02		16.04	0.000095	1.16	392.19	189.56	0.10
Mill Brook	100451	100 low	670.00	10.30	16.39		16.43	0.000182	1.69	465.58	208.26	0.14
Mill Brook	100420	5 cfs	5.00	10.94	12.37	11.37	12.39	0.000166	1.07	4.68	3.34	0.16
Mill Brook	100420	bankfull	34.00	10.94	14.00	12.44	14.17	0.001033	3.35	10.14	3.38	0.34
Mill Brook	100420	100 cfs	100.00	10.94	15.50	14.88	15.52	0.000155	1.52	116.18	164.35	0.13
Mill Brook	100420	2 low	233.00	10.94	15.78	15.24	15.83	0.000347	2.36	163.88	174.70	0.19
Mill Brook	100420	10 low	405.00	10.94	15.92	15.49	16.03	0.000718	3.47	188.76	179.87	0.28
Mill Brook	100420	100 low	670.00	10.94	16.26	15.73	16.41	0.000906	4.07	250.13	189.25	0.31
Mill Brook	100419		Inl Struct									
Mill Brook	100418	5 cfs	5.00	10.98	11.64	11.40	11.73	0.002407	2.37	2.11	3.17	0.51
Mill Brook	100418	bankfull	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.67	3.22	0.51
Mill Brook	100418	100 cfs	100.00	10.98	15.48	14.56	15.52	0.000605	2.31	94.96	160.89	0.21
Mill Brook	100418	2 low	233.00	10.98	15.75	15.47	15.82	0.001198	3.40	139.65	172.82	0.30
Mill Brook	100418	10 low	405.00	10.98	15.86	15.62	16.01	0.002514	5.02	159.47	177.86	0.44
Mill Brook	100418	100 low	670.00	10.98	16.21	15.90	16.38	0.002597	5.40	223.75	189.15	0.45
Mill Brook	100414		Bridge									
Mill Brook	100409	5 cfs	5.00	10.50	11.37	11.37	11.64	0.031310	4.20	1.19	2.24	1.01
Mill Brook	100409	bankfull	34.00	10.50	12.70	12.70	13.58	0.027984	7.53	4.51	2.60	1.01
Mill Brook	100409	100 cfs	100.00	10.50	14.50	14.50	15.20	0.009331	6.70	14.92	10.87	1.01
Mill Brook	100409	2 low	233.00	10.50	15.46	15.46	15.69	0.002678	4.90	93.08	167.14	0.63
Mill Brook	100409	10 low	405.00	10.50	15.64	15.64	15.99	0.003980	6.36	124.28	170.87	0.79
Mill Brook	100409	100 low	670.00	10.50	15.90	15.90	16.34	0.004865	7.60	167.79	175.95	0.89
Mill Brook	100377	5 cfs	5.00	9.79	10.91	10.30	10.96	0.002064	1.79	2.79	2.54	0.30
Mill Brook	100377	bankfull	34.00	9.79	12.32	11.58	12.72	0.007354	5.14	6.87	7.18	0.58
Mill Brook	100377	100 cfs	100.00	9.79	12.58	13.14	14.55	0.035099	11.85	10.03	16.82	1.26
Mill Brook	100377	2 low	233.00	9.79	14.93	13.69	14.97	0.000409	1.93	171.95	129.01	0.15
Mill Brook	100377	10 low	405.00	9.79	15.36	14.31	15.41	0.000559	2.38	228.63	139.50	0.18
Mill Brook	100377	100 low	670.00	9.79	15.88	14.60	15.96	0.000681	2.79	305.18	152.30	0.20
Mill Brook	100375		Bridge									
Mill Brook	100373	5 cfs	5.00	9.55	10.91		10.94	0.001368	1.54	3.24	2.80	0.25

HEC-RAS Plan: EC River: Mill Brook Reach: Mill Brook (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100373	bankfull	34.00	9.55	12.37		12.62	0.004078	4.03	8.49	6.75	0.57
Mill Brook	100373	100 cfs	100.00	9.55	12.60	13.05	14.03	0.020709	9.83	11.68	24.17	1.34
Mill Brook	100373	2 low	233.00	9.55	14.92		14.96	0.000409	2.47	171.18	129.42	0.22
Mill Brook	100373	10 low	405.00	9.55	15.34		15.40	0.000566	3.10	227.75	140.07	0.26
Mill Brook	100373	100 low	670.00	9.55	15.87		15.95	0.000692	3.69	304.44	153.08	0.29
Mill Brook	100359	5 cfs	5.00	9.97	10.66	10.63	10.88	0.017154	3.71	1.35	2.60	0.91
Mill Brook	100359	bankfull	34.00	9.97	11.96	11.96	12.50	0.010689	5.95	5.82	6.89	0.98
Mill Brook	100359	100 cfs	100.00	9.97	12.40	12.79	13.71	0.020120	10.16	15.37	37.82	1.42
Mill Brook	100359	2 low	233.00	9.97	14.94		14.95	0.000136	1.58	247.42	124.17	0.14
Mill Brook	100359	10 low	405.00	9.97	15.36		15.39	0.000236	2.22	301.99	134.26	0.18
Mill Brook	100359	100 low	670.00	9.97	15.88		15.94	0.000354	2.93	375.51	146.59	0.23
Mill Brook	100262	5 cfs	5.00	9.22	10.01		10.07	0.004514	2.00	2.50	5.51	0.52
Mill Brook	100262	bankfull	34.00	9.22	11.27	10.65	11.37	0.001578	2.59	16.66	32.07	0.39
Mill Brook	100262	100 cfs	100.00	9.22	12.45	11.56	12.48	0.000417	2.01	82.88	78.61	0.22
Mill Brook	100262	2 low	233.00	9.22	14.94		14.94	0.000046	1.04	360.12	142.81	0.08
Mill Brook	100262	10 low	405.00	9.22	15.36		15.37	0.000092	1.57	423.16	157.71	0.12
Mill Brook	100262	100 low	670.00	9.22	15.88		15.91	0.000158	2.18	510.27	176.56	0.16
Mill Brook	100239	5 cfs	5.00	9.08	9.93		9.98	0.003088	1.86	2.69	3.45	0.37
Mill Brook	100239	bankfull	34.00	9.08	11.26		11.33	0.001090	2.14	17.80	32.83	0.32
Mill Brook	100239	100 cfs	100.00	9.08	12.45		12.47	0.000219	1.46	89.80	78.81	0.16
Mill Brook	100239	2 low	233.00	9.08	14.93		14.94	0.000030	0.84	344.71	134.03	0.07
Mill Brook	100239	10 low	405.00	9.08	15.35		15.37	0.000056	1.22	402.18	138.59	0.09
Mill Brook	100239	100 low	670.00	9.08	15.87		15.91	0.000090	1.65	475.11	142.27	0.12
Mill Brook	100222	5 cfs	5.00	8.89	9.94		9.95	0.000596	0.82	6.16	8.99	0.16
Mill Brook	100222	bankfull	34.00	8.89	11.28		11.30	0.000332	1.24	36.28	40.10	0.15
Mill Brook	100222	100 cfs	100.00	8.89	12.45		12.47	0.000199	1.30	108.64	76.88	0.13
Mill Brook	100222	2 low	233.00	8.89	14.94		14.94	0.000022	0.63	395.52	147.90	0.05
Mill Brook	100222	10 low	405.00	8.89	15.36		15.37	0.000039	0.89	458.65	151.65	0.06
Mill Brook	100222	100 low	670.00	8.89	15.88		15.90	0.000060	1.16	538.72	155.51	0.08
Mill Brook	100221	5 cfs	5.00	8.90	9.94		9.95	0.000380	0.71	7.09	9.69	0.15
Mill Brook	100221	bankfull	34.00	8.90	11.28		11.30	0.000288	1.22	36.11	38.85	0.15
Mill Brook	100221	100 cfs	100.00	8.90	12.45		12.47	0.000196	1.36	105.44	73.75	0.13
Mill Brook	100221	2 low	233.00	8.90	14.94		14.94	0.000023	0.69	381.29	141.96	0.05
Mill Brook	100221	10 low	405.00	8.90	15.36		15.37	0.000042	0.97	442.84	148.54	0.07
Mill Brook	100221	100 low	670.00	8.90	15.88		15.90	0.000064	1.26	521.13	152.40	0.09
Mill Brook	100196	5 cfs	5.00	8.56	9.90		9.93	0.001812	1.41	3.76	5.45	0.24
Mill Brook	100196	bankfull	34.00	8.56	11.26		11.29	0.000708	1.45	28.83	35.41	0.17
Mill Brook	100196	100 cfs	100.00	8.56	12.44		12.46	0.000364	1.37	84.28	57.26	0.13
Mill Brook	100196	2 low	233.00	8.56	14.93		14.94	0.000062	0.81	279.14	113.00	0.06
Mill Brook	100196	10 low	405.00	8.56	15.34		15.37	0.000113	1.15	329.24	128.18	0.08
Mill Brook	100196	100 low	670.00	8.56	15.85		15.90	0.000170	1.49	399.05	144.76	0.10
Mill Brook	100188	5 cfs	5.00	9.00	9.81		9.90	0.006787	2.29	2.18	3.86	0.54
Mill Brook	100188	bankfull	34.00	9.00	11.24		11.28	0.000775	1.74	22.89	26.03	0.22
Mill Brook	100188	100 cfs	100.00	9.00	12.42		12.46	0.000419	1.76	73.01	54.25	0.18
Mill Brook	100188	2 low	233.00	9.00	14.93		14.94	0.000067	1.05	256.99	105.72	0.08
Mill Brook	100188	10 low	405.00	9.00	15.34		15.36	0.000125	1.50	303.48	121.32	0.11
Mill Brook	100188	100 low	670.00	9.00	15.84		15.90	0.000192	1.96	370.15	141.18	0.14
Mill Brook	100166	5 cfs	5.00	8.70	9.47	9.47	9.64	0.020775	3.39	1.48	4.55	1.04
Mill Brook	100166	bankfull	34.00	8.70	11.20		11.26	0.000858	2.34	17.88	13.39	0.29
Mill Brook	100166	100 cfs	100.00	8.70	12.39		12.44	0.000488	2.39	53.81	39.05	0.23
Mill Brook	100166	2 low	233.00	8.70	14.92		14.94	0.000081	1.43	220.66	102.80	0.10
Mill Brook	100166	10 low	405.00	8.70	15.32		15.36	0.000156	2.08	265.35	119.46	0.15
Mill Brook	100166	100 low	670.00	8.70	15.82		15.89	0.000250	2.77	330.31	141.02	0.19
Mill Brook	100135	5 cfs	5.00	7.30	8.93	8.07	8.98	0.001710	1.78	2.81	1.95	0.26
Mill Brook	100135	bankfull	34.00	7.30	11.18		11.24	0.000717	1.91	17.41	10.50	0.20
Mill Brook	100135	100 cfs	100.00	7.30	12.30		12.42	0.000803	2.58	36.71	20.54	0.32
Mill Brook	100135	2 low	233.00	7.30	14.90		14.93	0.000138	1.84	171.70	89.20	0.15
Mill Brook	100135	10 low	405.00	7.30	15.28		15.35	0.000255	2.64	208.89	105.53	0.21
Mill Brook	100135	100 low	670.00	7.30	15.76		15.87	0.000374	3.40	265.53	128.96	0.25
Mill Brook	100103	5 cfs	5.00	7.20	8.26	8.26	8.79	0.035657	5.81	0.86	0.83	1.01
Mill Brook	100103	bankfull	34.00	7.20	10.64	10.64	11.14	0.010866	5.63	6.04	6.57	1.03
Mill Brook	100103	100 cfs	100.00	7.20	11.59	11.59	12.30	0.006906	6.89	14.82	10.48	0.90
Mill Brook	100103	2 low	233.00	7.20	14.84		14.92	0.000308	2.83	127.52	94.49	0.24
Mill Brook	100103	10 low	405.00	7.20	15.16		15.33	0.000606	4.17	161.15	110.80	0.34
Mill Brook	100103	100 low	670.00	7.20	15.57		15.84	0.000991	5.64	209.47	128.64	0.45
Mill Brook	100088	5 cfs	5.00	6.95	7.31	7.47	7.86	0.130999	5.91	0.85	3.61	2.15
Mill Brook	100088	bankfull	34.00	6.95	7.75	8.41	10.49	0.171734	13.29	2.56	4.32	3.04
Mill Brook	100088	100 cfs	100.00	6.95	8.63	9.52	11.82	0.064850	14.34	6.97	5.61	2.27

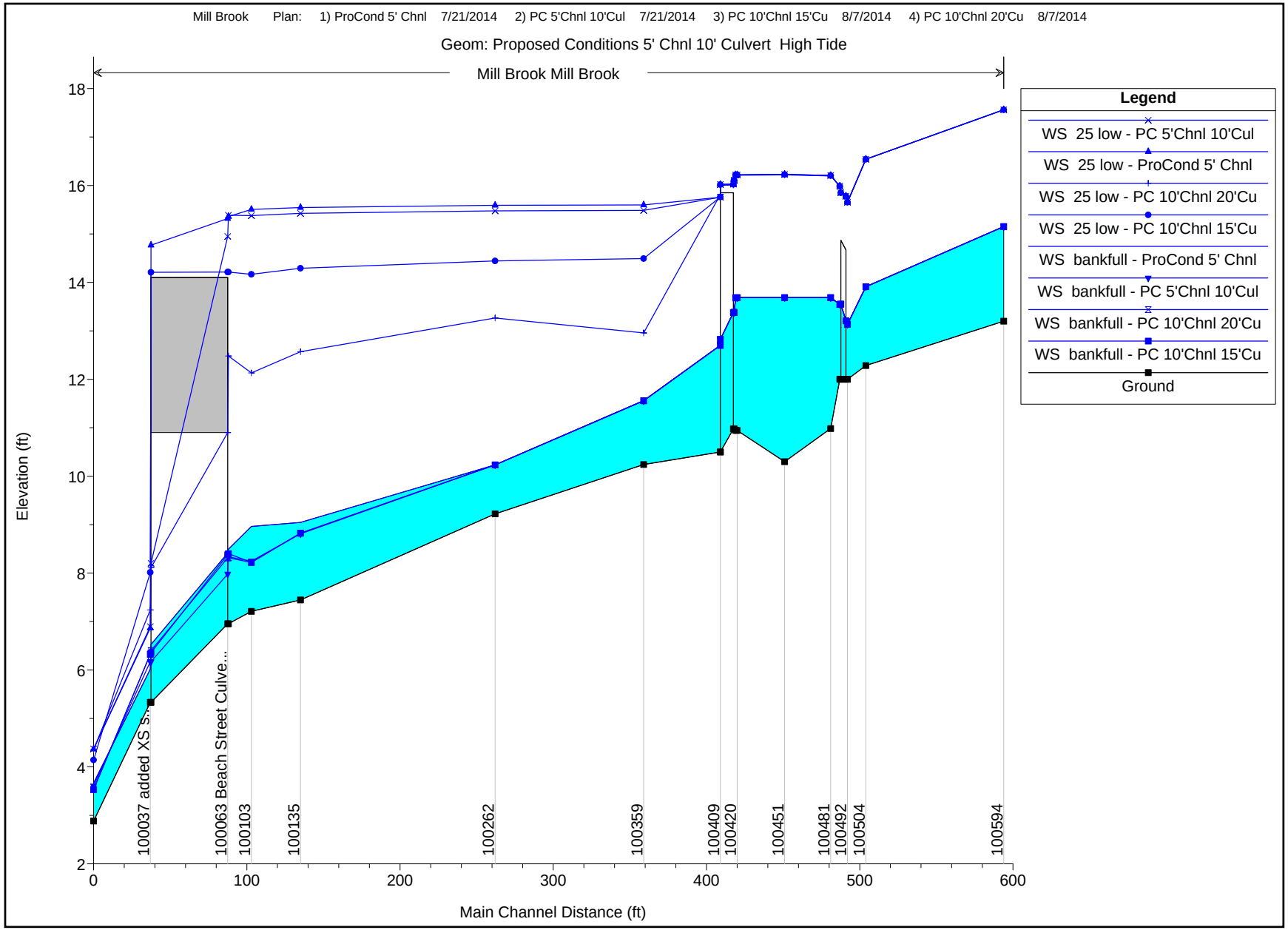
HEC-RAS Plan: EC River: Mill Brook Reach: Mill Brook (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100088	2 low	233.00	6.95	14.77	11.14	14.91	0.000511	3.60	106.33	105.18	0.23
Mill Brook	100088	10 low	405.00	6.95	15.10	14.66	15.31	0.000837	4.75	145.90	128.62	0.30
Mill Brook	100088	100 low	670.00	6.95	15.58	15.15	15.81	0.000952	5.27	213.02	153.10	0.33
Mill Brook	100063		Bridge									
Mill Brook	100037	5 cfs	5.00	5.33	5.67	5.73	5.88	0.089362	3.71	1.35	6.95	1.48
Mill Brook	100037	bankfull	34.00	5.33	6.02	6.32	6.97	0.146041	7.83	4.34	10.45	2.14
Mill Brook	100037	100 cfs	100.00	5.33	6.32	6.83	8.49	0.273609	11.83	8.45	17.61	3.01
Mill Brook	100037	2 low	233.00	5.33	6.48	7.29	12.77	0.712197	20.14	11.57	22.27	4.92
Mill Brook	100037	10 low	405.00	5.33	6.75	7.75	13.45	0.491281	21.14	20.58	38.99	4.34
Mill Brook	100037	100 low	670.00	5.33	7.07	8.34	14.04	0.334410	22.17	33.18	39.96	3.80
Mill Brook	100000	5 cfs	5.00	2.88	3.37	3.38	3.47	0.048566	2.58	2.02	14.33	1.07
Mill Brook	100000	bankfull	34.00	2.88	3.67	3.73	3.88	0.047784	4.09	9.78	38.02	1.19
Mill Brook	100000	100 cfs	100.00	2.88	3.93	4.02	4.23	0.051431	4.74	23.27	64.70	1.28
Mill Brook	100000	2 low	233.00	2.88	4.14	4.31	4.70	0.075402	6.77	39.68	93.84	1.61
Mill Brook	100000	10 low	405.00	2.88	4.29	4.58	5.18	0.089833	8.76	56.02	112.76	1.84
Mill Brook	100000	100 low	670.00	2.88	4.46	4.87	5.81	0.109433	11.09	75.77	132.06	2.10


Mill Brook Plan: 1) ProCond 5' Chnl 7/21/2014 2) ProCond 10' Chnl 8/7/2014 3) ProCond 20' Chnl 7/21/2014
Geom: Proposed Conditions 5' Channel High Tide
Mill Brook Mill Brook
Legend
WS bankfull - ProCond 10' Chnl
WS bankfull - ProCond 5' Chnl
WS bankfull - ProCond 20' Chnl
Ground
Elevation (ft)
Main Channel Distance (ft)
100037 added XS stat...
100063 Beach Street Culvert
100103
100135
100262
100359
100409
100420
100451
100481
100492
100504
100594

HEC-RAS River: Mill Brook Reach: Mill Brook Profile: bankfull

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100594	bankfull	ProCond 5' Chnl	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	ProCond 10' Chnl	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	ProCond 20' Chnl	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100504	bankfull	ProCond 5' Chnl	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	ProCond 10' Chnl	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	ProCond 20' Chnl	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100492	bankfull	ProCond 5' Chnl	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	ProCond 10' Chnl	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	ProCond 20' Chnl	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100490			Bridge									
Mill Brook	100487	bankfull	ProCond 5' Chnl	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.38	8.40	0.60
Mill Brook	100487	bankfull	ProCond 10' Chnl	34.00	12.00	13.55	13.22	13.75	0.007012	3.62	9.39	8.40	0.60
Mill Brook	100487	bankfull	ProCond 20' Chnl	34.00	12.00	13.55	13.22	13.75	0.007012	3.62	9.39	8.40	0.60
Mill Brook	100481	bankfull	ProCond 5' Chnl	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	ProCond 10' Chnl	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	ProCond 20' Chnl	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100451	bankfull	ProCond 5' Chnl	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	bankfull	ProCond 10' Chnl	34.00	10.30	13.69		13.69	0.000006	0.20	170.23	62.87	0.02
Mill Brook	100451	bankfull	ProCond 20' Chnl	34.00	10.30	13.69		13.69	0.000006	0.20	170.23	62.87	0.02
Mill Brook	100420	bankfull	ProCond 5' Chnl	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	ProCond 10' Chnl	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	ProCond 20' Chnl	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100419			Inl Struct									
Mill Brook	100418	bankfull	ProCond 5' Chnl	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.66	3.22	0.51
Mill Brook	100418	bankfull	ProCond 10' Chnl	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.67	3.22	0.51
Mill Brook	100418	bankfull	ProCond 20' Chnl	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.67	3.22	0.51
Mill Brook	100414			Bridge									
Mill Brook	100409	bankfull	ProCond 5' Chnl	34.00	10.50	12.70	12.70	13.58	0.027899	7.53	4.52	2.60	1.01
Mill Brook	100409	bankfull	ProCond 10' Chnl	34.00	10.50	12.70	12.70	13.58	0.028066	7.54	4.51	2.60	1.01
Mill Brook	100409	bankfull	ProCond 20' Chnl	34.00	10.50	12.70	12.70	13.58	0.028066	7.54	4.51	2.60	1.01
Mill Brook	100359	bankfull	ProCond 5' Chnl	34.00	10.24	11.56	11.25	11.78	0.007113	3.73	9.21	8.98	0.62
Mill Brook	100359	bankfull	ProCond 10' Chnl	34.00	10.24	11.19	10.93	11.35	0.007032	3.12	10.89	12.87	0.60
Mill Brook	100359	bankfull	ProCond 20' Chnl	34.00	10.24	10.90	10.68	10.99	0.006467	2.48	13.73	21.97	0.55
Mill Brook	100262	bankfull	ProCond 5' Chnl	34.00	9.22	10.23	10.23	10.65	0.020642	5.18	6.57	7.99	1.01
Mill Brook	100262	bankfull	ProCond 10' Chnl	34.00	9.22	9.91	9.91	10.22	0.021575	4.50	7.55	12.03	1.00
Mill Brook	100262	bankfull	ProCond 20' Chnl	34.00	9.22	9.66	9.66	9.88	0.024205	3.73	9.12	21.31	1.00
Mill Brook	100135	bankfull	ProCond 5' Chnl	34.00	7.44	9.05	8.49	9.18	0.004226	2.96	11.49	9.67	0.48
Mill Brook	100135	bankfull	ProCond 10' Chnl	34.00	7.44	8.66	8.16	8.76	0.003310	2.44	13.96	13.52	0.42
Mill Brook	100135	bankfull	ProCond 20' Chnl	34.00	7.44	9.00	7.91	9.02	0.000370	1.00	34.07	24.55	0.15
Mill Brook	100103	bankfull	ProCond 5' Chnl	34.00	7.21	8.96		9.06	0.002757	2.54	13.41	10.29	0.39
Mill Brook	100103	bankfull	ProCond 10' Chnl	34.00	7.21	8.61		8.67	0.001886	2.01	16.90	14.21	0.33
Mill Brook	100103	bankfull	ProCond 20' Chnl	34.00	7.21	9.00		9.01	0.000217	0.84	40.61	25.39	0.12
Mill Brook	100088	bankfull	ProCond 5' Chnl	34.00	6.95	8.49	8.41	8.95	0.010664	5.45	6.24	5.53	0.90
Mill Brook	100088	bankfull	ProCond 10' Chnl	34.00	6.96	8.56	7.85	8.64	0.001891	2.21	15.41	12.91	0.36
Mill Brook	100088	bankfull	ProCond 20' Chnl	34.00	6.95	8.49	8.41	8.95	0.010664	5.45	6.24	5.53	0.90
Mill Brook	100063			Bridge									
Mill Brook	100037	bankfull	ProCond 5' Chnl	34.00	5.33	6.02	6.32	6.97	0.146041	7.83	4.34	10.45	2.14
Mill Brook	100037	bankfull	ProCond 10' Chnl	34.00	5.33	6.02	6.32	6.98	0.148927	7.90	4.30	10.38	2.16
Mill Brook	100037	bankfull	ProCond 20' Chnl	34.00	5.33	6.02	6.32	6.97	0.146041	7.83	4.34	10.45	2.14
Mill Brook	100000	bankfull	ProCond 5' Chnl	34.00	2.88	3.67	3.73	3.88	0.048111	4.10	9.75	37.94	1.20
Mill Brook	100000	bankfull	ProCond 10' Chnl	34.00	2.88	3.67	3.73	3.88	0.047785	4.09	9.78	38.02	1.19
Mill Brook	100000	bankfull	ProCond 20' Chnl	34.00	2.88	3.67	3.73	3.88	0.048111	4.10	9.75	37.94	1.20


Mill Brook Plan: 1) ProCond 5' Chnl 7/21/2014 2) PC 5'Chnl 10'Cul 7/21/2014 3) PC 10'Chnl 15'Cu 8/7/2014 4) PC 10'Chnl 20'Cu 8/7/2014
Geom: Proposed Conditions 5' Chnl 10' Culvert High Tide
Mill Brook Mill Brook
Legend
WS 25 low - PC 5'Chnl 10'Cul
WS 25 low - ProCond 5' Chnl
WS 25 low - PC 10'Chnl 20'Cu
WS 25 low - PC 10'Chnl 15'Cu
WS bankfull - ProCond 5' Chnl
WS bankfull - PC 5'Chnl 10'Cul
WS bankfull - PC 10'Chnl 20'Cu
WS bankfull - PC 10'Chnl 15'Cu
Ground
Elevation (ft)
Main Channel Distance (ft)
100037 added XS S...
100063 Beach Street Culve...
100103
100135
100262
100359
100409
100420
100451
100481
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100504
100594

HEC-RAS River: Mill Brook Reach: Mill Brook

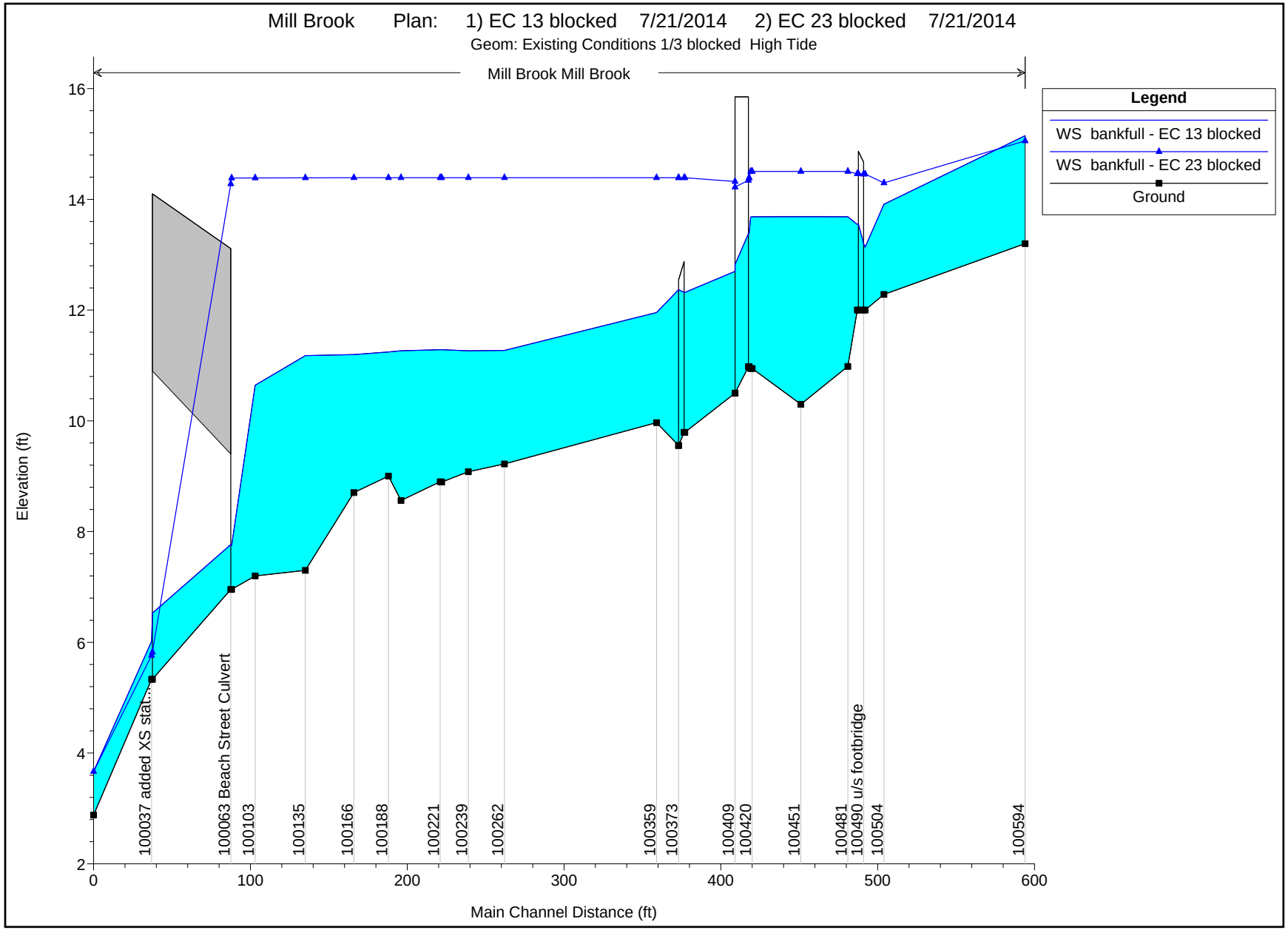
Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100594	bankfull	ProCond 5' Chnl	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	PC 5'Chnl 10'Cul	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	PC 10'Chnl 15'Cu	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	PC 10'Chnl 20'Cu	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	25 low	ProCond 5' Chnl	520.00	13.20	17.56	17.89	18.79	0.012901	10.86	67.60	47.70	0.99
Mill Brook	100594	25 low	PC 5'Chnl 10'Cul	520.00	13.20	17.56	17.89	18.79	0.012901	10.86	67.60	47.70	0.99
Mill Brook	100594	25 low	PC 10'Chnl 15'Cu	520.00	13.20	17.56	17.89	18.79	0.012901	10.86	67.60	47.70	0.99
Mill Brook	100594	25 low	PC 10'Chnl 20'Cu	520.00	13.20	17.56	17.89	18.79	0.012901	10.86	67.60	47.70	0.99
Mill Brook	100504	bankfull	ProCond 5' Chnl	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	PC 5'Chnl 10'Cul	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	PC 10'Chnl 15'Cu	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	PC 10'Chnl 20'Cu	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	25 low	ProCond 5' Chnl	520.00	12.28	16.54	16.90	17.67	0.011301	10.64	68.98	78.57	1.06
Mill Brook	100504	25 low	PC 5'Chnl 10'Cul	520.00	12.28	16.54	16.90	17.67	0.011301	10.64	68.98	78.57	1.06
Mill Brook	100504	25 low	PC 10'Chnl 15'Cu	520.00	12.28	16.54	16.90	17.67	0.011301	10.64	68.98	78.57	1.06
Mill Brook	100504	25 low	PC 10'Chnl 20'Cu	520.00	12.28	16.54	16.90	17.67	0.011301	10.64	68.98	78.57	1.06
Mill Brook	100492	bankfull	ProCond 5' Chnl	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	PC 5'Chnl 10'Cul	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	PC 10'Chnl 15'Cu	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	PC 10'Chnl 20'Cu	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	25 low	ProCond 5' Chnl	520.00	12.00	15.66	16.25	17.40	0.029577	12.30	54.32	57.90	1.24
Mill Brook	100492	25 low	PC 5'Chnl 10'Cul	520.00	12.00	15.66	16.25	17.40	0.029577	12.30	54.32	57.90	1.24
Mill Brook	100492	25 low	PC 10'Chnl 15'Cu	520.00	12.00	15.66	16.25	17.40	0.029577	12.30	54.32	57.90	1.24
Mill Brook	100492	25 low	PC 10'Chnl 20'Cu	520.00	12.00	15.66	16.25	17.40	0.029577	12.30	54.32	57.90	1.24
Mill Brook	100490			Bridge									
Mill Brook	100487	bankfull	ProCond 5' Chnl	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.38	8.40	0.60
Mill Brook	100487	bankfull	PC 5'Chnl 10'Cul	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.38	8.40	0.60
Mill Brook	100487	bankfull	PC 10'Chnl 15'Cu	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.38	8.40	0.60
Mill Brook	100487	bankfull	PC 10'Chnl 20'Cu	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.38	8.40	0.60
Mill Brook	100487	25 low	ProCond 5' Chnl	520.00	12.00	15.99	16.31	16.90	0.010883	9.40	78.74	67.08	0.88
Mill Brook	100487	25 low	PC 5'Chnl 10'Cul	520.00	12.00	15.99	16.31	16.90	0.010883	9.40	78.74	67.08	0.88
Mill Brook	100487	25 low	PC 10'Chnl 15'Cu	520.00	12.00	15.99	16.31	16.90	0.010883	9.40	78.74	67.08	0.88
Mill Brook	100487	25 low	PC 10'Chnl 20'Cu	520.00	12.00	15.99	16.31	16.90	0.010883	9.40	78.74	67.08	0.88
Mill Brook	100481	bankfull	ProCond 5' Chnl	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	PC 5'Chnl 10'Cul	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	PC 10'Chnl 15'Cu	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	PC 10'Chnl 20'Cu	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	25 low	ProCond 5' Chnl	520.00	10.98	16.21	13.34	16.27	0.000369	2.13	262.03	117.33	0.18
Mill Brook	100481	25 low	PC 5'Chnl 10'Cul	520.00	10.98	16.21	13.34	16.27	0.000369	2.13	262.03	117.33	0.18
Mill Brook	100481	25 low	PC 10'Chnl 15'Cu	520.00	10.98	16.21	13.34	16.27	0.000369	2.13	261.85	117.21	0.18
Mill Brook	100481	25 low	PC 10'Chnl 20'Cu	520.00	10.98	16.20	13.34	16.27	0.000372	2.14	260.97	116.65	0.18
Mill Brook	100451	bankfull	ProCond 5' Chnl	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	bankfull	PC 5'Chnl 10'Cul	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	bankfull	PC 10'Chnl 15'Cu	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	bankfull	PC 10'Chnl 20'Cu	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	25 low	ProCond 5' Chnl	520.00	10.30	16.23		16.26	0.000128	1.39	432.91	202.46	0.11
Mill Brook	100451	25 low	PC 5'Chnl 10'Cul	520.00	10.30	16.23		16.26	0.000128	1.39	432.91	202.46	0.11
Mill Brook	100451	25 low	PC 10'Chnl 15'Cu	520.00	10.30	16.23		16.25	0.000128	1.39	432.60	202.39	0.11
Mill Brook	100451	25 low	PC 10'Chnl 20'Cu	520.00	10.30	16.22		16.25	0.000129	1.39	431.09	202.03	0.11
Mill Brook	100420	bankfull	ProCond 5' Chnl	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	PC 5'Chnl 10'Cul	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	PC 10'Chnl 15'Cu	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	PC 10'Chnl 20'Cu	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	25 low	ProCond 5' Chnl	520.00	10.94	16.22	12.92	16.25	0.000195	1.76	383.06	188.36	0.14
Mill Brook	100420	25 low	PC 5'Chnl 10'Cul	520.00	10.94	16.22	12.92	16.25	0.000195	1.76	383.06	188.36	0.14
Mill Brook	100420	25 low	PC 10'Chnl 15'Cu	520.00	10.94	16.22	12.92	16.25	0.000196	1.77	382.77	188.32	0.14
Mill Brook	100420	25 low	PC 10'Chnl 20'Cu	520.00	10.94	16.21	12.92	16.24	0.000197	1.77	381.34	188.12	0.14
Mill Brook	100419			Inl Struct									
Mill Brook	100418	bankfull	ProCond 5' Chnl	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.66	3.22	0.51
Mill Brook	100418	bankfull	PC 5'Chnl 10'Cul	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.66	3.22	0.51
Mill Brook	100418	bankfull	PC 10'Chnl 15'Cu	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.66	3.22	0.51
Mill Brook	100418	bankfull	PC 10'Chnl 20'Cu	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.66	3.22	0.51
Mill Brook	100418	25 low	ProCond 5' Chnl	520.00	10.98	16.10	15.74	16.24	0.002054	4.72	204.00	186.33	0.40
Mill Brook	100418	25 low	PC 5'Chnl 10'Cul	520.00	10.98	16.10	15.74	16.24	0.002054	4.72	204.00	186.33	0.40
Mill Brook	100418	25 low	PC 10'Chnl 15'Cu	520.00	10.98	16.10	15.74	16.23	0.002062	4.73	203.71	186.29	0.40
Mill Brook	100418	25 low	PC 10'Chnl 20'Cu	520.00	10.98	16.10	15.74	16.23	0.002105	4.77	202.28	186.09	0.41
Mill Brook	100414			Bridge									
Mill Brook	100409	bankfull	ProCond 5' Chnl	34.00	10.50	12.70	12.70	13.58	0.027899	7.53	4.52	2.60	1.01
Mill Brook	100409	bankfull	PC 5'Chnl 10'Cul	34.00	10.50	12.70	12.70	13.58	0.027916	7.53	4.52	2.60	1.01
Mill Brook	100409	bankfull	PC 10'Chnl 15'Cu	34.00	10.50	12.70	12.70	13.58	0.027916	7.53	4.52	2.60	1.01
Mill Brook	100409	bankfull	PC 10'Chnl 20'Cu	34.00	10.50	12.70	12.70	13.58	0.027916	7.53	4.52	2.60	1.01
Mill Brook	100409	25 low	ProCond 5' Chnl	520.00	10.50	15.76	15.76	16.15	0.004470	6.99	143.76	173.17	0.84
Mill Brook	100409	25 low	PC 5'Chnl 10'Cul	520.00	10.50	15.76	15.76	16.15	0.004470	6.99	143.76	173.17	0.84
Mill Brook	100409	25 low	PC 10'Chnl 15'Cu	520.00	10.50	15.76	15.76	16.15	0.004392	6.94	144.71	173.28	0.83
Mill Brook	100409	25 low	PC 10'Chnl 20'Cu	520.00	10.50	15.81	15.81	16.15	0.003748	6.51	153.42	174.29	0.77
Mill Brook	100359	bankfull	ProCond 5' Chnl	34.00	10.24	11.56	11.25	11.78	0.007113	3.73	9.21	8.98	0.62
Mill Brook	100359	bankfull	PC 5'Chnl 10'Cul	34.00	10.24	11.56	11.25	11.77	0.007209	3.74	9.17	8.96	0.62

HEC-RAS River: Mill Brook Reach: Mill Brook (Continued)

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100359	bankfull	PC 10'Chnl 15'Cu	34.00	10.24	11.56	11.25	11.77	0.007185	3.74	9.18	8.97	0.62
Mill Brook	100359	bankfull	PC 10'Chnl 20'Cu	34.00	10.24	11.56	11.25	11.77	0.007185	3.74	9.18	8.97	0.62
Mill Brook	100359	25 low	ProCond 5' Chnl	520.00	10.24	15.60	13.53	15.64	0.000216	1.79	336.95	139.91	0.14
Mill Brook	100359	25 low	PC 5'Chnl 10'Cul	520.00	10.24	15.49	13.53	15.53	0.000246	1.88	321.38	137.26	0.15
Mill Brook	100359	25 low	PC 10'Chnl 15'Cu	520.00	10.24	14.49	13.53	14.60	0.000964	3.22	196.64	112.61	0.28
Mill Brook	100359	25 low	PC 10'Chnl 20'Cu	520.00	10.24	12.96	13.53	15.48	0.038694	14.85	50.07	78.37	1.65
Mill Brook	100262	bankfull	ProCond 5' Chnl	34.00	9.22	10.23	10.23	10.65	0.020642	5.18	6.57	7.99	1.01
Mill Brook	100262	bankfull	PC 5'Chnl 10'Cul	34.00	9.22	10.23	10.23	10.65	0.020642	5.18	6.57	7.99	1.01
Mill Brook	100262	bankfull	PC 10'Chnl 15'Cu	34.00	9.22	10.23	10.23	10.65	0.020642	5.18	6.57	7.99	1.01
Mill Brook	100262	bankfull	PC 10'Chnl 20'Cu	34.00	9.22	10.23	10.23	10.65	0.020642	5.18	6.57	7.99	1.01
Mill Brook	100262	25 low	ProCond 5' Chnl	520.00	9.22	15.59		15.62	0.000139	1.62	407.64	166.25	0.11
Mill Brook	100262	25 low	PC 5'Chnl 10'Cul	520.00	9.22	15.48		15.51	0.000157	1.70	388.95	162.15	0.12
Mill Brook	100262	25 low	PC 10'Chnl 15'Cu	520.00	9.22	14.44		14.52	0.000566	2.85	239.63	128.96	0.22
Mill Brook	100262	25 low	PC 10'Chnl 20'Cu	520.00	9.22	13.27	13.35	13.83	0.005132	7.18	104.73	103.05	0.65
Mill Brook	100135	bankfull	ProCond 5' Chnl	34.00	7.44	9.05	8.49	9.18	0.004226	2.96	11.49	9.67	0.48
Mill Brook	100135	bankfull	PC 5'Chnl 10'Cul	34.00	7.44	8.81	8.49	9.02	0.007674	3.66	9.30	8.96	0.63
Mill Brook	100135	bankfull	PC 10'Chnl 15'Cu	34.00	7.44	8.82	8.49	9.03	0.007448	3.62	9.39	9.00	0.62
Mill Brook	100135	bankfull	PC 10'Chnl 20'Cu	34.00	7.44	8.82	8.49	9.03	0.007429	3.62	9.40	9.00	0.62
Mill Brook	100135	25 low	ProCond 5' Chnl	520.00	7.44	15.54		15.60	0.000158	1.94	293.56	118.32	0.12
Mill Brook	100135	25 low	PC 5'Chnl 10'Cul	520.00	7.44	15.43		15.48	0.000179	2.04	279.77	112.53	0.13
Mill Brook	100135	25 low	PC 10'Chnl 15'Cu	520.00	7.44	14.29		14.44	0.000583	3.30	177.61	73.55	0.23
Mill Brook	100135	25 low	PC 10'Chnl 20'Cu	520.00	7.44	12.57	11.64	13.15	0.003110	6.14	87.40	32.24	0.51
Mill Brook	100103	bankfull	ProCond 5' Chnl	34.00	7.21	8.96		9.06	0.002757	2.54	13.41	10.29	0.39
Mill Brook	100103	bankfull	PC 5'Chnl 10'Cul	34.00	7.21	8.25	8.22	8.63	0.018937	5.00	6.80	8.13	0.96
Mill Brook	100103	bankfull	PC 10'Chnl 15'Cu	34.00	7.21	8.22	8.22	8.63	0.020520	5.14	6.61	8.06	1.00
Mill Brook	100103	bankfull	PC 10'Chnl 20'Cu	34.00	7.21	8.22	8.22	8.63	0.020655	5.15	6.60	8.06	1.00
Mill Brook	100103	25 low	ProCond 5' Chnl	520.00	7.21	15.51		15.59	0.000261	2.52	270.58	125.95	0.16
Mill Brook	100103	25 low	PC 5'Chnl 10'Cul	520.00	7.21	15.38		15.47	0.000295	2.65	255.06	120.38	0.17
Mill Brook	100103	25 low	PC 10'Chnl 15'Cu	520.00	7.21	14.17		14.41	0.000831	3.96	145.63	57.15	0.28
Mill Brook	100103	25 low	PC 10'Chnl 20'Cu	520.00	7.21	12.13		13.00	0.004956	7.50	71.53	27.27	0.63
Mill Brook	100088	bankfull	ProCond 5' Chnl	34.00	6.95	8.49	8.41	8.95	0.010664	5.45	6.24	5.53	0.90
Mill Brook	100088	bankfull	PC 5'Chnl 10'Cul	34.00	6.96	8.33	7.85	8.45	0.003482	2.71	12.54	12.09	0.47
Mill Brook	100088	bankfull	PC 10'Chnl 15'Cu	34.00	6.96	8.40	7.78	8.45	0.001879	1.79	19.03	21.12	0.33
Mill Brook	100088	bankfull	PC 10'Chnl 20'Cu	34.00	6.96	8.32	7.78	8.38	0.002456	1.97	17.27	20.22	0.38
Mill Brook	100088	25 low	ProCond 5' Chnl	520.00	6.95	15.36	14.91	15.57	0.000855	4.91	180.14	141.50	0.31
Mill Brook	100088	25 low	PC 5'Chnl 10'Cul	520.00	6.96	15.39	11.01	15.46	0.000198	2.54	252.21	142.90	0.16
Mill Brook	100088	25 low	PC 10'Chnl 15'Cu	520.00	6.96	14.21	10.09	14.38	0.000450	3.31	163.92	60.69	0.23
Mill Brook	100088	25 low	PC 10'Chnl 20'Cu	520.00	6.96	12.48	10.09	12.80	0.001272	4.55	115.49	24.49	0.36
Mill Brook	100063			Bridge									
Mill Brook	100037	bankfull	ProCond 5' Chnl	34.00	5.33	6.02	6.32	6.97	0.146041	7.83	4.34	10.45	2.14
Mill Brook	100037	bankfull	PC 5'Chnl 10'Cul	34.00	5.33	6.17	6.32	6.64	0.063496	5.49	6.19	13.59	1.43
Mill Brook	100037	bankfull	PC 10'Chnl 15'Cu	34.00	5.33	6.33	6.33	6.57	0.030607	3.96	8.58	17.81	1.01
Mill Brook	100037	bankfull	PC 10'Chnl 20'Cu	34.00	5.33	6.33	6.33	6.57	0.030607	3.96	8.58	17.81	1.01
Mill Brook	100037	25 low	ProCond 5' Chnl	520.00	5.33	6.86	8.02	14.43	0.472771	22.76	24.90	39.32	4.36
Mill Brook	100037	25 low	PC 5'Chnl 10'Cul	520.00	5.33	6.90	8.02	13.75	0.410878	21.74	26.13	39.42	4.08
Mill Brook	100037	25 low	PC 10'Chnl 15'Cu	520.00	5.33	8.02	8.02	8.87	0.018092	7.88	72.16	42.84	0.98
Mill Brook	100037	25 low	PC 10'Chnl 20'Cu	520.00	5.33	7.24	8.02	10.11	0.114885	14.32	39.86	40.47	2.28
Mill Brook	100000	bankfull	ProCond 5' Chnl	34.00	2.88	3.67	3.73	3.88	0.048111	4.10	9.75	37.94	1.20
Mill Brook	100000	bankfull	PC 5'Chnl 10'Cul	34.00	2.88	3.61	3.73	3.97	0.080773	5.27	7.66	32.01	1.55
Mill Brook	100000	bankfull	PC 10'Chnl 15'Cu	34.00	2.88	3.53	3.73	4.23	0.186077	7.18	5.50	26.77	2.29
Mill Brook	100000	bankfull	PC 10'Chnl 20'Cu	34.00	2.88	3.53	3.73	4.23	0.186077	7.18	5.50	26.77	2.29
Mill Brook	100000	25 low	ProCond 5' Chnl	520.00	2.88	4.37	4.71	5.48	0.101376	9.93	64.51	121.43	1.98
Mill Brook	100000	25 low	PC 5'Chnl 10'Cul	520.00	2.88	4.37	4.71	5.46	0.098269	9.83	65.27	122.17	1.96
Mill Brook	100000	25 low	PC 10'Chnl 15'Cu	520.00	2.88	4.14	4.71	6.89	0.362301	14.95	40.21	94.51	3.54
Mill Brook	100000	25 low	PC 10'Chnl 20'Cu	520.00	2.88	4.36	4.71	5.51	0.106288	10.09	63.38	120.32	2.03

HEC-RAS River: Mill Brook Reach: Mill Brook Profile: bankfull

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mill Brook	100594	bankfull	EC 13 blocked	34.00	13.20	15.15	14.46	15.34	0.004036	3.50	9.72	5.66	0.47
Mill Brook	100594	bankfull	EC 23 blocked	34.00	13.20	15.06	14.46	15.27	0.004825	3.70	9.19	5.66	0.51
Mill Brook	100504	bankfull	EC 13 blocked	34.00	12.28	13.91	13.91	14.59	0.020869	6.62	5.14	3.81	1.01
Mill Brook	100504	bankfull	EC 23 blocked	34.00	12.28	14.30		14.67	0.009014	4.99	7.12	6.81	0.67
Mill Brook	100492	bankfull	EC 13 blocked	34.00	12.00	13.14	13.41	14.10	0.096384	7.90	4.31	7.75	1.87
Mill Brook	100492	bankfull	EC 23 blocked	34.00	12.00	14.46	13.41	14.54	0.001995	2.31	14.98	12.70	0.30
Mill Brook	100490			Bridge									
Mill Brook	100487	bankfull	EC 13 blocked	34.00	12.00	13.55	13.22	13.75	0.007013	3.62	9.39	8.40	0.60
Mill Brook	100487	bankfull	EC 23 blocked	34.00	12.00	14.46		14.52	0.001023	1.99	17.09	8.44	0.25
Mill Brook	100481	bankfull	EC 13 blocked	34.00	10.98	13.69	12.04	13.69	0.000036	0.37	91.02	49.72	0.05
Mill Brook	100481	bankfull	EC 23 blocked	34.00	10.98	14.50		14.50	0.000010	0.26	133.29	54.40	0.03
Mill Brook	100451	bankfull	EC 13 blocked	34.00	10.30	13.69		13.69	0.000006	0.20	170.22	62.87	0.02
Mill Brook	100451	bankfull	EC 23 blocked	34.00	10.30	14.50		14.50	0.000003	0.15	223.04	67.57	0.01
Mill Brook	100420	bankfull	EC 13 blocked	34.00	10.94	13.68	11.41	13.69	0.000017	0.33	108.67	46.36	0.04
Mill Brook	100420	bankfull	EC 23 blocked	34.00	10.94	14.50	11.41	14.50	0.000006	0.24	148.79	52.51	0.02
Mill Brook	100419			Inl Struct									
Mill Brook	100418	bankfull	EC 13 blocked	34.00	10.98	13.38	12.50	13.69	0.004531	4.44	7.67	3.22	0.51
Mill Brook	100418	bankfull	EC 23 blocked	34.00	10.98	14.39	12.50	14.50	0.001352	2.76	13.51	10.08	0.30
Mill Brook	100414			Bridge									
Mill Brook	100409	bankfull	EC 13 blocked	34.00	10.50	12.70	12.70	13.58	0.027984	7.53	4.51	2.60	1.01
Mill Brook	100409	bankfull	EC 23 blocked	34.00	10.50	14.32		14.43	0.001559	2.59	13.10	9.85	0.40
Mill Brook	100377	bankfull	EC 13 blocked	34.00	9.79	12.32	11.58	12.72	0.007354	5.14	6.87	7.18	0.58
Mill Brook	100377	bankfull	EC 23 blocked	34.00	9.79	14.39	11.58	14.39	0.000032	0.50	105.81	114.05	0.04
Mill Brook	100375			Bridge									
Mill Brook	100373	bankfull	EC 13 blocked	34.00	9.55	12.37		12.62	0.004078	4.03	8.49	6.75	0.57
Mill Brook	100373	bankfull	EC 23 blocked	34.00	9.55	14.39		14.39	0.000030	0.61	106.01	114.64	0.06
Mill Brook	100359	bankfull	EC 13 blocked	34.00	9.97	11.96	11.96	12.50	0.010689	5.95	5.82	6.89	0.98
Mill Brook	100359	bankfull	EC 23 blocked	34.00	9.97	14.39		14.39	0.000007	0.32	183.47	110.02	0.03
Mill Brook	100262	bankfull	EC 13 blocked	34.00	9.22	11.27	10.65	11.37	0.001578	2.59	16.66	32.07	0.39
Mill Brook	100262	bankfull	EC 23 blocked	34.00	9.22	14.39		14.39	0.000002	0.19	286.49	127.50	0.02
Mill Brook	100239	bankfull	EC 13 blocked	34.00	9.08	11.26		11.33	0.001090	2.14	17.80	32.83	0.32
Mill Brook	100239	bankfull	EC 23 blocked	34.00	9.08	14.39		14.39	0.000001	0.16	276.41	116.98	0.01
Mill Brook	100222	bankfull	EC 13 blocked	34.00	8.89	11.28		11.30	0.000332	1.24	36.28	40.10	0.15
Mill Brook	100222	bankfull	EC 23 blocked	34.00	8.89	14.39		14.39	0.000001	0.12	317.45	138.52	0.01
Mill Brook	100221	bankfull	EC 13 blocked	34.00	8.90	11.28		11.30	0.000288	1.22	36.11	38.85	0.15
Mill Brook	100221	bankfull	EC 23 blocked	34.00	8.90	14.39		14.39	0.000001	0.14	306.35	133.23	0.01
Mill Brook	100196	bankfull	EC 13 blocked	34.00	8.56	11.26		11.29	0.000708	1.45	28.83	35.41	0.17
Mill Brook	100196	bankfull	EC 23 blocked	34.00	8.56	14.39		14.39	0.000003	0.15	223.61	93.33	0.01
Mill Brook	100188	bankfull	EC 13 blocked	34.00	9.00	11.24		11.28	0.000775	1.74	22.89	26.03	0.22
Mill Brook	100188	bankfull	EC 23 blocked	34.00	9.00	14.39		14.39	0.000003	0.20	205.19	87.92	0.02
Mill Brook	100166	bankfull	EC 13 blocked	34.00	8.70	11.20		11.26	0.000858	2.34	17.88	13.39	0.29
Mill Brook	100166	bankfull	EC 23 blocked	34.00	8.70	14.39		14.39	0.000003	0.27	171.04	84.99	0.02
Mill Brook	100135	bankfull	EC 13 blocked	34.00	7.30	11.18		11.24	0.000717	1.91	17.41	10.50	0.20
Mill Brook	100135	bankfull	EC 23 blocked	34.00	7.30	14.39		14.39	0.000006	0.35	129.84	76.09	0.03
Mill Brook	100103	bankfull	EC 13 blocked	34.00	7.20	10.64	10.64	11.14	0.010866	5.63	6.04	6.57	1.03
Mill Brook	100103	bankfull	EC 23 blocked	34.00	7.20	14.39		14.39	0.000012	0.51	90.67	69.41	0.05
Mill Brook	100088	bankfull	EC 13 blocked	34.00	6.95	7.75	8.41	10.49	0.171734	13.29	2.56	4.32	3.04
Mill Brook	100088	bankfull	EC 23 blocked	34.00	6.95	14.38	8.41	14.39	0.000020	0.69	71.62	74.95	0.05
Mill Brook	100063			Bridge									
Mill Brook	100037	bankfull	EC 13 blocked	34.00	5.33	6.02	6.32	6.97	0.146042	7.83	4.34	10.45	2.14
Mill Brook	100037	bankfull	EC 23 blocked	34.00	5.33	5.77	6.32	9.99	1.179689	16.49	2.06	7.81	5.66
Mill Brook	100000	bankfull	EC 13 blocked	34.00	2.88	3.67	3.73	3.88	0.047783	4.09	9.78	38.02	1.19
Mill Brook	100000	bankfull	EC 23 blocked	34.00	2.88	3.67	3.73	3.88	0.047887	4.09	9.77	38.00	1.20


Mill Brook Plan: 1) EC 13 blocked 7/21/2014 2) EC 23 blocked 7/21/2014
Geom: Existing Conditions 1/3 blocked High Tide
Mill Brook Mill Brook
Legend
WS bankfull - EC 13 blocked
WS bankfull - EC 23 blocked
Ground
Elevation (ft)
Main Channel Distance (ft)
100037 added XS stat...
100063 Beach Street Culvert
100103
100135
100166
100188
100221
100239
100262
100359
100373
100409
100420
100451
100481
100490 u/s footbridge
100504
100594

Plan: PC 5'Chnl 10'Cul Mill Brook Mill Brook RS: 100063 Profile: 25 low

E.G. US. (ft)	15.46	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	15.39	E.G. Elev (ft)	15.42	13.97
Q Total (cfs)	520.00	W.S. Elev (ft)	14.95	8.20
Q Bridge (cfs)	133.59	Crit W.S. (ft)	14.95	9.86
Q Weir (cfs)		Max Chl Dpth (ft)	7.99	2.87
Weir Sta Lft (ft)		Vel Total (ft/s)	5.07	19.27
Weir Sta Rgt (ft)		Flow Area (sq ft)	102.65	26.99
Weir Submerg		Froude # Chl	0.34	2.00
Weir Max Depth (ft)		Specif Force (cu ft)	291.34	346.52
Min El Weir Flow (ft)	14.11	Hydr Depth (ft)	0.86	2.62
Min El Prs (ft)	10.90	W.P. Total (ft)	143.37	15.16
Delta EG (ft)	1.71	Conv. Total (cfs)	6211.7	1472.5
Delta WS (ft)	8.49	Top Width (ft)	119.30	10.28
BR Open Area (sq ft)	29.90	Frctn Loss (ft)		0.92
BR Open Vel (ft/s)	4.95	C & E Loss (ft)		0.53
Coef of Q		Shear Total (lb/sq ft)	0.31	13.86
Br Sel Method	Energy only	Power Total (lb/ft s)	-51.00	-12.20

Errors Warnings and Notes

Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Warning:	The velocity head has changed by more than 0.5 ft (0.15 m). This may indicate the need for additional cross sections.
Warning:	The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4.
	This may indicate the need for additional cross sections.
Warning:	The energy loss was greater than 1.0 ft (0.3 m). between the current and previous cross section. This may indicate the
	need for additional cross sections.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	The energy method has computed a class B profile.

Plan: ProCond 10' Chnl Mill Brook Mill Brook RS: 100063 Profile: 100 low

E.G. US. (ft)	15.60	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	15.49	E.G. Elev (ft)	15.60	15.41
Q Total (cfs)	670.00	W.S. Elev (ft)	15.49	14.92
Q Bridge (cfs)	215.03	Crit W.S. (ft)	15.25	14.92
Q Weir (cfs)	454.97	Max Chl Dpth (ft)	8.52	9.59
Weir Sta Lft (ft)	-22.02	Vel Total (ft/s)	3.66	3.38
Weir Sta Rgt (ft)	132.13	Flow Area (sq ft)	183.00	198.22
Weir Submerg	0.00	Froude # Chl	0.28	0.32
Weir Max Depth (ft)	1.50	Specif Force (cu ft)	316.04	348.77
Min El Weir Flow (ft)	14.11	Hydr Depth (ft)	1.24	1.75
Min El Prs (ft)	10.90	W.P. Total (ft)	167.69	135.28
Delta EG (ft)	0.94	Conv. Total (cfs)		
Delta WS (ft)	7.15	Top Width (ft)	148.03	113.05
BR Open Area (sq ft)	20.80	Frctn Loss (ft)		
BR Open Vel (ft/s)	10.34	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	-51.00	-12.20

Errors Warnings and Notes

Note:	Momentum answer is not valid if the water surface is above the low chord or if there is weir flow. The momentum
	answer has been disregarded.
Note:	The downstream water surface is below the minimum elevation for pressure flow. The sluice gate equations were used
	for pressure flow.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the upstream end, the water surface and energy have been projected from
	the upstream cross section. The selected bridge modeling method does not compute answers inside the bridge.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the downstream end, the water surface and energy are based on critical
	depth over the weir.

Plan: PC 10'ch 10'culv Mill Brook Mill Brook RS: 100063 Profile: 100 low

E.G. US. (ft)	15.27	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	15.09	E.G. Elev (ft)	15.27	14.98
Q Total (cfs)	670.00	W.S. Elev (ft)	15.09	14.69
Q Bridge (cfs)	382.32	Crit W.S. (ft)	15.04	10.66
Q Weir (cfs)	287.68	Max Chl Dpth (ft)	8.14	9.36
Weir Sta Lft (ft)	-15.45	Vel Total (ft/s)	4.39	3.59
Weir Sta Rgt (ft)	121.69	Flow Area (sq ft)	152.61	186.44
Weir Submerg	0.00	Froude # Chl	0.36	0.35
Weir Max Depth (ft)	1.17	Specif Force (cu ft)	388.08	443.21
Min El Weir Flow (ft)	14.11	Hydr Depth (ft)	1.19	1.65
Min El Prs (ft)	10.90	W.P. Total (ft)	156.18	143.99
Delta EG (ft)	1.26	Conv. Total (cfs)		
Delta WS (ft)	6.75	Top Width (ft)	128.06	112.67
BR Open Area (sq ft)	38.21	Frctn Loss (ft)		
BR Open Vel (ft/s)	10.01	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	-51.00	-12.20

Errors Warnings and Notes

Note:	Momentum answer is not valid if the water surface is above the low chord or if there is weir flow. The momentum
	answer has been disregarded.
Note:	The downstream water surface is below the minimum elevation for pressure flow. The sluice gate equations were used
	for pressure flow.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the upstream end, the water surface and energy have been projected from
	the upstream cross section. The selected bridge modeling method does not compute answers inside the bridge.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the downstream end, the water surface and energy are based on critical
	depth over the weir.

Plan: PC 10'Chnl 15'Cu Mill Brook Mill Brook RS: 100063 Profile: 100 low

E.G. US. (ft)	14.94	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	14.74	E.G. Elev (ft)	14.94	14.67
Q Total (cfs)	670.00	W.S. Elev (ft)	14.74	14.48
Q Bridge (cfs)	522.81	Crit W.S. (ft)	14.82	9.67
Q Weir (cfs)	147.19	Max Chl Dpth (ft)	7.78	9.15
Weir Sta Lft (ft)	-9.68	Vel Total (ft/s)	5.34	3.89
Weir Sta Rgt (ft)	108.92	Flow Area (sq ft)	125.36	172.04
Weir Submerg	0.00	Froude # Chl	0.43	0.36
Weir Max Depth (ft)	0.84	Specif Force (cu ft)	458.20	430.20
Min El Weir Flow (ft)	14.11	Hydr Depth (ft)	1.22	1.53
Min El Prs (ft)	10.90	W.P. Total (ft)	141.27	152.76
Delta EG (ft)	1.60	Conv. Total (cfs)		
Delta WS (ft)	6.40	Top Width (ft)	102.99	112.33
BR Open Area (sq ft)	54.13	Frctn Loss (ft)		
BR Open Vel (ft/s)	9.66	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	-51.00	-12.20

Errors Warnings and Notes

Note:	Momentum answer is not valid if the water surface is above the low chord or if there is weir flow. The momentum
	answer has been disregarded.
Note:	The downstream water surface is below the minimum elevation for pressure flow. The sluice gate equations were used
	for pressure flow.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the upstream end, the water surface and energy have been projected from
	the upstream cross section. The selected bridge modeling method does not compute answers inside the bridge.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the downstream end, the water surface and energy are based on critical
	depth over the weir.

Plan: PC 10'Chnl 20'Cu Mill Brook Mill Brook RS: 100063 Profile: 100 low

E.G. US. (ft)	14.44	Element	Inside BR US	Inside BR DS
W.S. US. (ft)	14.15	E.G. Elev (ft)	14.44	14.30
Q Total (cfs)	670.00	W.S. Elev (ft)	14.15	14.15
Q Bridge (cfs)	639.20	Crit W.S. (ft)	10.69	9.15
Q Weir (cfs)	30.80	Max Chl Dpth (ft)	7.20	8.82
Weir Sta Lft (ft)	-7.18	Vel Total (ft/s)	7.33	4.84
Weir Sta Rgt (ft)	74.73	Flow Area (sq ft)	91.39	138.44
Weir Submerg	0.00	Froude # Chl	0.62	0.39
Weir Max Depth (ft)	0.34	Specif Force (cu ft)	542.62	716.58
Min El Weir Flow (ft)	14.11	Hydr Depth (ft)	1.67	1.24
Min El Prs (ft)	10.90	W.P. Total (ft)	101.07	161.42
Delta EG (ft)	3.58	Conv. Total (cfs)		
Delta WS (ft)	5.81	Top Width (ft)	54.78	111.79
BR Open Area (sq ft)	69.48	Frctn Loss (ft)		
BR Open Vel (ft/s)	9.20	C & E Loss (ft)		
Coef of Q		Shear Total (lb/sq ft)		
Br Sel Method	Press/Weir	Power Total (lb/ft s)	-51.00	-12.20

Errors Warnings and Notes

Warning:	For the final momentum answer at the bridge, the upstream energy was computed lower than the downstream energy.
	This is not physically possible, the momentum answer has been disregarded.
Note:	Momentum answer is not valid if the water surface is above the low chord or if there is weir flow. The momentum
	answer has been disregarded.
Note:	The downstream water surface is below the minimum elevation for pressure flow. The sluice gate equations were used
	for pressure flow.
Note:	Multiple critical depths were found at this location. The critical depth with the lowest, valid, water surface was used.
Note:	For the cross section inside the bridge at the upstream end, the water surface and energy have been projected from
	the upstream cross section. The selected bridge modeling method does not compute answers inside the bridge.
Note:	For the cross section inside the bridge at the downstream end, the energy is based on critical depth over the weir. The
	water surface has been projected.

APPENDIX F

MILL POND DAM DOCUMENTATION

GEI Consultants

Geotechnical
Environmental and
Water Resources
Engineering

Final Design Report and Chapter 253 Permit Application Mill Pond Dam

Rockport, Massachusetts

Volume I of III

Submitted to:
Commonwealth of Massachusetts
Department of Conservation and Recreation
Office of Dam Safety
180 Beaman Street
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May 2010
Project 07104-1

COMMONWEALTH OF MASSACHUSETTS
R. LEE WOOTEN
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Principal

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Appendix G – Technical Memo 4 – Structural Analyses
Appendix H – Low-Level Outlet Waiver Request Letter
Appendix I – Preliminary Construction Schedule

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Executive Summary

This report describes the basis for the design of modifications planned for Mill Pond Dam. This final design report forms part of the submittal to the Department of Conservation and Recreation (DCR) Office of Dam Safety (ODS) as required by the Dam Safety Regulations (302 CMR 10.09) for a Chapter 253 Permit to repair the Mill Pond Dam. This final design report is submitted with (1) a series of photos from 2001 and 2006 flood events (Appendix A), (2) selected site photographs (Appendix B), (3) a completed Chapter 253 Permit Application (Appendix C), (4) a series of technical memorandums (Appendices D through G), (5) a low-level outlet waiver request letter (Appendix H), (6) a preliminary construction schedule (Appendix I), (7) the design drawings (Volume II), and (8) technical specifications for the proposed modifications (Volume III).

Mill Pond Dam is owned and operated by the Town of Rockport, Massachusetts for historical, aesthetic, and recreational purposes. The dam is approximately 110 feet long and about 13.7 feet high at the maximum section. Under the normal pond conditions, the elevation of the water surface is approximately 27.3 feet[1] with an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Under the design flood conditions (the 100-year flood, discussed below) the surface area and volume are estimated to be approximately 3.4 acres and 10 acre-feet, respectively.

The repairs and modifications to the dam are intended (1) to increase the stability of the dam, (2) to prevent the dam from being breached due to internal erosion or piping, (3) to provide overtopping protection and decrease the risk of failure due to erosion or scour during overtopping, (4) to contain flood flows and direct them to the overflow section, and (5) to bring the dam into compliance with ODS Standards. Spillway capacity will not be increased, and therefore, the dam will be designed to safely overtop during the design storm. Partial drawdown of the pond to about El. 22.0 (5.3 feet below normal pool) will be required to complete the majority of this work.

The proposed modifications include:

- Demolishing the existing stone masonry parapet walls.
- Removing and replacing the upstream and downstream granite masonry walls and earthen core.
- Constructing a new reinforced concrete wall near the existing downstream face and a new stone masonry wall along the alignment of the existing upstream stone masonry wall.

[1] Elevations are referenced to NGVD, 1929.

- Reinstalling the existing granite masonry on the downstream face of the new retaining wall to preserve the aesthetic and historic appearance of the dam.
- Installing a filter sand drainage layer under the wall footing, and a granular filter at the toe of the dam.
- Removing and replacing the existing granite slab culvert spillways.
- Constructing a masonry and steel handrail system along the downstream side of the crest and a granite bench system on the upstream side of the crest.
- Constructing a parapet wall along the left abutment to direct flows to the overflow section of the dam and reduce damage to private property and historic structures.
- Regrading and armoring portions of the right abutment to reduce erosion during overtopping.
- Installing bituminous concrete pavement or concrete pavers along the dam crest to provide erosion protection during overtopping.

Currently, no low-level outlet exists at Mill Pond Dam. A former mill sluiceway does exist near the left abutment; however, its current intake is about 1.5 feet above the normal pool elevation. We also understand that the sluiceway is inoperable. In January 2010, we submitted a letter to ODS requesting a waiver from the low-level outlet requirements in 302 CMR 10.14 (7), and have included a copy of this letter in Appendix H.

The existing spillway currently operates as an inlet-controlled culvert spillway. The existing spillway consists of two rectangular granite culverts, which are each approximately 2.5 feet wide and 2.5 feet high, located on the right side of the dam. The invert elevation will remain at El. 25.6 and the culverts will be reconnected with a 2% slope towards the downstream face to allow drainage. The capacity of the culverts will not be changed.

Due to historical constraints, the existing vertical granite slab on the right or north end of the downstream side of the former mill sluiceway cannot be removed during construction. However, it is our intent to support the vertical granite slab during construction and extend the toe of the wall footing to the vertical granite slab of the former sluiceway. A portion of the granite block sluiceway walls will be demolished to allow the construction of the new cantilevered retaining wall, and the void between the wall stem and the existing granite facing will be backfilled with flowable fill.

A number of large trees currently exist on or near the dam, and these trees will be removed prior to the construction of the dam safety modifications. In accordance with safe dam practices, an approximately 20-foot-wide buffer will be maintained between the downstream toe of the dam and any woody vegetation.

An eel ladder will be provided on the downstream side of the dam that will allow eels to pass upstream through the spillway culverts.

1. Introduction

1.1 Purpose

This report describes the basis for the design of modifications that are planned for Mill Pond Dam. This final design report forms part of the submittal to the Department of Conservation and Recreation (DCR) Office of Dam Safety (ODS) as required by the Dam Safety Regulations (302 CMR 10.09) for a Chapter 253 Permit to repair Mill Pond Dam. This final design report is submitted with: (1) Description of 2001 and 2006 Flood Events (Appendix A); (2) Selected Site Photographs (Appendix B); (3) Completed Chapter 253 Permit Application (Appendix C); (4) Series of Technical Memorandums (Appendices D through G); (5) Copy of our Low-Level Outlet (LLO) Waiver Request Letter (Appendix H); (6) Preliminary Construction Schedule (Appendix I); (7) 100% Design Drawings (Volume II); (8) 100% Technical Specifications for the Proposed Modifications (Volume III).

1.2 Existing Conditions and Dam Safety Deficiencies

1.2.1 Location

Mill Pond Dam is located in the Town of Rockport, Essex County, Massachusetts at latitude 42° 39’ 30” North, longitude 70° 37’ 24” West. The dam is located near the center of Rockport, about 30 miles northeast of Boston, Massachusetts. The pond and dam are located on Mill Brook, which flows in a northeasterly direction to the Atlantic Ocean (Sandy Bay) about 550 feet downstream of the dam. The dam and reservoir location is shown on the Site Location Map, provided as Fig. 1. An aerial view of Mill Pond Dam is presented in Fig. 2.

The dam can be accessed from Route 128, easterly to Gloucester, then to Rockport via Route 127 (Upper Main Street) to Route 127A (Beach Street) and in a northerly direction towards King Street. Prior to reaching King Street, the dam can be accessed from Beach Street by foot through the town park.

1.2.2 History

According to the Federal Emergency Management Agency (FEMA) Preliminary Engineering Report (FEMA, 2006, “Preliminary Engineering Report, Dam Site Visit Report, Mill Pond Dam, Rockport, MA,” July 12), the Mill Pond Dam was built in 1701 as a mill pond and was used until 1932 when the mill was destroyed by a fire. A resident near the dam, Mr. Martin Ross, stated to GEI that the pond has steadily become filled with sediment and was dredged in 1886 after a flood. Since 1980, Mr. Ross has observed the depth of the pond decrease due to sedimentation at a rate of 1.1 to 1.7 inches per year. He also stated that the dam was

overtopped in March 2001. Photographs taken by Mr. Ross during the 2001 flood are provided in Appendix A (Photographs A-1 to A-3). Based on precipitation data recorded in Beverly, Massachusetts, (approximately 15 miles southwest of Rockport), 4.3 inches of rain fell on March 22, 2001.

The dam was also overtopped during two days of heavy precipitation on May 13 and 14, 2006. The precipitation recorded on these two days in Beverly was 4.32 and 4.95 inches, respectively. As a result of the May 2006 flooding, large stone blocks on the downstream face became dislodged and a 45-foot-long portion of the downstream wall collapsed and toppled downstream of the dam. Since that event, the Town of Rockport has made temporary repairs to the dam by placing stone in the breached area and blocking vehicular traffic from crossing the dam crest. Photographs of the flooding and dam breach are provided in Appendix A as part of a memorandum prepared by Weston & Sampson Engineers, Inc. (Weston & Sampson) of Peabody, Massachusetts. The memorandum summarizes their Emergency Response Inspection on May 15, 2006.

1.2.3 Survey and Site Visits

GEI engineers visited the site on December 27, 2006, May 10, 2007, and August 4, 2009 to observe the existing site conditions, watershed, outlets, and area downstream of the dam. Selected photographs taken during GEI's site visits are provided in Appendix B.

BSC Group (BSC) of Boston, Massachusetts performed a topographic survey of the dam and downstream area in April and May of 2007 and revised the survey to include property line information in June and July of 2009. The survey is based on the Town of Rockport, Benchmark T26 (El. 62.89 feet) located in a ledge outcrop on the corner of Main and Hale Streets. The datum of the benchmark and elevations used in this report and our analyses is the National Geodedic Vertical Datum of 1929 (NGVD 29) datum. The existing conditions survey plan prepared by BSC is provided on Drawing C-03 in Volume II.

BSC's wetland scientists inspected the area around the dam to delineate wetland areas and to collect data as part of the permitting assessment. The wetland delineations marked in the field were surveyed by BSC surveyors and are shown on the site plan on Drawing C-03 (Volume II).

1.2.4 Dam Description

Mill Pond Dam is owned and operated by the Town of Rockport, Massachusetts for historical, aesthetic, and recreational purposes. The dam is approximately 110 feet long and about 13.7 feet high at the maximum section.

According to the FEMA Preliminary Engineering Report (FEMA, 2006), the dam was constructed with a clay core and hand-laid unmortared stone walls on the upstream and

downstream face. Based on our observations of the subsurface conditions encountered in recent borings, the core of the dam appears to be a silty sand. The crest of the dam consists of stone aggregate surface with field stone parapet walls that are each about 95 feet long and extend along most of the upstream and downstream face of the dam. Because the parapets do not extend across the entire crest of the dam they act more as guard rails than parapet walls. It is speculated that the parapets were added to the dam in the 1930's and may have contributed to the dam failure by causing the pond level to reach higher levels during flood events. Under the normal pond conditions, the elevation of the water surface is approximately 27.3 feet with an estimated surface area of 1.3 acres and an estimated storage volume of about 3 acre-feet. Under the design flood conditions (the 100-year flood, discussed below) the surface area and volume are estimated to be approximately 3.4 acres and 10 acre-feet.

The crest of the dam varies from El. 28.5 to 29.5 feet and is about 15 feet wide at the narrowest section. The upstream and downstream faces of the dam are nearly vertical. Only about 4 feet of the upstream face is exposed above the sediment in the pond. The tops of the parapet walls are about 2.0 feet above the dam crest at El. 31.0 feet but do not extend along the full length of the dam. The highest section of the dam is located near the former mill sluiceway on the left side of the dam (looking downstream). The ground surface at the base of the downstream masonry wall near the former sluiceway is at El. 15.8 feet. Based on a maximum crest of El. 29.5 feet, the maximum height of the dam is about El. 13.7 feet.

The dam appears to have been constructed without a filter zone to control seepage and internal erosion or piping through the downstream masonry wall. We observed significant seepage flowing through the downstream face of the dam, just below the spillway outlet during our December 2006 site visit (see flow on left of spillway in Photograph B-9, Appendix B).

<u>Former Mill Sluiceway</u> – The intake to the sluiceway consists of a rectangular granite culvert on the upstream face of the dam (Photograph B-7). Vegetation and sedimentation along the upstream face of the dam have partially blocked the intake to this structure, and we understand that the sluiceway is inoperable. The intake opening is about 3 feet wide and the height varies from 0.7 feet on the left side to 1.5 feet on the right side. The current invert of the sluiceway intake is approximately El. 28.9 feet; however, the historic invert elevation is unknown. The inlet was constructed with several granite slabs that served as the top of the sluiceway intake structure and the crest of the dam. The condition and dimensions of the sluiceway inside the dam are unknown.

The blocked inlet apparently leads to a chamber within the dam with an 8-inch-diameter cast iron pipe and valve that exits on the downstream face of the dam (Photograph B-6). There is a remnant of an old control device on the crest of the dam above the outlet pipe that likely operated a vertical slide gate. The Town of Rockport reports that the controls and the valve

are not operational. Based on photos taken during the 2006 breach, we believe that the sluiceway walls are constructed from granite blocks that extend through the dam. The bottom elevation of the sluiceway is unknown, and the Town of Rockport has informed us that a majority of the sluiceway was backfilled with crushed stone or gravel as part of the temporary repairs following the 2006 breach.

Spillway – The Mill Pond Dam spillway consists of two rectangular granite culverts which are each approximately 2.5 feet wide and 2.5 feet high located on the right side of the dam (Photographs B-8 and B-9). The upstream invert of the culverts is at about El. 25.6 feet with a slightly higher outlet elevation on the downstream side at about El. 26.0 feet. Based on an assumed normal pond elevation of 27.3 feet and a normal base flow of 1.3 cubic feet per second (cfs), we estimated that stoplogs about 1.6 feet high were historically installed in the spillway culverts to maintain the normal pond elevation.

1.2.5 Potential Downstream Hazards

Flow from the dam is contained within Mill Brook. The town maintains a park (Mill Brook Meadow) immediately downstream of the dam. The park includes a gravel walkway and a grassy area that contains a small playground. There are several residential structures located on the left side (looking downstream) of the brook. The sill elevations (first or ground floor) of the residential properties range from about El. 19 to El. 15 feet. The house that almost abuts the dam on the left side of the brook has a walkout basement with an estimated elevation of about 17 feet. Commercial properties (bed and breakfast/restaurant establishments) are located on the left and right side of Mill Brook near Beach Street, about 550 feet downstream of the dam. The first floor elevations of these structures range from approximately El. 15 to 19 feet. Mill Brook passes through a granite block culvert under Beach Street, which has a paved surface elevation of about 15 feet.

1.3 Design Criteria

Mill Pond Dam is classified by 302 CMR 10.06 as a Small, Significant Hazard (Class II) dam. Pursuant to 302 CMR 10.14, a dam of this classification is required to pass or store the potential spillway design flood (SDF), equal to or greater than the 100-year storm per 302 CMR 10.14 (6) without failure of the dam occurring. The dam is currently not designed to safely pass this flow. Due to historical constraints, the historical granite culvert spillways will remain. Therefore, the spillway capacity will not be significantly increased and the dam will be overtopped by the SDF. The proposed modifications will allow Mill Pond Dam to safely pass the SDF by overtopping and provide protection against the risk of failure due to internal erosion or piping. The modifications will also provide a stabilized crest, a splash apron, and riprap armoring at the toe of the dam to reduce erosion and scour during overtopping. A summary of the parameters used during design are provided in Appendix D, and detailed Hydrologic and Hydraulic (H&H) Calculations are provided in Appendix E.

The proposed modifications do not change the pond or the watershed from their current condition. Removal of trees will be required downstream and along the abutments for the proposed modifications.

1.4 Proposed Modifications

1.4.1 Dam Repair and Reconstruction Activities

The repairs and modifications to the dam are intended (1) to increase the stability of the dam, (2) to protect the dam from internal erosion or piping, (3) to provide overtopping protection and decrease the risk of failure due to erosion or scour during overtopping, (4) to contain flood flows and direct them to the overflow section, and (5) to bring the dam into compliance with ODS Standards. Design Drawings for the repairs and modifications are included in Volume II for reference. Technical Specifications are included in Volume III.

The existing granite masonry walls and earthen core will be removed and replaced with a reinforced concrete cantilevered retaining wall. The existing stone masonry parapet walls will also be removed. The new retaining wall has been designed to resist active soil pressures and hydrostatic pressures. A filter sand drainage layer will be installed directly below the wall footing, and a granular filter will be constructed at the downstream toe of the dam. The filters will help to safely collect and discharge under-seepage flows and relieve uplift pressures. The existing stone masonry on the downstream face will be reinstalled over the concrete retaining wall to preserve the aesthetic and historic appearance of the dam, and the upstream stone masonry wall will be reconstructed. The parapet walls along the crest will be replaced with a granite bench on the upstream face of the dam and a handrail system on the downstream face of the dam. Replacing the parapet walls with a handrail will allow water and debris to safely pass over the reconstructed part of the dam during significant storm events at lower peak water surface elevation and allow a more uniform flow direction and velocity over the dam to reduce erosion and scour.

The top of the proposed crest will be at El. 29.0, about the same elevation as the existing crest. The crest surface will be stabilized with bituminous concrete or cobblestone pavers to protect against erosion during overtopping. Because the dam is being designed to be overtopped, the crest was not raised, and therefore does not provide any freeboard during the SDF. Due to historic and site constraints, the proposed repairs do not include significant modifications to the existing granite culvert spillways, and therefore the hydraulic efficiency of the spillways is unchanged. Our calculations indicate that during the SDF, the dam will be overtopped by up to about 2 feet of water for approximately 13 hours.

Improvements to the upstream side of the dam will only include the removal and replacement of the existing stone masonry walls with a new stone masonry wall and granite bench system. The new upstream stone masonry wall will absorb wave energy and prevent erosion of the backfill behind the new cantilevered retaining wall.

A parapet wall will be constructed on the left (west) abutment to prevent the design flood from overtopping the abutment and impacting a historic foundation and an abutting residence immediately downstream of the left abutment. The right abutment and the stone walkway will function as an overflow spillway. The gradual slope of the stone walkway and the right abutment might sustain surficial damage from the design storm but should not erode enough to jeopardize the dam.

Partial drawdown of the pond to about El. 22.0 (5.3 feet below normal pool) will be required to complete the majority of this work. The contractor will be required to design and implement a temporary cofferdam and bypass system for construction of the walls in the dry and to protect the work against flood flows that may occur during construction. Temporary cofferdam and bypass design by a Professional Engineer registered in the Commonwealth of Massachusetts will be required and will be subject to review by GEI and the Town of Rockport.

1.4.3 Low-Level Outlet

Currently, no low-level outlet exists at Mill Pond Dam. A former mill sluiceway does exist near the left abutment; however, its current intake is about 1.5 feet above the normal pool elevation. Refer to Section 1.2.4 above for more information about the former mill sluiceway. In January 2010, we submitted a letter to ODS requesting a waiver from the requirements in 302 CMR 10.14 (7), and have included a copy of this letter in Appendix H. We understand that ODS will evaluate this request as part of the Ch. 253 permit application review.

Because drawdown will not be required to reduce the loading on the dam and because of the small size of the Mill Pond impoundment (3 acre-feet), it is our opinion that the pond can be easily be drawn down with pumps owned or rented by the Rockport Department of Public Works (DPW) and that a low-level outlet would not be required for drawdown. Once construction is complete, drawdown will only be required for routine maintenance.

Given the relatively small size of the watershed, pond, and the normal inflow, our calculations indicate that the pond can be lowered using a mobile end suction centrifugal pump. The normal inflow is approximately 580 gallons per minute (gpm) (1.3 cfs) and the total volume of the pond at normal pool is approximately 3 acre-feet (1 million gallons). Therefore, assuming normal inflows, a pump operating at 800 gpm (e.g. Rain for Rent DV-100) could drain the pond in about 72 hours (3 days). A detailed Drawdown Plan will be included in the Operation and Maintenance Manual. A larger pump could lower the pond faster. However, a large increase in outflow may adversely affect the stream and other resource areas immediately downstream of the dam.

It is our opinion that a mechanical drawdown allows Mill Pond to be lowered in a safe and cost effective manner. Therefore, we request a waiver from the requirement in 302 CMR

10.14 (7) for a low-level outlet at Mill Pond Dam in Rockport, Massachusetts, and request that mechanical means be permitted to draw down Mill Pond for routine maintenance.

1.4.4 Spillway Reconstruction

The existing spillway currently operates as an inlet-controlled culvert spillway. The existing spillway consists of two rectangular granite culverts which are each approximately 2.5 feet wide and 2.5 feet high located on the right side of the dam.

In accordance with 302 CMR 10.14(6), the modifications to the Mill Pond Dam and spillway have been designed to allow the SDF to be stored and pass over the dam without failure occurring. The culvert spillway will flow full during the SDF, and the dam crest and right embankment will act as overflow sections to pass the remainder of the discharge from the SDF. We calculated that the existing spillway will not safely pass the SDF without overtopping the dam, and that the SDF would overtop the overflow section of the dam by up to about 2 feet for about 13 hours. We have included a stabilized crest, a reinforced concrete retaining wall, a splash apron, and riprap armoring in the design to minimize the risk of failure during overtopping.

Significant modifications to the spillway will not be performed as part of the remedial design. The spillway will be removed and reset in a reinforced concrete cradle with a seepage cutoff key near the upstream face of the dam. The invert elevation will remain at El. 26.5 and the culverts will be installed with a 2% slope towards the downstream face to allow drainage. The capacity of the culverts will not be changed.

1.4.5 Former Mill Sluiceway

Due to historical constraints, the existing vertical granite slab and abutting granite block walls on the north end of the downstream side of the former mill sluiceway will not be removed during construction. However, the Contractor will be required to support this section of wall during construction and extend the toe of the wall footing to the upstream side of the vertical granite slab and abutting granite block walls. A portion of the sluiceway walls internal to the dam will be demolished to allow the construction of the new cantilevered retaining wall, and the void between the wall stem and the existing granite walls will be backfilled with flowable fill. At the crest a parapet wall will extend to El. 33.0 feet to direct flood flows to the overflow section, minimize damage to the historic features (grist mill foundation and former mill sluiceway façade) and to provide fall protection.

1.4.6 Maintenance Activities

A number of large trees currently exist on or near the dam, and these trees will be removed prior to the construction of the dam safety modifications. In accordance with safe dam practices, an approximately 20-foot-wide buffer will be maintained between the downstream

toe of the dam and any woody vegetation. This area will act as a non-woody vegetated buffer and will be routinely mowed and trimmed to control the growth of woody vegetation that could damage the integrity of the dam.

2. Design Data and Analyses

2.1 Subsurface Investigations

In May 2009, we advanced two borings at Mill Pond Dam. B-1 was advanced to the west (left) of the former Mill Sluiceway and B-2 was advanced to the west (left) of the existing granite block culverts. Based on our review of photographs and discussions with Town of Rockport personnel, it is our opinion that the borings were advanced outside of the portions of the dam that were repaired following the 2006 breach. The boring logs and a boring location plan are provided in Appendix D.

2.2 Soil Conditions at the Dam

Two generalized soil layers were encountered in the borings:

- Embankment Fill
- Foundation Soil

The soil layers encountered in the borings are described below, in order of increasing depth. Bedrock was not encountered in either of the borings.

Embankment Fill – The embankment fill sampled is generally described as sand and gravel with a wide range of non-plastic fines content. Embankment fill was observed in both borings and had an average thickness of about 13 feet. Fines content in the samples ranged from less than 5% to approximately 40%. Based on Standard Penetration Test (SPT) N-values, the embankment fill can be described as loose to medium dense.

Foundation Soil – The foundation soils sampled generally consisted of non plastic to slightly plastic fines and sands with gravel and occasional cobbles and granite boulders. Based on regional geology, the foundation soils likely consist of glacial till, and by correlating SPT N-values, the foundation soils can be described as medium dense to very dense. Borings B-1 and B-2 were advanced from the dam crest (El. 29.0 ± feet) and were terminated in the foundation soil layer at a depth of 29 feet and 31 feet, respectively.

2.3 Groundwater Conditions

Water levels were not measured during our subsurface investigations, and as a result, water levels measurements are not provided in this report. During normal operating conditions, seepage has been observed in the downstream face. Therefore, we have assumed that the phreatic surface is approximately at the water surface elevation on the upstream side and slightly above tailwater elevation on the downstream face. Appendix F provides a more

detailed description of the phreatic surface and seepage forces that we modeled through the dam.

2.4 Seismic Design Information

From information provided in the Geologic Hazard Maps, we estimated that Mill Pond Dam is in Geologic Hazard Map Zone 3. Per 302 CMR 10.14 (i), we used a Peak Ground Acceleration (PGA) of 0.15g (gravity) in our analyses. Additional information is provided in Appendix G.

2.5 Dam Stability

As part of our design, we performed stability analyses on the cantilevered retaining wall and associated soil mass for a number of different loading conditions under normal and maximum pool. The factors of safety for the cantilever retaining wall calculated during our analyses were greater than or equal to the recommended minimums required by 302 CMR 10.14. Additional information is provided in Appendix G.

2.6 Safety During Overtopping

Hydraulic analyses were performed to determine the maximum reservoir level and overtopping flows associated with the SDF (the 100-year flood). In the proposed configuration, the pond would rise during the SDF to an elevation of 31.0, and overtop the dam by 2.0 feet for approximately 13 hours. The dam will be designed to overtop and should not experience a failure during the overtopping. The H&H analyses for the dam under the existing conditions and with the proposed modifications under the SDF are presented in Appendix E. The removal of the existing parapet walls and installation of a handrail system on the downstream side will provide open area that will allow flow and help reduce the duration and depth of overtopping. The construction of the proposed parapet walls on the left side of the dam and along the left abutment will help prevent flood flows from damaging historic structures and private property.

2.7 Seepage Controls

The proposed modifications to the dam address the current seepage issues that we identified during design. The reinforced cantilever retaining wall will be relatively water tight and will not be prone to failure caused by internal erosion or piping. The installation of the filter sand layer below the wall footing and the granular filter along the downstream toe will collect seepage flows and reduce the movement of fines. The filters will intercept under-seepage and allow seepage flows to be safely channeled below the dam to the downstream channel.

Because of the relatively short seepage path between the parapet wall and the existing historical wall near the left abutment, a layer of low permeability fill will be installed to

decrease gradients under the parapet wall footing and help the historic wall remain stable during the SDF.

Exit gradients are not of concern as the piezometric surface does not extend through the cantilever retaining wall and gradients are reduced by the more permeable filter sand layer under the wall footing. These gradients appear to have reasonable values as water moves from the sand filter under the dam and into granular filter. A detailed description of the seepage analyses is provided in Appendix F.

2.8 Spillway Design

The existing twin granite block culvert spillways will be removed and reconstructed in a concrete cradle at their existing locations and elevations. The re-installed spillway will discharge flow to a riprapped stilling pool in the downstream channel. Riprap armoring and a concrete splash apron will be installed below the water level in the pool. The flow will then travel down the existing channel, under Beach Street, and enter the Atlantic Ocean (Sandy Bay) about 550 feet downstream of the dam.

The calculated spillway capacity with stoplogs removed is about 130 cfs, and the estimated peak discharge during the SDF is 670 cfs. Therefore, the spillway does not have adequate capacity to pass the SDF and the dam will act as an overflow structure to safely pass the SDF. Detailed H&H analyses for the spillway are provided in Appendix E and the structural analyses are provided in Appendix G.

Stoplog guides will be installed on the upstream side of the culvert spillway and an approach channel will extend upstream of the spillway to channel flow to the spillway. The stoplogs will be used to vary the pond level, allow eel migration, and also allow the Town of Rockport DPW to lower the normal pool by about 2 feet for periodic maintenance. The stoplogs will be removed during any large storm event.

2.9 Crest Protection

The proposed modifications include installing concrete or cobblestone pavers or bituminous paving along the crest of the dam, the left and right abutments, and the path down to the meadow on the right abutment. The features will help prevent soil erosion during overtopping and should require minimal maintenance after an overtopping event. The new crest and abutment surfaces will be designed to permit vehicular access across the dam crest for maintenance.

2.10 Eel Ladder

The reconstructed dam will provide passage for American Eels during their migration to and from Mill Pond. American Eels are the only catadromous fish in North America. The term

catadromous refers to fish that are born in the ocean, travel to fresh water to mature, and then return to the ocean to spawn. Currently, the eels are thought to crawl through the stone masonry walls and burrow through the soil to reach Mill Pond. However, since the eels will not be able to penetrate the new reinforced concrete retaining wall, the reconstructed dam must provide alternate passage via an eel ladder or ramp.

The eel ladder will consist of a steel trough and an eel substrate attached to the bottom of the trough to allow the eels to move up the ladder. The ladder will be permanently attached to the downstream face of the dam. The ladder will be located on the right side of the dam, near the spillway culverts and will extend through the spillway culvert to provide passage from the downstream channel to Mill Pond. Normal flow through the culvert should provide adequate flows to attract eels to the ladder and allow them to ascend the ladder. When normal fluctuations in flow rate occur, we expect that the eels will wait in the downstream channel until favorable flows return.

(PART B cont.)

HYDROLOGIC, HYDRAULIC AND STRUCTURAL CONSIDERATIONS

Hydrologic, hydraulic and structural design procedures should be used, as established by one of the following: The U.S Army Corps of Engineers, the U.S. Bureau of Reclamation, the U.S. Soil Conservation Service and other procedures universally accepted as sound engineering practice.

1. Contributory drainage area (sq. mi.): **0.75 (Appendix E of the Design Report, Fig. 1)**
   (Attach topographic map with outline of drainage area)

2. Design storm duration: **100-year storm**

   Rainfall Intensity (inches/hour): Maximum Intensity = **0.275 inches/hour**

3. Runoff (%) **74** Inches: **6.6**

4. Peak Outflow (cfs): **670**

5. Previous known flood of record (month/year): **Flood of record unknown although the dam has overtopped in March 2001 and May 2006. As a result of the May 2006 flooding, large stone blocks on the downstream face became dislodged and a 45-foot-long portion of the downstream parapet wall collapsed and toppled downstream of the dam. Since that event, the Town of Rockport has made temporary repairs to the dam by placing stone in the breached area and blocking vehicular traffic from crossing the dam crest.**

6. Design maximum flood level elevation: **31.5 ft**

7. Additional information:

DESIGN CRITERIA

1. Datum used: MSL of 1929 (**X**) NAVD 88 _____ Assumed_____Other_____

2. Type of structure (earth, concrete, etc.): **Concrete Cantilever Retaining Wall with Soil Backfill**

3. Maximum structural height of the dam (feet): **14 +/-**

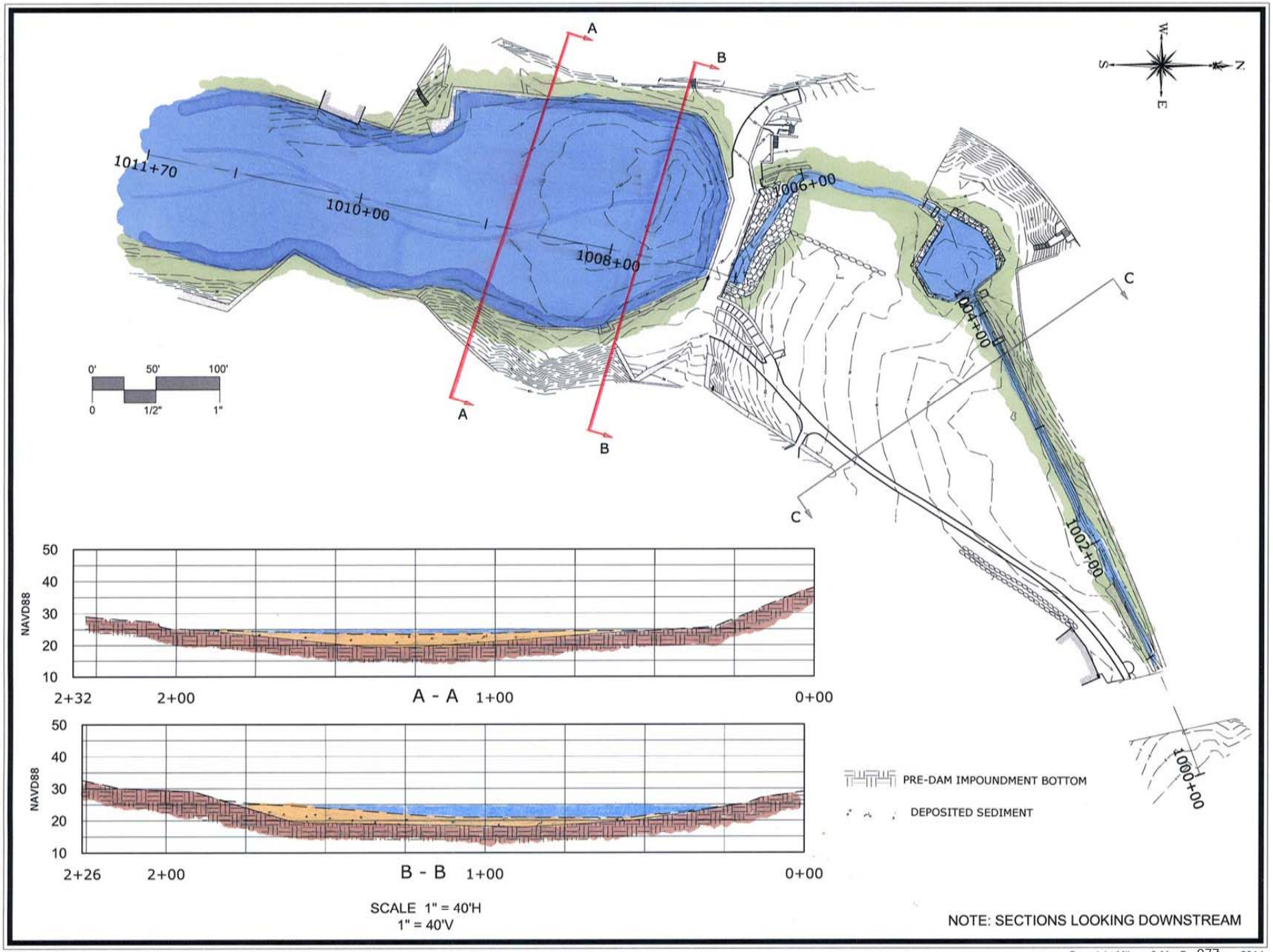
4. Crest length (ft): **107 +/- ; Crest width (ft): 14 +/-**

5. Top elevation of dam: **29.0**

6. Present river or channel elevation at dam (ft): **15.8 to 20.0 +/-**

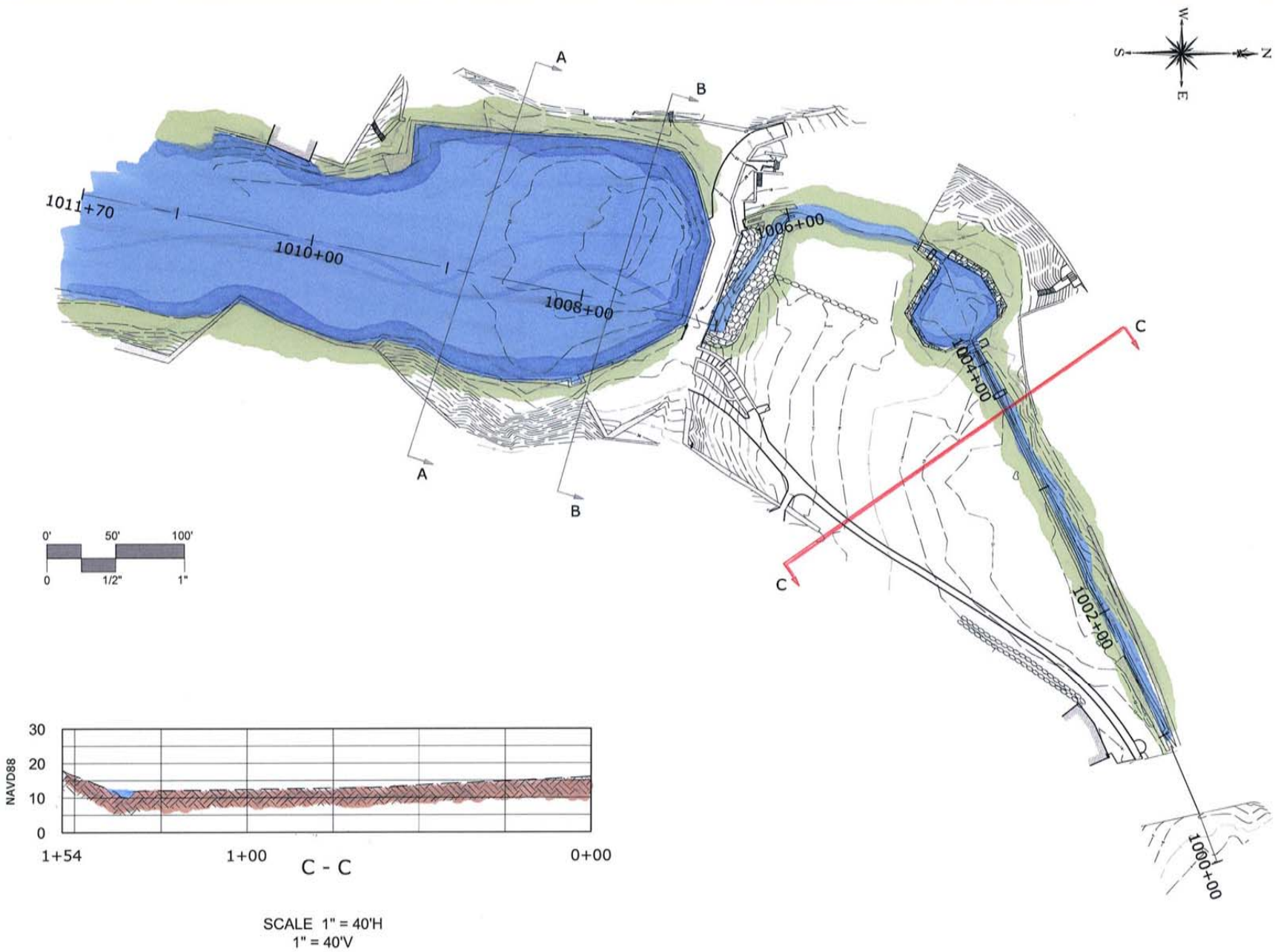
7. Normal pool elevation (ft)**: 27.3**

8. Normal pool surface area (acre): **1.3**



Copyright Milone & MacBroom, Inc - 2014

Drawing: W:\DESIGN\5220-02-DE\CAD\HEC-RAS.DWG Layout Tab:MPOUNDMENT
Plotted by: MKEA On this date: Mon, 2014 May 19 - 1:04pm



Drawing: W:\DESIGN\5220-02-DE\CAD\HEC-RAS.DWG Layout Tab:CHANNEL

Plotted by: MIKEA On this date: Mon, 2014 May 19 - 1:04pm

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9. Normal impoundment (acre-ft): **3**

10. Maximum pool elevation (ft): **100 year storm – 31.0**

11. Maximum pool surface area (acre): **3.4**

12. Maximum impoundment (acre-ft): **10**

13. Freeboard, as measured from the maximum design pool elevation to the crest of dam (ft): **- 2.0**

14. Nature of slope protection: **N/A**

15. Primary Spillway information:

Spillway type: **Twin Rectangular Granite Block Culverts**

Top elevation: **25.6 ft. (Invert)**

Dimensions (ft): **Two culverts each: 2.5 ft High by 2.5 Wide.**

Capacity (cfs): **120 cfs with WSEL at El. 31.0**

Percentage of design flood: **18%**

16. Emergency Spillway information:

Spillway type: **Dam overtopping**

Top elevation: **29.0**

Dimensions (ft): **85 ft long**

Capacity (cfs): **550 CFS (when water level is El. 31.0)**

Percentage of design flood: **82%**

17. Low Level Outlet (s): **None. See Waiver Request Letter provided in Appendix H of the Design Report.**

Type(s):

Invert elevation(s):

Dimension(s) (ft):

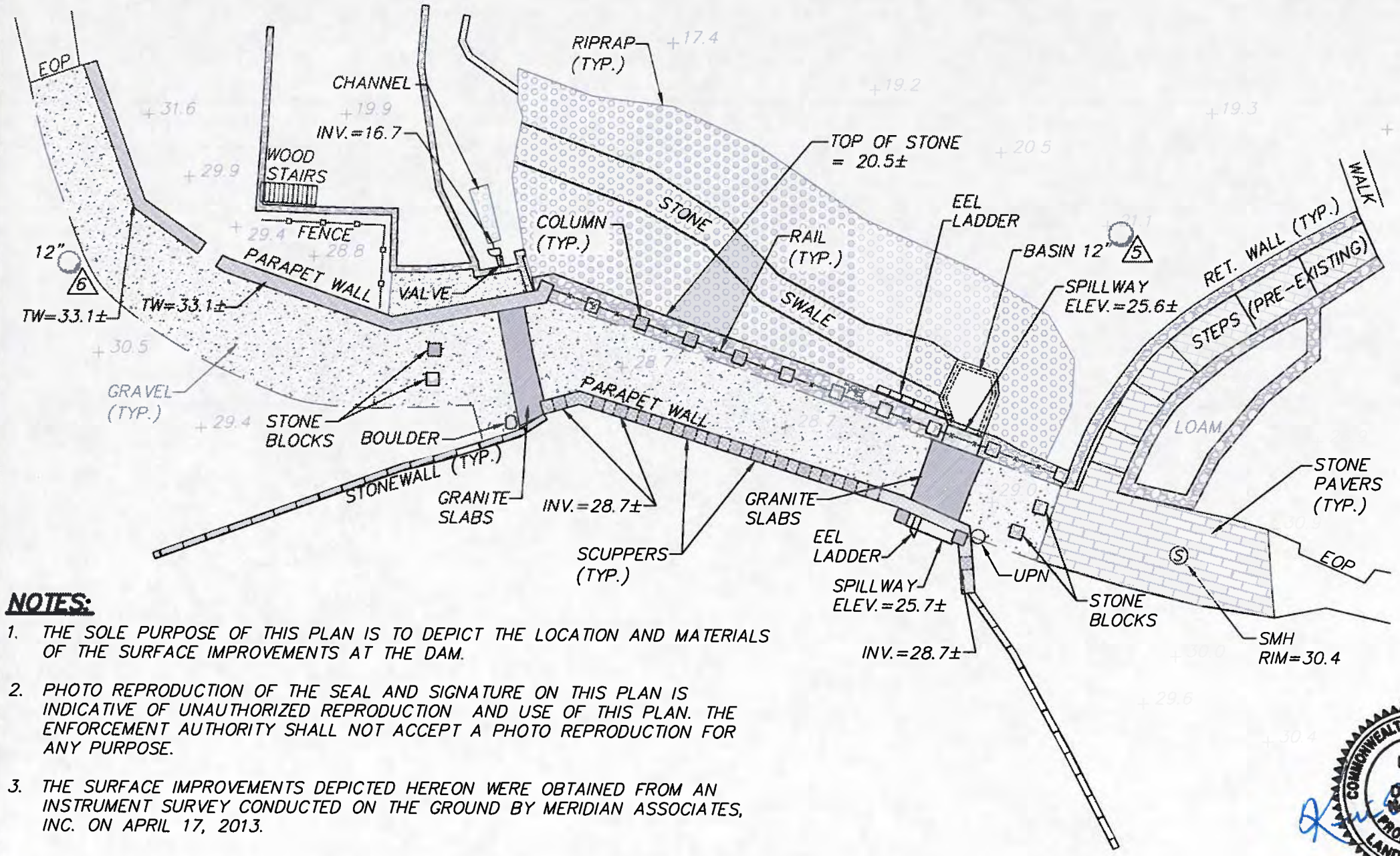
Capacity (cfs):

Percentage of design flood:

SUBSURFACE INVESTIGATION

Boring logs, analysis and recommendations to accompany this application.

Boring logs and design parameters are provided in Appendix D of the Design Report.



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MAI

LEGEND:

BK	BOOK
PG	PAGE
TYP.	TYPICAL
ELEV.	ELEVATION
INV.	INVERT
EOP	EDGE OF PAVEMENT
RET. WALL	RETAINING WALL
SMH	SEWER MANHOLE
UPN	UTILITY POLE
+ 21.1	SPOT GRADE
—D—	DRAIN LINE
	FENCE
	RAIL
	STONE PAVERS
	GRAVEL
	RIPRAP
	STONEWALL
	PILLARS
	BOULDER
	WALL

NOTES:

1. THE SOLE PURPOSE OF THIS PLAN IS TO DEPICT THE LOCATION AND MATERIALS OF THE SURFACE IMPROVEMENTS AT THE DAM.
2. PHOTO REPRODUCTION OF THE SEAL AND SIGNATURE ON THIS PLAN IS INDICATIVE OF UNAUTHORIZED REPRODUCTION AND USE OF THIS PLAN. THE ENFORCEMENT AUTHORITY SHALL NOT ACCEPT A PHOTO REPRODUCTION FOR ANY PURPOSE.
3. THE SURFACE IMPROVEMENTS DEPICTED HEREON WERE OBTAINED FROM AN INSTRUMENT SURVEY CONDUCTED ON THE GROUND BY MERIDIAN ASSOCIATES, INC. ON APRIL 17, 2013.
4. THIS PLAN DOES NOT DEPICT ANY UNDERGROUND UTILITIES. THE CONTRACTOR, PRIOR TO COMMENCEMENT OF CONSTRUCTION, SHALL VERIFY THE LOCATION OF ALL UTILITIES AND CONTACT DIG SAFE AT 1-888-344-7233.
5. THIS PLAN DOES NOT SHOW ANY UNRECORDED OR UNWRITTEN EASEMENTS WHICH MAY EXIST. A REASONABLE AND DILIGENT ATTEMPT HAS BEEN MADE TO OBSERVE ANY APPARENT, VISIBLE USES OF THE LAND; HOWEVER, THIS DOES NOT CONSTITUTE A GUARANTEE THAT NO SUCH EASEMENTS EXIST.
6. SEE SHEET 2 FOR CONCRETE FOUNDATION AND WALL AS-BUILT INFORMATION.
7. ELEVATION DATUM IS BASED UPON THE TOWN OF ROCKPORT BENCHMARK T26, ELEV. 62.89, A COPPER PLUG IN THE TOP OF THE LEDGE OUTCROP AT THE CORNER OF MAIN AND HALE STREETS. ELEVATION OF SAID BENCHMARK IS NGVD 1929.
8. HORIZONTAL DATUM FOR THE PROJECT IS THE MASSACHUSETTS STATE PLANE COORDINATE SYSTEM, NAD 1983.

TEMPORARY BENCHMARK CHART:

T.B.M.#	DESCRIPTION	ELEVATION
5	CUT SPIKE SET IN 18" DECIDUOUS TREE	22.51
6	CUT SPIKE SET IN 12" DECIDUOUS TREE	32.54

(SEE NOTE 7)

COMMONWEALTH OF MASSACHUSETTS
KEVIN E. DANAHY
No. 38653
PROFESSIONAL LAND SURVEYOR

7-8-13

MILL POND DAM
RECORD CONDITIONS PLAN

LOCATED IN

ROCKPORT, MASSACHUSETTS

(ESSEX COUNTY)

PREPARED FOR

DEPARTMENT OF PUBLIC WORKS
ROCKPORT, MASSACHUSETTS

SCALE: 1"= 20' DATE: JULY 8, 2013

10 0 10 20 40

MERIDIAN ASSOCIATES

500 CUMMINGS CENTER, SUITE 5950
BEVERLY, MASSACHUSETTS 01915
TELEPHONE: (978) 299-0447

69 MILK STREET, SUITE 302
WESTBOROUGH, MASSACHUSETTS 01581
TELEPHONE: (508) 871-7030

WWW.MERIDIANASSOC.COM

DWG. No. 5471-REC

BK. 599, PG. 10

MEMORANDUM

TO: Millbrook Meadow Advisory Committee

FROM: Jason Williams, L.A.

DATE: March 12, 2014

RE: Potential Grant Opportunities and Project Permitting

Committee Members,

We have reviewed some of the potential improvements that will be associated with the Millbrook Meadow and Pond Restoration project, and for your use have come up with a list of the permits that may be required.

- Notice of Intent/ Order of Conditions- *Local Conservation Commission*
- Secretarial Certificate - *Massachusetts Environmental Policy Act (MEPA Unit)*
- 401 Water Quality Certification – *MassDEP*
- Chapter 91 license – *MassDEP*
- Federal Consistency Concurrence – *Massachusetts Office of Coastal Zone Management*
- Section 404 permit – *U.S. Army Corps of Engineers*

Additionally, to assist with your fundraising efforts we have compiled a list of potential funding sources that you may wish to explore:

- Adams Arts Program: http://www.masscultural council.org/programs/adamsarts.asp (Application would need to stress improvements to local culture and tourism)
- Cultural Facilities Fund: http://www.masscultural council.org/facilities/facilities.htm
- Massachusetts Environmental Trust (MET): http://www.mass.gov/eea/grants-and-tech-assistance/grants-and-loans/mass-enviro-trust/
- National Endowment for the Arts "Our Town" Program: http://arts.gov/news/2013/national-endowment-arts-releases-funding-guidelines-our-town
- MassDEP 604(b) Water Quality Management Planning Grants: http://www.mass.gov/eea/agencies/massdep/water/grants/604b-water-quality-management-planning-grants.html
- The Max and Anna Levinson Foundation: http://www.levinsonfoundation.org/
- The RBC Blue Water Project: http://www.rbc.com/community-sustainability/apply-for-funding/guidelines-and-eligibility/blue-water-project.html

Focus on Park Revitalization

- Division of Conservation Services: http://www.mass.gov/eea/grants-and-tech-assistance/grants-and-loans/dcs/

- PARC – Parkland Acquisitions and Renovations for Communities: http://www.mass.gov/eea/grants-and-tech-assistance/grants-and-loans/eea-grants-guide/land-and-recreation.html

Focus on Coastal Resiliency

- Five Star and Urban Waters Restoration Programs: http://www.nfwf.org/fivestar/Pages/2014rfp.aspx
- Coastal Community Resilience Grant Program: http://www.mass.gov/eea/agencies/czm/program-areas/stormsmart-coasts/grants/

Additional Considerations:

- The Mattina R. Proctor Foundation makes grants for cultural, health, conservation and other purposes, primarily in Massachusetts and Maine. Grants are in the $1,000-$50,000 range (occasionally larger), and a total of about $600,000 is distributed annually.
- The Virginia S. Warner Foundation makes grants to educational and conservation organizations, primarily in Virginia and Massachusetts.
- The Lebensfeld Foundation (no web page) primarily supports nonprofit organizations in the Mid-Atlantic and New England areas. The Foundation's areas of interest include health, education, civic affairs, children and youth, and arts and culture. Types of support include general operating and project support. Typical grant size is in the $2,500 – 10,000 range.

Appendix E

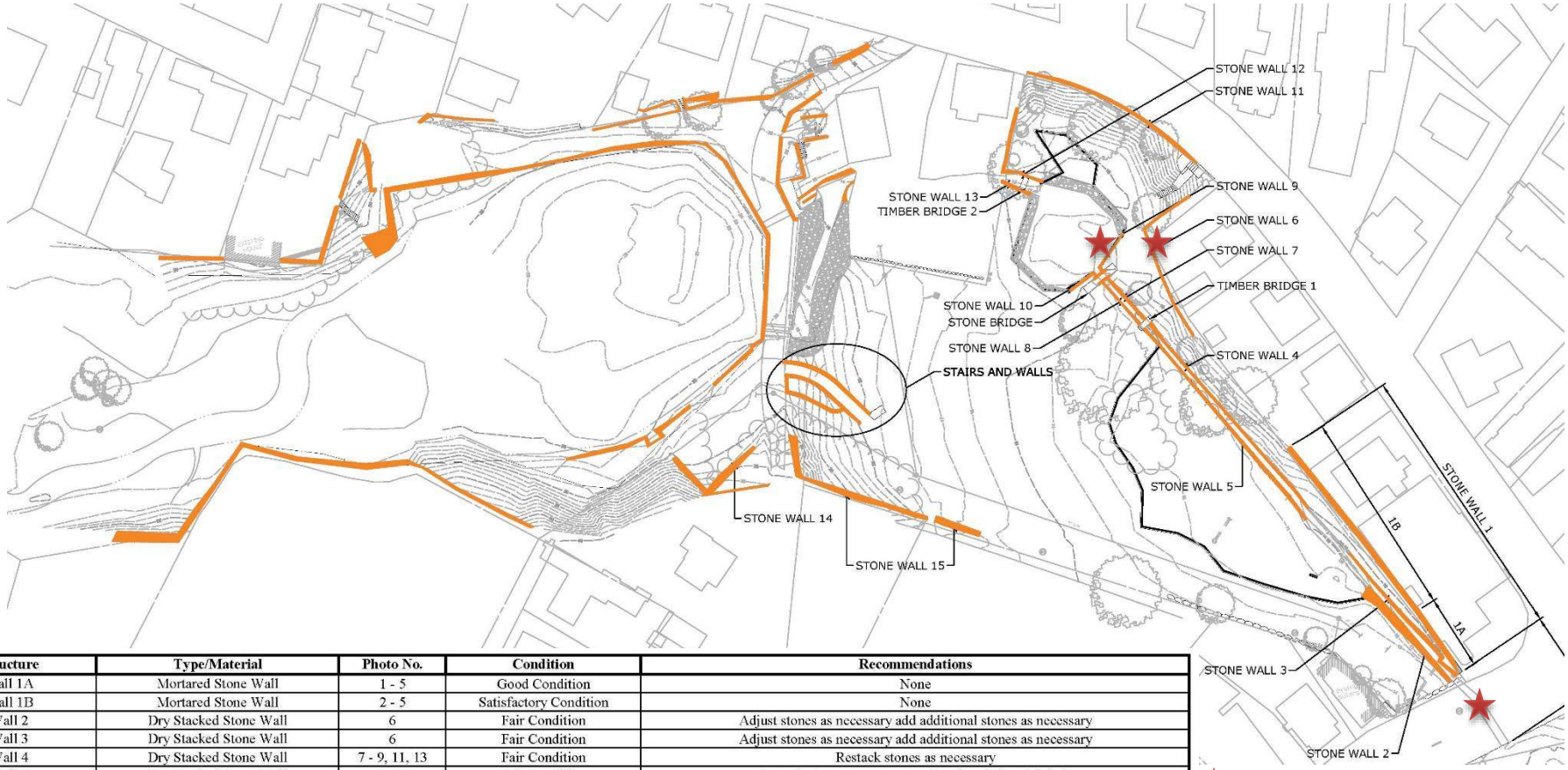
MILONE & MACBROOM

Recommendations & Concepts

Millbrook Meadow and Mill Pond Restoration

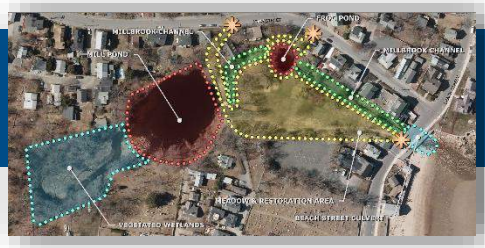
Town of Rockport, Massachusetts | July 23, 2014

Structures Inspection Report

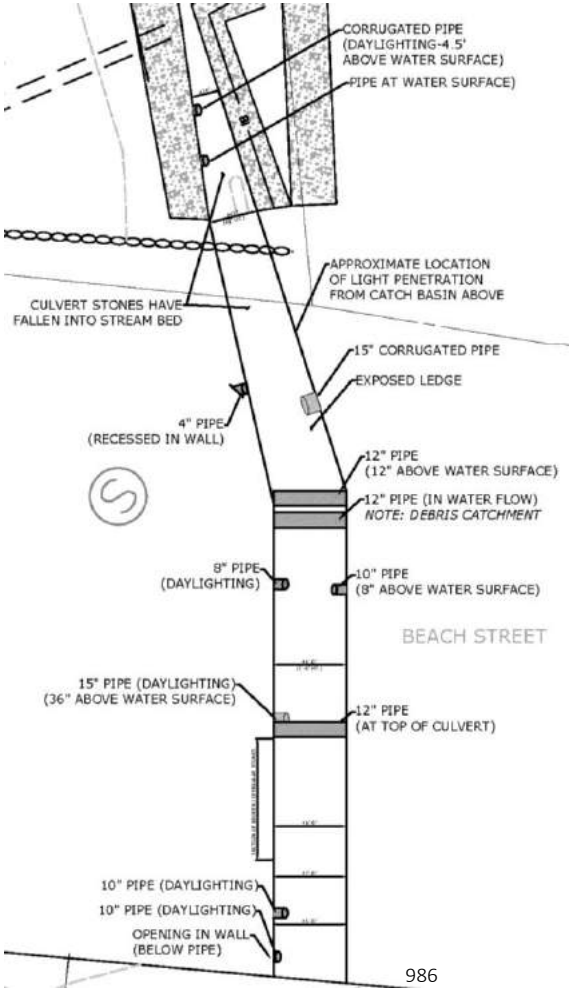


Structure	Type/Material	Photo No.	Condition	Recommendations
Wall 1A	Mortared Stone Wall	1 - 5	Good Condition	None
Wall 1B	Mortared Stone Wall	2 - 5	Satisfactory Condition	None
Wall 2	Dry Stacked Stone Wall	6	Fair Condition	Adjust stones as necessary add additional stones as necessary
Wall 3	Dry Stacked Stone Wall	6	Fair Condition	Adjust stones as necessary add additional stones as necessary
Wall 4	Dry Stacked Stone Wall	7 - 9, 11, 13	Fair Condition	Restack stones as necessary
Wall 5	Dry Stacked Stone Wall	7 - 15	Fair Condition	Restack stones as necessary. Repair existing sink hole
Wall 6	Dry Stacked Stone Wall	16 - 21	Poor to Fair Condition	Wall is overturning in locations. Re-lay stones has necessary
Wall 7	Mortared Stone Wall	24, 25	Fair Condition	None
Wall 8	Dry Stacked Stone Wall	24	Fair Condition	None
Wall 9	Dry Stacked Stone Wall	27, 31 - 34	Poor Condition	Wall is overturning in locations. Re-lay stones and repair existing sinkhole as necessary.
Wall 10	Mortared Stone Wall	28	Fair Condition	Repoint as necessary
Wall 11	Dry Stacked Stone Wall	38, 39	Satisfactory Condition	Fill in voids
Wall 12	Dry Stacked Stone Wall	41 - 43	Fair Condition	None
Wall 13	Dry Stacked Stone Wall	36, 37	Fair Condition	None
Wall 14	Dry Stacked Stone Wall	40	Fair Condition	Re-lay stones as necessary
Wall 15	Dry Stacked Stone Wall	45	Fair Condition	Re-lay stones as necessary
Timber Bridge 1	Timber	22, 23	Fair Condition	Stones beneath structure shall be re-laid as necessary
Timber Bridge 2	Timber	35 - 37	Fair Condition	Stones beneath structure shall be re-laid as necessary
Stone Bridge	CIP Concrete and Mortared Stone	25 - 27, 29, 30	Fair Condition	Repoint as necessary
Beach Street Culvert	Stone	46 - 57	Poor to Serious Condition	Culvert requires an in-depth inspection for rehabilitation options
Stone Walkway/Walls	Stone	44	Satisfactory Condition	None

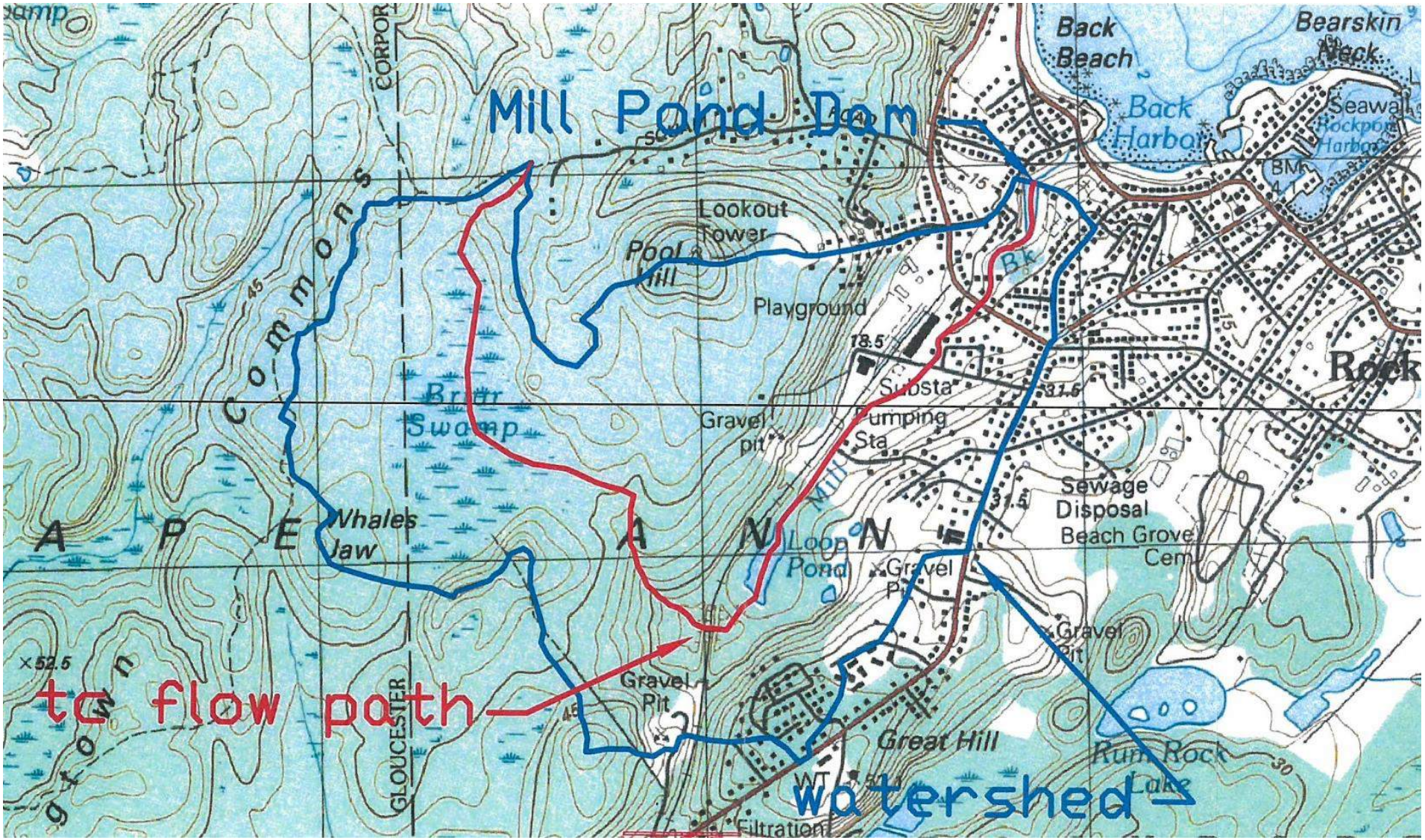
BEACH ST. CULVERT

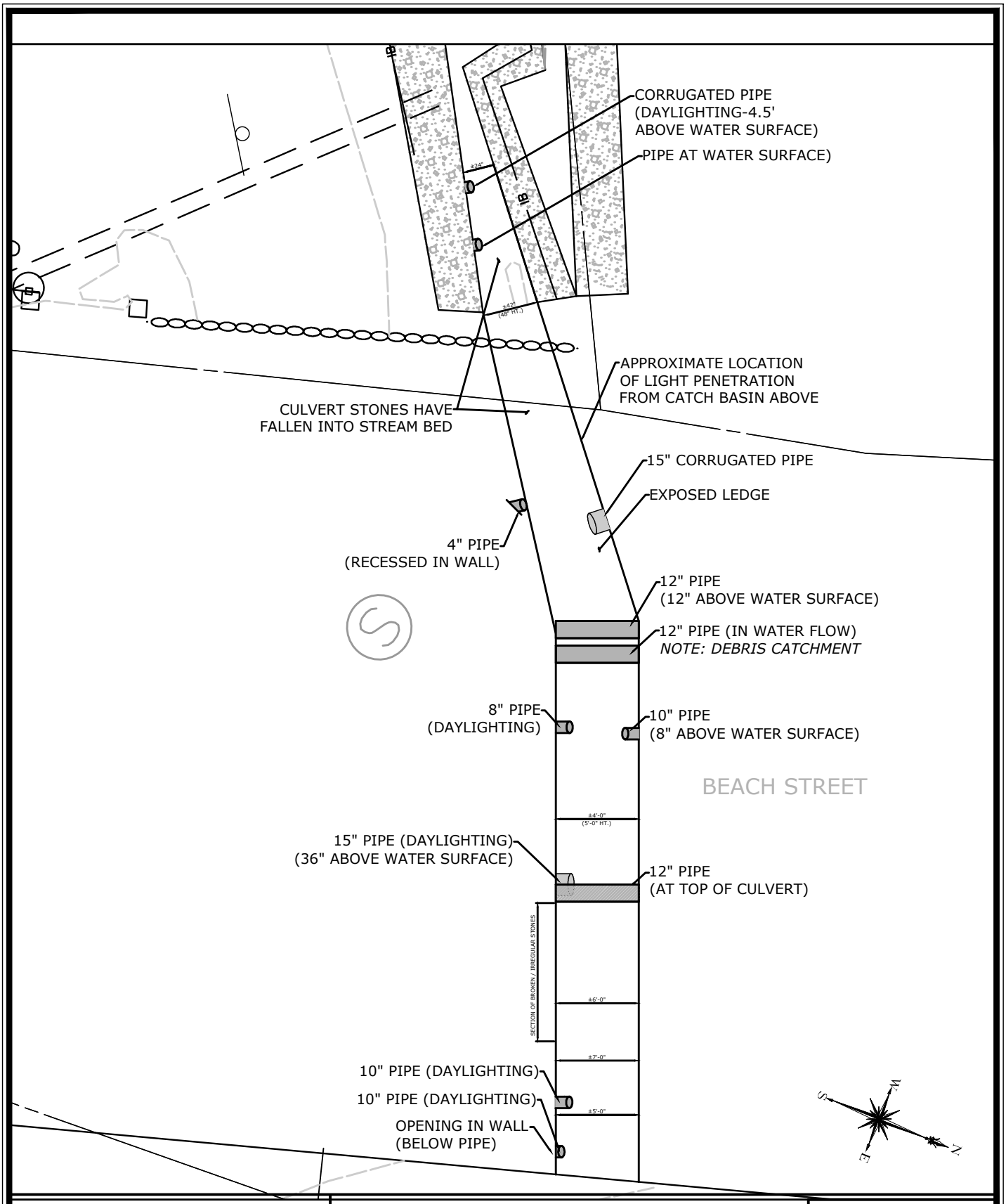


DOUBLE 12" PIPES AND FALLING STONES CATCH DEBRIS



Watershed


Mill Pond Dam
tc flow path
watershed
Back Beach
Bearskin Neck
Back Harbor
Seawall
Rockport Harbor
Lookout Tower
Pool Hill
Playground
Substa
Pumping Sta
Gravel pit
Briar Swamp
Whales Jaw
Loon Pond
Gravel Pit
Sewage Disposal
Beach Grove Cem
Great Hill
Rum Rock Lake
Filtration
GLOUCESTER
CORPOR
A P E A N N
Commons
Town



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EXISTING CULVERT DIMENSIONS

MILL BROOK MEADOW AND MILL POND RESTORATION

ROCKPORT, MASSACHUSETTS

DATE	**2014-9-2**
SCALE	**NTS**
PROJ. NO.	**5220-02**
DRAWING NAME:	**FIG. 1**

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Drawing: W:\DESIGN\5220-02-DE\CAD\NONPLANSET\CULVERT STRUCTURAL ANALYSIS.DWG Layout Tab:LAYOUT1-8.5X11V
Plotted by: JASONW On this date: Mon, 2014 December 8 - 3:00pm

H & H Analysis and Results

Existing Watershed and Channel	
Drainage Area	0.88 mi2
2-year Flow	233 cfs
“Bankfull” Flow	34 cfs

- Straight, confined
- Poor substrate
- Frequent Access to “floodplain”

Proposed Channel	
“Bankfull” Width	15 feet

- Sinuous, riffle-pool
- Sand/Gravel/Cobble substrate
- Less frequent access to floodplain
- Increased storm buffer capacity
- Increased channel diversity

Wetland Function and Values

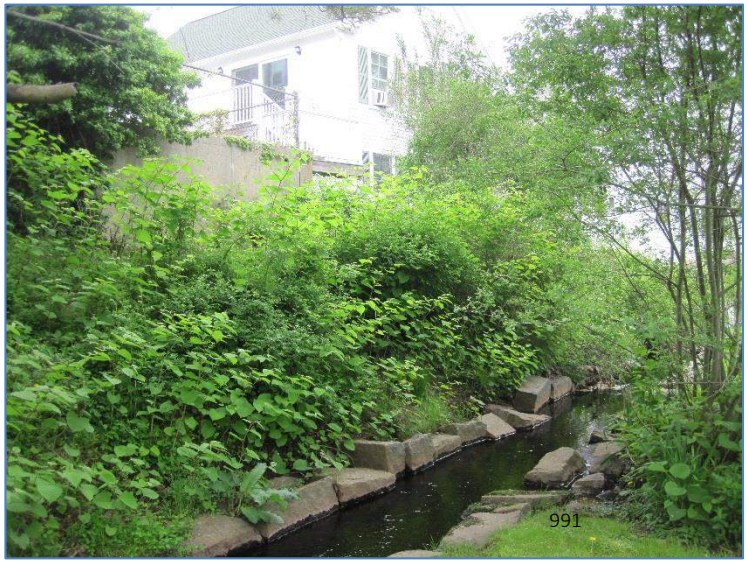
- **Mill Pond Palustrine Emergent/Scrub Shrub Wetlands**
  - **Wildlife Habitat**
  - **Nutrient Removal**
  - **Sediment/Toxicant Retention**
  - **Visual Quality**

- **Mill Pond Palustrine Open Water**
  - **Wildlife Habitat**
  - **Fish Habitat**
  - **Recreational**
  - **Nutrient Removal**
  - **Sediment/Toxicant Retention**
  - **Visual Quality**

Wetland Function and Values

- **Frog Pond**
  - **Wildlife Habitat**
  - **Nutrient Removal**
  - **Sediment/Toxicant Retention**
  - **Visual Quality**

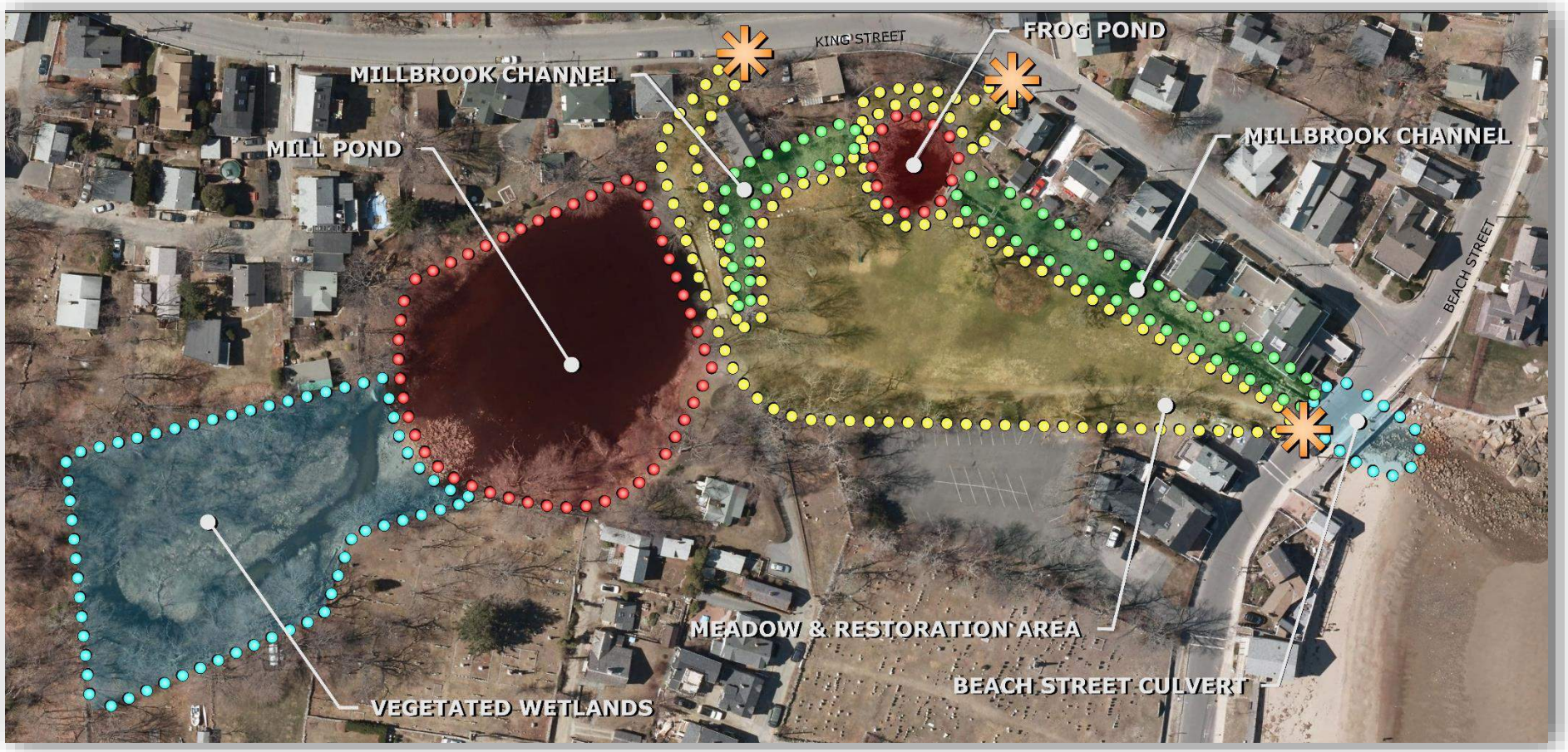
- **Mill Brook Channel and Wet Lawn**
  - **Fish Habitat**
  - **Recreational**
  - **Groundwater Discharge (Lawn Area)**
  - **Visual Quality**



Invasive Plant Management

Types of Invasive Plant Species	Recommended Treatment and Management
Japanese Knotweed	Multi-Year Herbicide Treatment (Habitat)
Common Reed (*Phragmites*)	Multi-Year Herbicide Treatment (Habitat)
Purple Loosestrife	Loosestrife Beetle and/or Multi-Year Herbicide Treatment (Habitat)
Multiflora Rose	Mechanical Excavation and/or Multi-Year Herbicide Treatment (Habitat)
Oriental Bittersweet	Mechanical Removal and/or Multi-Year Herbicide Treatment (Habitat)
Climbing Euonymus	Mechanical Removal and/or Multi-Year Herbicide Treatment (Habitat)
Poison Ivy	Multi-Year Herbicide Treatment (Habitat)
Dodder	Hand Removal, Pre-Emergent and/or Multi-Year Herbicide Treatment (Habitat)
Garlic Mustard	Hand Removal and/or Multi-Year Herbicide Treatment (Habitat)

Existing Conditions



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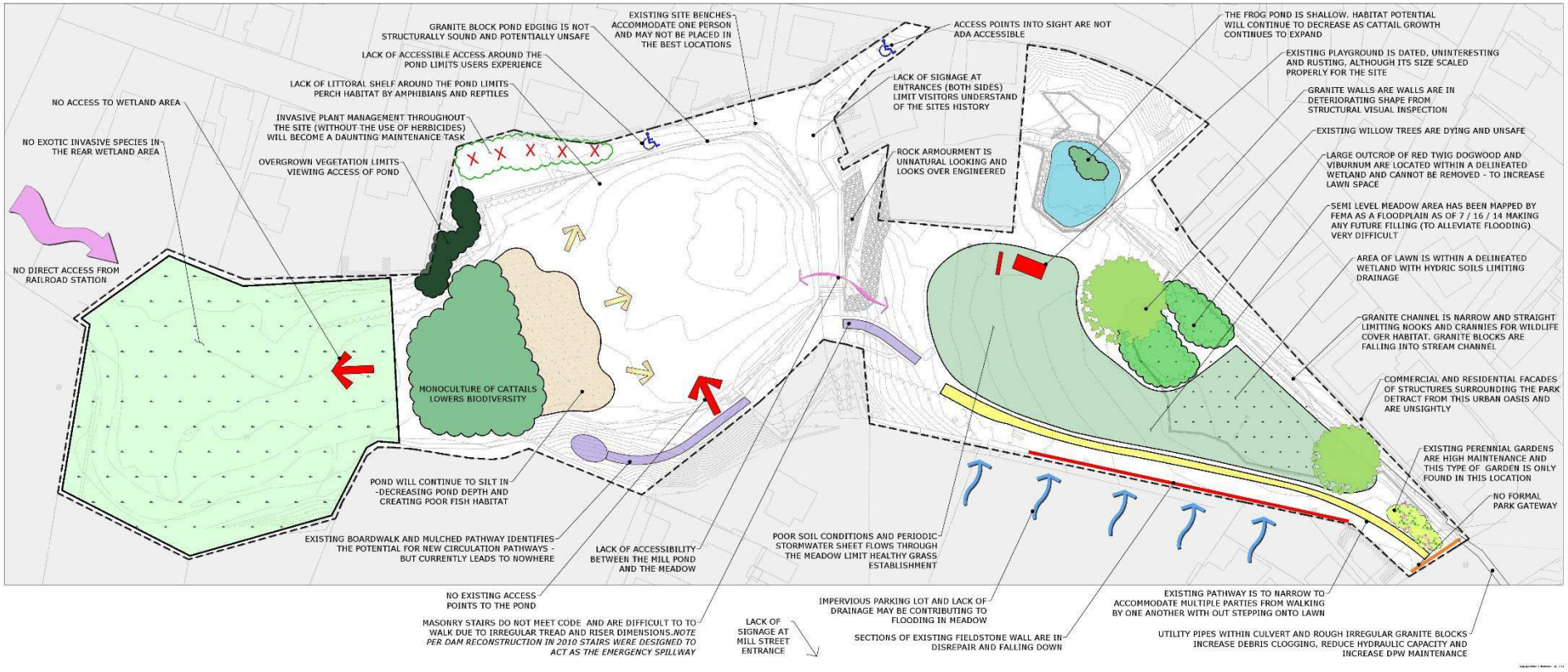
Site Analysis



Concept –A

Ecology of Site

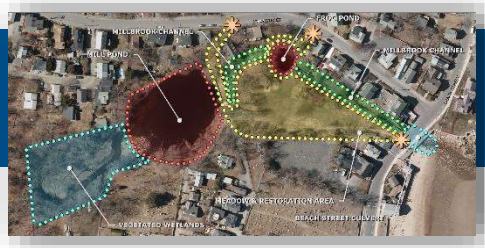
Concept –A (Meadow)

Ecology of Site

RESTORATION AREA
IDENTIFIED DURING SITE ANALYSIS
OUTDOOR SCULPTURES CREATE DESTINATIONS
PROVIDE BENCHES
LOOP TRAIL
SIGNAGE
ALONG EX. GRANITE BLOCK WALL - ILLUSTRATES HOW LARGE GRANITE STONE WERE CUT
HISTORICAL SIGNAGE AT DAM
EMERGENCY SPILLWAY RECONNECTION
• PROVIDE NATURALIZED CONVEYANCE CHANNEL TO WATERCOURSE
• MINIMIZE & PREVENT TRAIL EROSION
NEW ENVIRONMENTAL - INTERACTIVE PLAY "VILLAGE"
• GRANITE FEATURES
• PERFORMANCE STAGE
• INSTRUMENTS
• VEGETATION EXPLORATION
CONSIDER DREDGING POND
INVASIVE REMOVAL / NATIVE HILLSIDE GARDEN
• BIRD BOXES OVERLOOK FROG POND
RECONFIGURED CHANNEL
• BASED ON HEC RAS MODELING
• CREATION OF MICRO HABITATS
• WIDE POOLS
• NARROW RIFFLES
VEGETATED PROGRAM AREA
• EDUCATIONAL SIGNAGE TIDAL INTEGRATION
• COVER / PERCH / FOOD SOURCE HABITAT
PEDESTRIAN WATER ACCESS
• RE-USE EXISTING GRANITE
POND
DAM
FROG POND
OPEN LAWN
POND LOOP TRAIL
• BENCHES
• LOW NATIVE SPRING EPHEMERALS PLANTED ALONG TRAIL
• SIGNAGE "WHAT ARE CATTAILS , WHY ARE THEY IMPORTANT "
CONSIDER DREDGING POND
TRAIL HEAD
• POTENTIAL WELCOME SIGNAGE INTO PARK
• TRASH RECEPTACLES / BIKE RACK / SEATING
• SMALL PARKING AREA (HC SPACE(S)
TREE INVENTORY IN WOODLAND AREAS
IDENTIFIES SPECIES TYPE AND HEALTH OF FOREST TO UNDERSTAND REVIVAL AND RE-USE OF DEAD TREES ALONG WITH SOIL SAMPLING HELPS TO IDENTIFY ECOSYSTEM IN ORDER TO SPECIFY NEW PLANT MATERIAL
BOARDWALK SYSTEM
• ACCESS ACROSS POND
• GATHERING SPACES W/ACCESS TO WATER & FISH (IE. SEATING DURING ICE SKATING)
• EDUCATIONAL MONITORING STATION FOR TEMPERATURE, WIND AND ICE THICKNESS
• SIGNAGE ILLUSTRATING POND ECOLOGY A & REASON BEHIND WETLAND PROCESS RESTORATION
OPEN LAWN
• IDENTIFY CURRENT AND FUTURE COMMUNITY NEEDS
• PLAN FOR FUTURE AMENITIES
• NEW UTILITIES (WATER & ELECTRIC) INTO PARK
STORMWATER INFILTRATION
• CATCHES STORMWATER, SILT AND SAND FROM PARKING LOT
• ALLOWS WATER TO INFILTRATE THROUGH LAWN AND INTO STREAM CHANNEL
• NATIVE FLOWERING UNDERSTORY TREES PROVIDE SPRING COLOR & FALL FOOD SOURCE FOR SONG BIRDS - LIVING @ FROG POND
AERATE , TOP DRESS & OVERSEED OPEN LAWN
• CONSIDER YEARLY MAINTENANCE PLAN IF DESIRED
ADA ACCESSIBLE LOOP PATH
• BENCHES
• LOW LEVEL LIGHTING
SMITH ST
KING ST
BEACH ST
UNION LN
MAIN ST
MILL LN
TIDAL BACKFLOW
NORTH

THE FROG POND

Concept - A



NATIVE PLANT CONTEMPLATIVE SHADE GARDEN

- MOVEABLE SEATING
- SIGNAGE DEVOTED TO "CHAMPIONS OF THE PARK"
- BIRD BOXES IN TREES
- UPLIGHT FLOWERING TREES
- GRANITE STEPS INTO POND
- CREATED LITTORAL ZONE AROUND POND

AMERICAN BURREED

PICKERELWEED

TURTLEHEAD

POLYGONATUM

CINNAMON FERN

DUCK POTATO

LURID SEDGE

Concept –A (Mill Pond)

Ecology of Site

BOARDWALK

Concept - A

Concept-A Boardwalk

+/- 800 LF @ $700/LF = $560,000

BOARDWALK

Perspective

Existing Conditions

Proposed Conditions

Concept –B

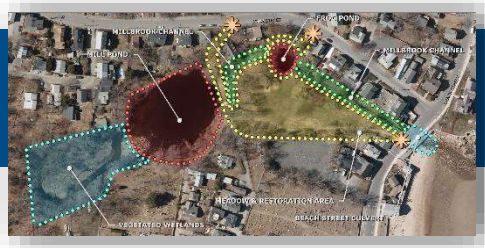
History of Place

Concept –B (Meadow)

History of Place

THE FROG POND

Concept - B



HORTICULTURAL WOODLAND SHADE GARDEN

- GARDEN CLUB
- SIGNAGE IDENTIFIES PLANT SPECIES
- GRANITE SEATING BLOCKS INTO POND
- UPLIGHT GRANITE PLINTHS AND SEDGES IN LITTORAL SHELF
- BOARDWALK OVERLOOK

HELLEBORES

BUGBANE

ASTILBE

HOSTA

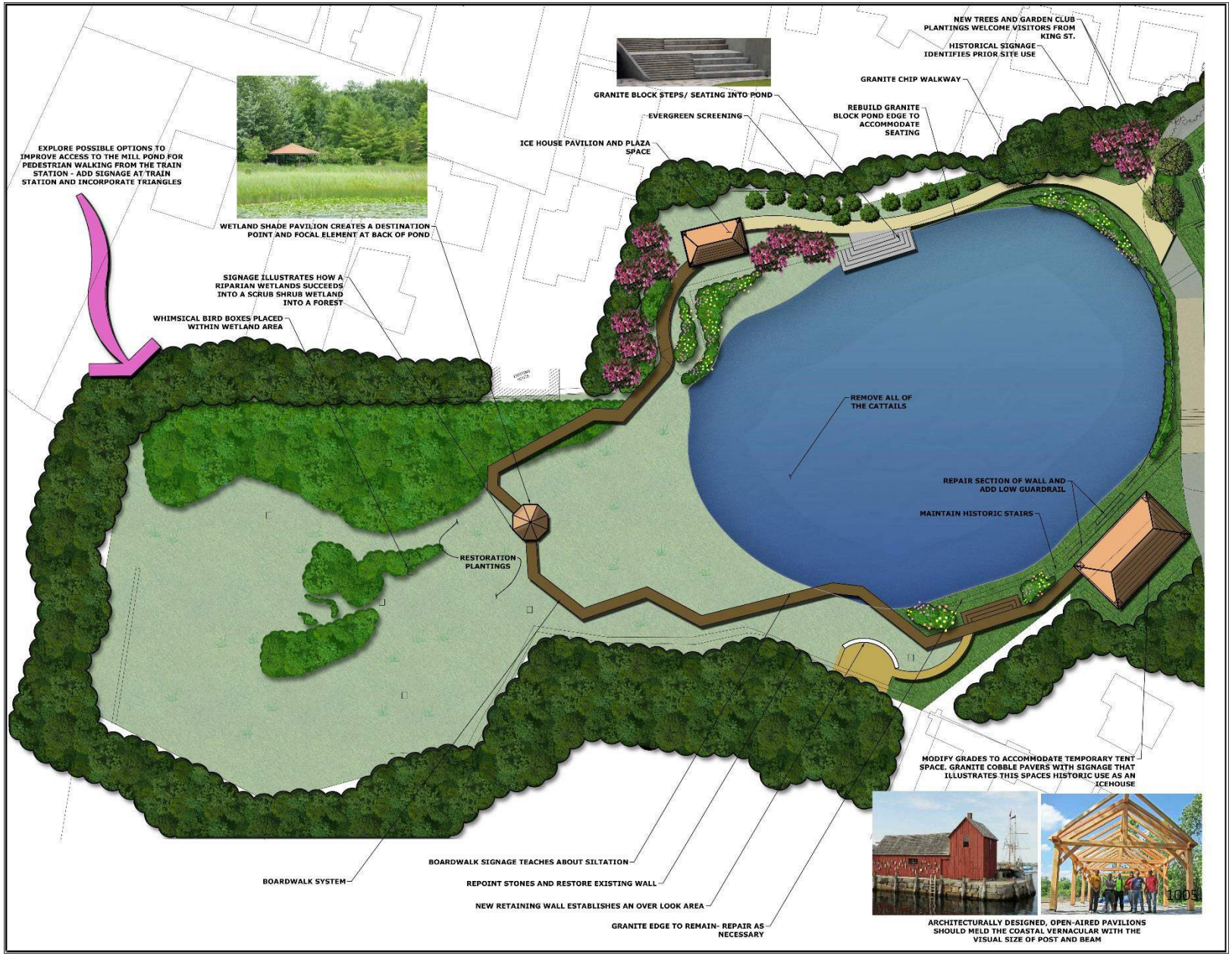
HAYSCENTED FERN

HEUCHERELLA

VARIEGATED POLYGONATUM

Concept –B (Mill Pond)

History of Place

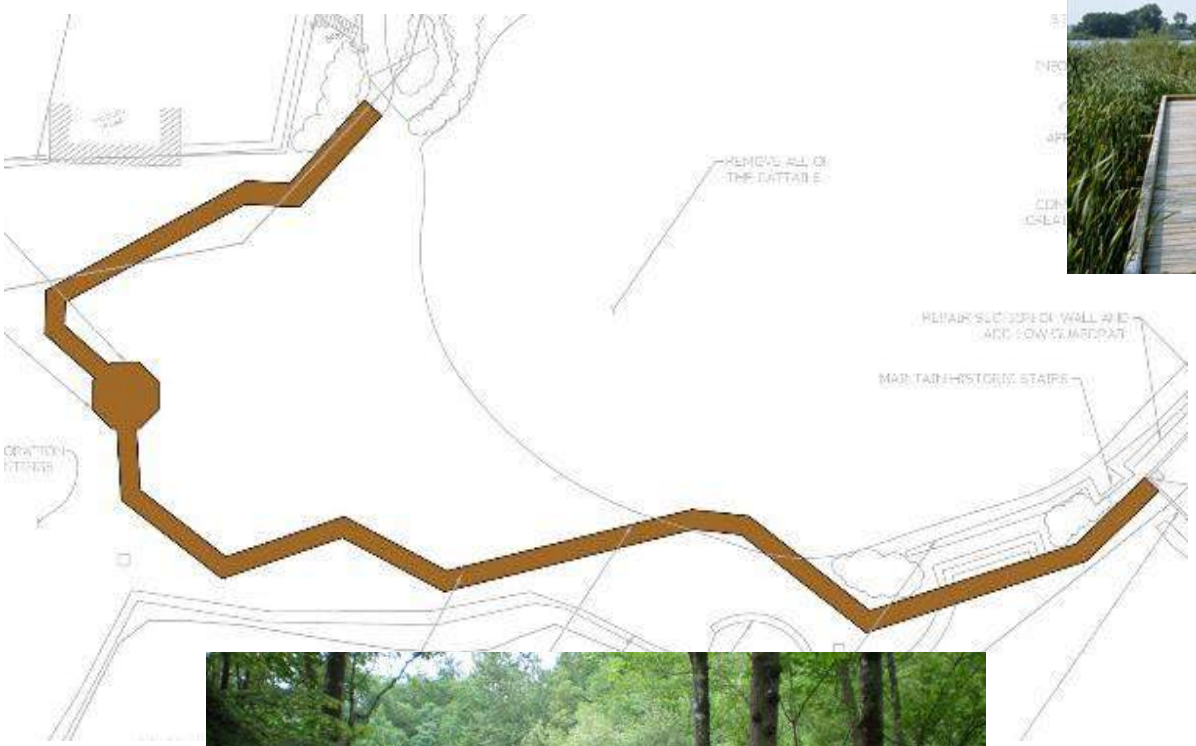


BOARDWALK

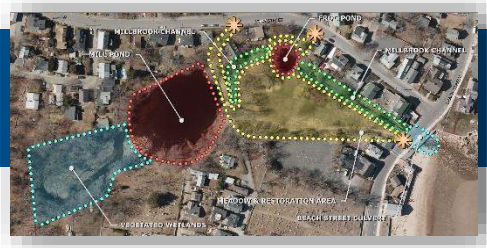
Concept - B

Concept-A Boardwalk

+/- 500 LF @ $700/LF = $350,000



REAR WETLAND (Restoration)



BLUE FLAG IRIS

LURID SEDGE

MILKWEED

STEEPLEBUSH

Concept – A & B

ROYAL FERN

DUCK POTATO

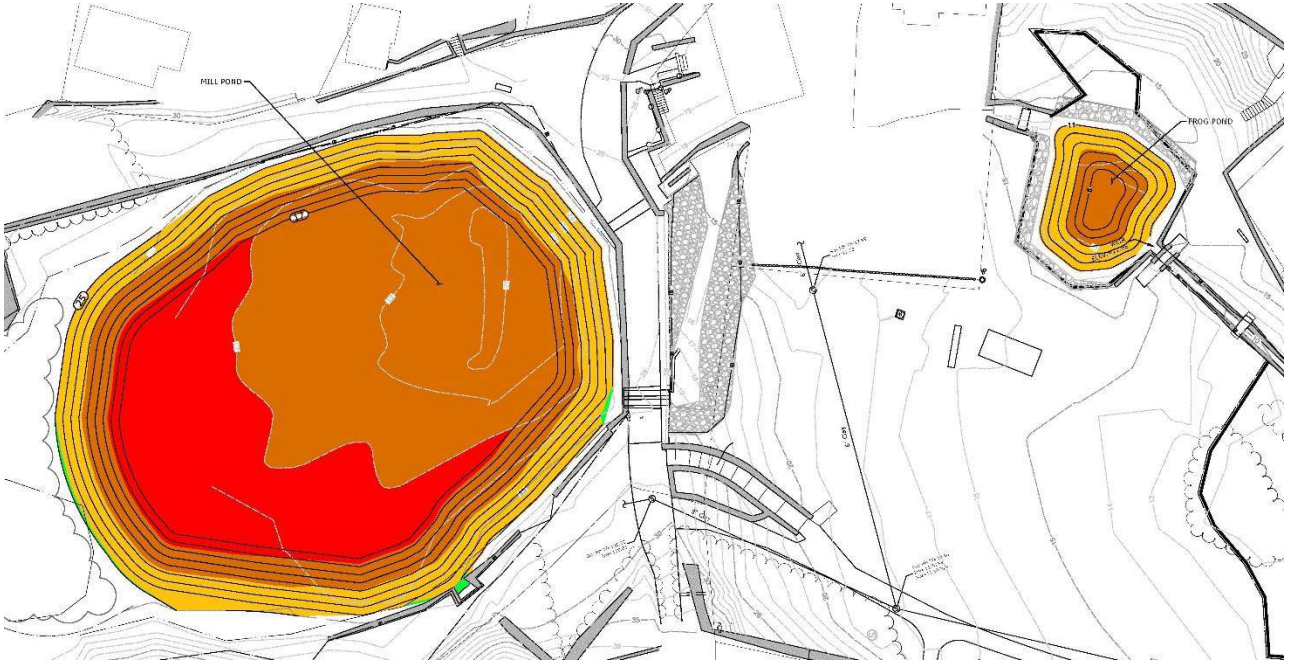
Mill and Frog Pond Dredging

Mill Pond

- Dredging from existing edges of pond to 8' depth with 3:1 (33% slopes)
- 5,000 CY of material dewatered = 3,750 TS
- Excavate, manage, transport & dispose = $265,000 - $300,000

Frog Pond

- Dredging from existing edges of pond to 4'-0" depth with 3:1 (33% slopes)
- 195 CY of material dewatered = 150 TS
- Excavate, manage, transport & dispose) = $10,500 - $12,000



EARTHWORK SUMMARY		
DESCRIPTION	QUANTITY	UNIT
MILL POND		
CUT	5,000 ±	CY
FILL	0 ±	CY
FROG POND		
CUT	195 ±	CY
FILL	0 ±	CY
MASS EARTHWORK SUMMARY		
MASS CUT	195 ±	CY
MASS FILL	0 ±	CY
TOTAL NET EXPORT	5,195 ±	CY

Elevations Table			
Number	Minimum Elevation (ft.)	Maximum Elevation (ft.)	Color
1	-7.457	-6.000	
2	-8.000	-8.000	
3	-8.000	[illegible]	

Questions

Park Maintenance

Spring

- Leaf removal as needed and during first mow of the year
- Remove debris and fallen branches following storm events
- Remove debris from Beach St. Culvert
- Mowing

Summer

- Mow, weed wack, blow and remove clippings (every Friday)
- Prune dead branches as necessary on smaller trees
- Trash pick up twice per week (2 barrels)
- Swing set maintenance

Winter

- Snow removal along Mill Lane only

Fall

- Mowing

Dredging Determination

Mill Pond

- Dredging from existing edges of pond to 8' depth with 3:1 (33% slopes)
- 5,000 CY of material - dewatered to 2,500 CY
- 2,500 CY x 1.5 = 3,750 tons
- @ $70 - $80/ ton
- Excavate, manage, transport & dispose = $265,000 - $300,000

Frog Pond

- Dredging from existing edges of pond to 4'-0" depth with 3:1 (33% slopes)
- 195 CY of material dewatered = 100 CY
- 100 CY x 1.5 = 150 tons
- @ $70 - $80/ ton
- Excavate, manage, transport & dispose) = $10,500 - $12,000