# Appendix H

# Scalley Dam Evaluation Study

**AECOM Environment**
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AECOM

March 13, 2009

Margaret White
Town Planner
Town of Winchester
Town Hall
71 Mount Vernon Street
Winchester, MA 01890

Re: Evaluation of Scalley Dam (Horn Pond Dam) Control Structure, Woburn
Aberjona River Flood Mitigation Program Element

Dear Ms. White:

Per the scope of our May 30, 2008 contract with the Town we have completed the evaluation of the feasibility of making modifications to the control structure at Scalley Dam (aka Horn Pond Dam) in Woburn. The structural evaluation, development of options, and cost estimating for this assignment was accomplished by our sub-consultant Weston and Sampson Inc. We have enclosed a copy of their report. AECOM evaluated the hydraulics of the options prepared by Weston & Sampson. Both Weston & Sampson and AECOM recommend Option B, the installation of a new 8-feet-wide primary spillway/sluice gate to augment the existing 5-feet-wide primary spillway/sluice gate.

*Project Background*

The objective of the engineering study was to evaluate the feasibility of improving the hydraulic opening of the primary outlet control for Horn Pond. The existing structure (5 feet wide by 4 feet high sluice gate) is overwhelmed by large storm events, causing uncontrolled flow over the secondary spillway of Horn Pond into Horn Pond Brook. This, in turn, causes problems downstream all the way through the confluence of Horn Pond Brook and the Aberjona River. Modeling of the system as part of the DEIR and SDEIR showed that by increasing the capacity of the primary spillway (the control structure) that the peak flow hydrograph for a storm event could be “flattened out” by letting more water out of the pond in a controlled fashion throughout the storm event. This helps to maintain the pool elevation of Horn Pond below the elevation of the secondary spillway and therefore helps to prevent overtopping of the secondary spillway and uncontrolled flow to the brook.

The current study was focused on the feasibility of constructing a new primary spillway, the costs of such a project, and the hydraulic impact of various options for the spillway. We also evaluated the impact on the Lake Avenue culvert which is immediately downstream of the spillway.

*Findings*

Working with Weston & Sampson we first evaluated increasing the size of the primary spillway from its current 5-feet-wide sluiceway to a 10-feet-wide sluiceway. To work effectively we also

proposed to raise the secondary spillway elevation by one foot, thereby forcing more water though the new primary sluiceway as opposed to allowing the secondary spillway to overtop. Operationally this configuration would necessitate opening the sluice gate earlier in the storm to drawdown the pool elevation of Horn Pond before the peak of the storm. The City Engineer in Woburn did not prefer this option because of the potential for it to cause an increased pool elevation of Horn Pond if the sluice gate was not operated correctly.

AECOM then ran an optimization analysis (using multiple model runs of the FEMA HEC-RAS model refined for the SDEIR for the Mystic River watershed) to see what size the primary spillway would have to be if the secondary spillway were kept at the same elevation to provide a similar level of improvement as seen by SDEIR Alternative 7. Based on our hydraulic evaluation the required additional sluice needs to be 8-feet-wide, rather than 5-feet-wide. We agree with Weston & Sampson's recommendation of placing the new sluice in close proximity to the old one.

Based on the results of our optimization analysis for the design flood (100 year) AECOM performed additional HEC-RAS model runs for the 2, 5, 10, 50, and 500 year storm events for Option B. Table 1 presents a comparison of the predicted max water surface elevations upstream and downstream of Scalley Dam for baseline conditions, the preferred Alternative from the SDEIR (Alternative 7), and Option B. The downstream predictions are located just above Lake Avenue. Table 2 presents a similar comparison of the predicted max water surface elevations upstream and downstream of Lake Avenue for baseline conditions, the preferred Alternative from the SDEIR (Alternative 7), and Option B. The 25- and 500- year storms were not run as part of the SDEIR so model results are not available. It was assumed for all model runs that 6 hours into the 24-hour storm event that both sluice gates were opened to their maximum extent. Actual sluice gate operation for smaller storms will likely vary, resulting in higher water elevations in Horn Pond but lower downstream flows. Table 3 presents peak flow and velocity predictions just downstream of Lake Avenue for baseline conditions, the preferred Alternative from the SDEIR (Alternative 7), and Option B.

**Table 1: Baseline Conditions, SDEIR Alternative 7, and Option B**
**Water Surface Elevation Comparisons (feet NAVD 88)**

| | Upstream[1] | | | Downstream[2] | | |
|---|---|---|---|---|---|---|
| **Event** | **Baseline** | **Alternative 7** | **Option B** | **Baseline** | **Alternative 7** | **Option B** |
| 2-Year | 40.1 | 37.9 | 37.7 | 36.9 | 36.5 | 36.7 |
| 5-Year | 40.5 | 38.7 | 38.3 | 37.3 | 37.1 | 37.3 |
| 10-Year | 41.1 | 39.2 | 38.9 | 37.8 | 37.6 | 38.0 |
| 25-Year | 41.6 | | 39.8 | 38.9 | | 39.0 |
| 50-Year | 42.1 | 41.5 | 40.8 | 40.8 | 40.0 | 40.4 |
| 100-Year | 42.6 | 42.8 | 42.0 | 42.0 | 41.1 | 41.6 |
| 500-Year | 43.6 | | 43.5 | 43.4 | | 43.3 |

*1. Predicted peak water surface elevations just upstream of Scalley Dam, model cross-section Horn Pond, 1, 6422.369.*
*2. Predicted peak water surface elevations just downstream of Scalley Dam, model cross-section Horn Pond, 1, 6389.713.*

**Table 2: Baseline Conditions, SDEIR Alternative 7, and Option B Impacts on the Lake Avenue Culvert[1]; Water Surface Elevation Comparisons (feet NAVD 88)**

| | Upstream[2] | | | Downstream[3] | | |
|---|---|---|---|---|---|---|
| **Event** | **Baseline** | **Alternative 7** | **Option B** | **Baseline** | **Alternative 7** | **Option B** |
| 2-Year | 36.9 | 36.5 | 36.7 | 36.2 | 35.9 | 35.9 |
| 5-Year | 37.3 | 37.1 | 37.3 | 36.4 | 36.3 | 36.4 |
| 10-Year | 37.8 | 37.6 | 38.0 | 36.7 | 36.6 | 36.8 |
| 25-Year | 38.9 | | 39.0 | 37.3 | | 37.3 |
| 50-Year | 40.8 | 40.0 | 40.4 | 37.7 | 37.6 | 37.6 |
| 100-Year | 42.0 | 41.1 | 41.6 | 38.1 | 37.7 | 37.9 |
| 500-Year | 43.4 | | 43.3 | 39.1 | | 39.0 |

*1. The Lake Avenue culvert invert at the upstream face is approximately 34.5 feet and the top of opening at the upstream face is approximately 38.7 feet. The elevation of Lake Avenue is approximately 42.5.*
*2. Predicted peak water surface elevations just upstream of Lave Ave, model cross-section Horn Pond, 1, 6389.713.*
*3. Predicted peak water surface elevations just downstream of Lave Ave, model cross-section Horn Pond, 1, 6273.34.*

**Table 3: Baseline Conditions, SDEIR Alternative 7, and Option B Peak Flows and Velocity Predictions Downstream of Lake Avenue Culvert (CFS)**

| | Flow | | | Velocity | | |
|---|---|---|---|---|---|---|
| **Event** | **Baseline** | **Alternative 7** | **Option B** | **Baseline** | **Alternative 7** | **Option B** |
| 2-Year | 100 | 71 | 78 | 3.7 | 3.3 | 3.4 |
| 5-Year | 130 | 117 | 130 | 4.0 | 3.8 | 4.0 |
| 10-Year | 179 | 158 | 188 | 4.4 | 4.2 | 4.5 |
| 25-Year | 287 | | 280 | 5.2 | | 5.1 |
| 50-Year | 402 | 369 | 377 | 5.9 | 5.7 | 5.7 |
| 100-Year | 572 | 434 | 483 | 6.8 | 6.1 | 6.4 |
| 500-Year | 1071 | | 1033 | 7.5 | | 7.8 |

*1. Predictions just downstream of Lave Ave, model cross-section Horn Pond, 1, 6273.34*
*2. The velocities and flows reported are associated the peak stage from the model run; the value reported is not necessarily the highest value.*

The analyses show that implementation of Option B is not predicted to have a negative impact on either water surface profiles or on velocities upstream or downstream of the Scalley Dam. The analyses also predicted no negative impacts on the functioning of the Lake Avenue culvert but that it reduces the peak flows and water elevations.

*Permitting Considerations*

Work on the outlet structure of Scalley Dam will require filing a Notice of Intent under the Massachusetts Wetlands Protection Act with the City of Woburn Conservation Commission. As currently envisioned, the project appears to also require permitting by the MADEP and USACE under the Clean Water Act (Sections 401 and 404, respectively) but this could change based on a constructability review during the design stage of the project. Amending or applying for a

Waterways (MGL Chapter 91) License may also be required and we recommend filing a Request for Determination of Applicability (under MGL Chapter 91, the Public Waterfront Act) to the MADEP. A Chapter 253 Dam Safety permit from the DCR will also be required. Permitting costs are estimated to be approximately $24,000.

Please contact me with any questions about this report. We are prepared to meet with you and the City of Woburn at your earliest convenience. Please let us know when you have arranged for this meeting.

Thank you again for asking AECOM to assist you with this project.

Sincerely,

Michael J. Toohill, PWS, CE
Project Manager

Jacob San Antonio, PE, CFM
Senior Engineer

Cc: Mark Twogood, Assistant Town Manager, Winchester
David Elmer, Project Manager, Weston & Sampson

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# MEMORANDUM

**TO:** Michael J. Toohill – ENSR Corporation

**FROM:** Mark P. Mitsch, PE
Benjamin T. Green

**DATE:** January 16, 2009 (updated February 27, 2009)

**SUBJECT:** Scalley Dam Spillway Enlargement
Woburn, Massachusetts

Weston & Sampson Engineers, Inc. prepared the January 16, 2009 Technical Memorandum to document our engineering study of methods to double the width of the existing 5 ft. wide gated sluice that forms the primary spillway at Scalley Dam in Woburn, Massachusetts. Subsequent hydraulic modeling by ENSR indicates the proposed gated sluice channel width needs to be 8 ft. rather than 5 ft. We have updated this memorandum to reflect the new requirement.

Scalley Dam regulates the water level in and discharge from Horn Pond. The study is included as Task Three of Chapter 122 of the Acts of 2006 grant for the Town of Winchester related to the Aberjona River Flood Mitigation Program. The project area is indicated on Figure 1, Site Plan.

Previous hydrologic/hydraulic evaluations conducted by ENSR Corporation (ENSR) indicate that peak flow from the main stem of the Aberjona River and peak flow from Horn Pond Brook are approximately coincident at Winchester Center. Alternatives to affect peak rates and timing were investigated. Initial ENSR evaluations indicated that doubling the width of the Scalley Dam sluice (i.e. from 5 ft. to 10 ft.) resulted in no overtopping of the Scalley Dam embankment during 100-year storm, a more controlled and safer discharge to Horn Pond Brook, and offset the timing of the peak discharge from the Horn Pond Brook from that of the main stem of the Aberjona River downstream in Winchester Center. We understand the modeling assumptions included raising the crest of the dam by 1 ft. Subsequent modeling assuming the crest of the dam is not raised indicates the required additional channel width is 8 ft. Weston & Sampson Engineers, Inc. has evaluated the site and determined that a proposed 8 ft. wide channel constructed next to the existing channel is feasible. The attached conceptual design plans illustrate the wider channel alternatives.

Note that this project is not intended to bring the dam into full compliance with the Massachusetts Dam Safety Regulations (302 CMR 10), which indicates the spillway design flood for this dam is the 500-year storm based on the current size and hazard classification for the structure as discussed below.

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## BACKGROUND/EXISTING OUTLET STRUCTURES

Scalley Dam impounds Horn Pond and is located along the southeastern shoreline of the pond. Lake Avenue is located immediately downstream of the dam. Horn Pond is a recreational impoundment. The City of Woburn owns, operates, and maintains the dam. The City also owns and operates seven municipal water supply wells on the west side of Horn Pond (GEI, 1998). An important aspect of the water supply operation is that the impoundment level must be maintained above a minimum elevation as a means of limiting flow of groundwater toward the well field from a known Trichloroethylene (TCE) plume in the area.

Scalley Dam is an approximately 150 ft. long, 14.5 ft. high earthen embankment dam with a primary spillway outlet structure (sluice), auxiliary spillway, and low level outlet works (LLO). Scalley Dam is classified as a LARGE size (based on a maximum pool storage of 1,508 acre-feet presented in GEI 2006 and confirmed in the National Inventory of Dams Database), SIGNIFICANT hazard dam. Based on the size and hazard classification, the spillway design flood for this dam is the 500-year storm according to 302 CMR 10.14.

The upstream and downstream slopes of the dam are armored with riprap slope protection and inclined at approximately 1.5 horizontal to 1.0 vertical (1.5H:1V). The crest width is variable between 10 ft. to 15 ft. in the vicinity of the primary sluice. The crest is reportedly at El. 44.19 (45.0). The elevations in this memorandum that are not in parentheses reference the North American Vertical Datum of 1988 (NAVD). The elevation in parentheses references the National Geodetic Vertical Datum (NGVD), which is the USGS Mean Sea Level Datum of 1929. The crest is surfaced with crushed stone and grass cover. The area to the right (referenced from the pond looking downstream at the dam) of the primary sluice is a grassed park area with a surface level at or above the crest elevation of the dam. Normal pool elevation in Horn Pond is generally maintained between El. 40.69 (41.5) and El. 41.19 (42.0).

The sluice was designed by Whitman & Howard, Inc. in March 1988 and was reportedly constructed in 1989. The following description is based on the 1988 drawings and field investigation of the structure. The sluice consists of a 5 ft. wide (inside width) by 8 ft. tall reinforced concrete channel. The foundation slab that forms the floor of the sluice is approximately 4.5 ft. thick [between El. 32.69 (33.5) and El. 37.19 (38.0)] and 12 ft. wide. The 1.5 ft. thick vertical channel walls are centered on and structurally connected to the foundation slab. The tops of the channel walls are at El. 45.19 (46.0), about 1 ft. above the crest of the earth embankment. The channel length in the direction of flow is approximately 10.5 ft. There is a 1 ft. thick concrete cutoff beam that extends outward from the outside surfaces of the base slab and channel walls for a distance of 1 ft. The tops of the cutoffs along the outsides of the channel walls are at El. 43.69 (44.5), approximately 0.5 ft. below the crest of the earth embankment.

Reinforced concrete wingwalls extend upstream from the upstream ends of the channel walls at a 60-degree angle away from the channel wall orientation. These wingwalls are about 15 ft. long. The bottoms of the wingwall footings are founded at El. 29.69 (30.5). The tops of the wingwalls

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vary from El. 45.19 (46.0) where they meet the channel walls to El. 40.69 (41.5) at the upstream ends. The backfilled sides of the wingwalls are battered at 3V:1H. The top width of the walls is 1.5 ft. and the bottom width of the walls is about 4.5 ft. so these wingwalls are massive.

The wingwalls on the downstream end of the channel are oriented and constructed similarly to those on the upstream end but are slightly smaller. The bottoms of the footings are at El. 32.69 (33.5) compared to El. 29.69 (30.5). The wall lengths are 11 ft. compared to 15 ft. The tops of the downstream wingwalls slope from El. 45.19 (46.0) to El. 43.69 (44.5). Finally, the wall thicknesses vary from 1 ft. at the top to about 3.5 ft. at the bottom.

There is an aluminum slide gate mounted to the upstream end of the culvert. The gate is 5 ft. wide by 4 ft. high and is mechanically actuated by a wheel located over the center of the gate. A reinforced concrete footbridge spans the channel at the upstream end to provide access to the actuating wheel and to the embankment to the left of the spillway outlet. The slide gate functions as a weir gate, which means the gate is raised to restrict flow and lowered to allow flow over the top of the gate. When operating properly, the range of weir operation is from El. 41.19 (42.0) to El. 37.19 (38.0).

There is a fixed weir auxiliary spillway channel located approximately 50 ft. to the left of the primary spillway. The auxiliary spillway channel is lined by 18 in. diameter riprap overlaying 12 in. of crushed stone and filter cloth. The crest of the auxiliary spillway riprap weir is between approximate El. 40.49 (41.3) (2003 CDM report) and El. 41.19 (42.0). The spillway is 24 ft. long and has an approximately 10H:1V slope and 5H:1V side slopes (1998 GEI).

A 16 in. diameter low-level outlet (LLO) is located to the right of the primary spillway. The 16 in. pipe is reportedly ductile iron with a water distribution style gate valve that is operated from a flush mount casing located on the crest. Reportedly, the invert of the LLO is at El. 35.19 (36.0). Though operable, the LLO is not generally used due to its limited hydraulic capability.

Flow from the existing sluice, auxiliary spillway and LLO converge immediately downstream of the sluice and at the upstream end of a 12 ft. wide by 4 ft high by 50 ft. long reinforced concrete culvert that conveys flow under Lake Avenue. This culvert was reportedly replaced in the late 1990's (1996 or 1998). The upstream invert is at approximate El. 34.49 (35.3). An electric utility cable crosses under the culvert at the location of the upstream sidewalk and two 16 in. diameter water mains are located under the culvert (GEI 1998). The culvert flows to Horn Pond Brook, which meets the Aberjona River downstream.

## SUBSURFACE EXPLORATION PROGRAM

Two borings (symbols designated B-1 and B-2) were shown on the plan titled, "Outlet Structure Horn Pond, Woburn, Massachusetts – Site Plan," which was provided to Weston & Sampson by the City of Woburn. Boring B-1 was reportedly located in the channel downstream of the primary sluice. Boring B-2 was reportedly located in the channel downstream of the auxiliary spillway. No information, soil samples, or logs were available for these borings.

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Geologic Earth Exploration, Inc., of Norfolk, Massachusetts drilled two test borings labeled B1 and B2 on the crest of Scalley Dam to the right (looking downstream) of the existing sluice on October 20, 2008. The area to the left of the sluice was not accessible for the drill rig so the boring program was limited to the two completed borings. Approximate boring locations are shown on Figure 1, Site Plan. Logs of the borings are provided in this Technical Memorandum. Weston & Sampson monitored the drilling activities in the field, classified the soils encountered and prepared the boring logs. The purpose of the investigation was to explore the embankment and foundation soil conditions at the dam.

The borings were advanced to approximately 22 ft. and 20 ft., respectively, below the dam crest by advancing steel casing using the drive-and-wash method. The drive-and-wash method entails driving 4 in. inside diameter steel casing with blows of a 300 lb. hammer falling 24 in. In this case, an automatic trip hammer simulating a 140 lb. hammer falling 30 in. was used to advance the casing. Due to difficult driving conditions encountered near the surface (to approximately 6 ft.), a carbide toothed "spin shoe" was installed on the tip of the casing and the casing was advanced through the upper layer of fill by spinning it using the drill rig drill head. The casing in each case (installed by spinning or driving) was then "washed" out using a soil roller bit and re-circulating water as the cutting fluid.

Standard penetration tests (SPTs) were conducted at continuous (2 ft.) intervals of depth using a standard 2 ft. by 1-3/8 in. inside diameter split spoon sampler driven by blows from an automatic hammer that simulates a 140 lb. hammer falling 30 in. The water surface elevation in Horn Pond during drilling operations was approximately El. 42.29 (43.1).

## SUBSURFACE CONDITIONS

The conditions encountered in each boring were as follows:

### *Test Boring B1*

B1 was located on the crest of the dam near the upstream slope to the right of the LLO operator valve casing as indicated on Figure 1. The boring was advanced to approximately 22 ft. below ground surface or approximately El. 22.19 (23.0). The boring encountered coarse to fine sand with some silt and gravel. It should be noted that an automatic hammer was used to advance the sampler. At an estimated hammer efficiency of 80% to 100%, the corrected N60 values indicate that the fill layer density can be described as medium dense to very dense.

A boulder was encountered while drilling through the fill layer from 2.75 ft. to 4.5 ft. below the dam crest.

The stratum change from the embankment fill layer to the foundation layer was estimated to be at approximately 11 ft. below the dam crest. At this depth, a layer of fine sandy silt that appeared

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undisturbed was encountered. The boring was terminated at 22 ft. Groundwater was measured to be 8.3 ft. below the dam crest [i.e. at about El. 35.89 (36.7)] during drilling operations.

The boring was backfilled with Quickcrete Portland Type I/II Cement with 10 to 15% Cetco Super Gel X High Yield Bentonite grout and patched at grade.

***Test Boring B2***

B2 was located on the downstream side of the dam crest near an area of surficial quarry rock fill as indicated on Figure 1. The boring was advanced to 20 ft. below ground surface or approximately El. 24.19 (25.0). The boring encountered medium dense to very dense, coarse to fine sand with some silt, gravel, and a little clay.

An approximately 4 ft. thick layer of crushed stone and quarry stone was encountered at the surface. Samples were not obtained from this depth range. The stratum change from fill to natural soil was estimated at 11 ft. below the dam crest. At this depth, a layer of organic silt and wood that appeared to be undisturbed was encountered. The boring was terminated at a depth of 20 ft. below ground surface.

Groundwater was measured at about 9.7 ft. below ground surface [about El. 34.49 (35.3)] during drilling operations.

The boring was backfilled with Quickcrete Portland Type I/II Cement with 10 to 15% Cetco Super Gel X High Yield Bentonite grout and patched at grade.

It should be noted that water levels observed in boreholes during drilling do not necessarily represent actual groundwater levels. Groundwater levels fluctuate with time, precipitation, construction activity, and other factors.

***Embankment and Foundation Conditions***

Based on the conditions encountered in the test borings, the embankment consists of medium dense to very dense, coarse to fine sand with varying amounts of silt and gravel. Crushed stone and boulders were observed near the surface of the embankment. The boulders are exposed on the upstream and downstream slopes of the embankment. The foundation soils appear to consist of naturally occurring sand, silt, gravel and clay (glacial till). No low permeability core materials such as a clay core layer, concrete cutoff, or sheeting cutoff were encountered in the explorations.

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## DISCUSSION OF ALTERNATIVES

The alternatives considered for increasing the width of the existing sluice include constructing either an open channel or a box culvert, with invert elevation matching the existing sluice invert and a width of 8 ft. The new channel or culvert could either be adjacent to the left or right walls of the existing sluice (i.e. within the limits of the existing wingwalls) or outside the limits of the wingwalls.

Accessing the left side of the existing channel would require temporary modification of the auxiliary spillway to allow for emergency operation in the event of a large storm. In addition, space for construction equipment and activities is limited to the left of the structure. The right side of the existing sluice is readily accessible and there is sufficient space for construction staging. These advantages of construction on the right side of the existing sluice compared to the left are obvious. Accordingly, the left side of the existing sluice was eliminated from further consideration as a location for the new sluice structure.

The new channel or culvert would be fitted with a slide gate. Two options for the new gate operation are a weir gate (i.e. gate slides down to allow flow over the crest) similar to the existing gate; or a bottom-opening gate (i.e. the gate slides up to allow flow below the gate). Weir gates generally allow more precise control of the reservoir level. Bottom gates allow greater initial discharge, hence more rapid initial reservoir lowering, which may be advantageous for the proposed second sluice. Weir gates can become clogged with floating debris. Bottom gates can become clogged by sediment accumulation or by rocks or other sinking debris accumulation. Both issues are manageable by deflectors or racks installed on the upstream side of the structure and by proper maintenance. In our opinion, a bottom-opening gate installed in the proposed sluice will allow greater operational flexibility and responsiveness to varying storm events, and is considered the preferred alternative.

Considerations for locating the new sluice include the required length of the sluice channel; the location, orientation and configuration of a dredged inlet channel at the upstream end of the new sluice; the construction impacts on the existing sluice wingwalls; and the orientation of the sluice discharge relative to the Lake Avenue culvert headwall. Figures 2 and 3 illustrate Option A and Option B, respectively, for the new sluice alignment. The following summary describes advantages and disadvantages of these options and our recommendation for design development.

As indicated by comparison of Figures 2 and 3, the channel alignment for Option A is somewhat longer than for Option B. Option A will require approximately three to four times the amount of upstream dredging compared to Option B for creating a suitable inlet approach channel. On the other hand, Option B would require demolition of the existing right upstream and downstream wingwalls. Finally, Option A would direct discharge at a greater angle to the orientation of the Lake Avenue culvert as compared to Option B, which has an impact on the efficiency of flow entering the culvert and discharge through the culvert.

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For either option, the existing sluice and auxiliary spillway operation would not be significantly impacted in the event of a large storm during construction, the existing 16-inch LLO pipe can be either removed or grouted in place during construction, and the excavation for the new sluice will provide an opportunity to explore embankment conditions that may have led to seepage observed from that area during past storms so that a suitable seepage remediation measure can be incorporated into the construction project. These issues impact the estimated cost of the options.

Our conceptual-level comparative estimate of design, permitting and construction costs indicates that Option A will have a total project cost on the order of $640,000 compared to approximately $520,000 for Option B. Based on our technical and cost evaluations, we recommend Option B for the proposed sluice.

The estimated construction costs include mobilization/demobilization, environmental controls, earthwork, approach channel dredging (with associated permit requirements), concrete demolition (for Option B), reinforced concrete sluice and discharge channel wall construction, a steel slide gate for the proposed sluice channel, and related site restoration work. We have assumed that the contractor will be allowed to use the area between the parking lot and existing sluice for construction staging. Contractor overhead and profit (assumed as 25 percent of estimated construction cost), an allocation for engineering design, permitting (assumed as 25 percent of the estimated construction cost), construction oversight/management (assumed as 15 percent of the estimated construction cost), and a 25 percent contingency are included in the estimates.

## CONSTRUCTION CONSIDERATIONS

Based on the soil conditions encountered in the borings, normal excavation equipment should be sufficient to conduct the excavation work. However, the equipment should be sufficiently sized to handle the large boulders on the existing discharge channel slope and which may exist within the embankment.

A temporary cofferdam will be necessary on the upstream side of the construction area to allow sluice construction “in-the-dry” and to maintain the water level in Horn Pond within acceptable levels. Dredging the inlet channel can probably be completed using hydraulic dredging methods without dewatering the pond. However, it is possible that the dredged sediments could contain levels of contamination that could impact material handling, transport and disposal costs.

## PROJECT DATA GAPS FOR FINAL DESIGN

Project data gaps include surveyed bathymetry upstream of the construction area to verify dredging quantities, and results of environmental sampling to confirm the presence or absence of environmental contaminants in the pond sediments in the project area. The following data gaps that should be addressed during final design include:

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- Detailed site topographic survey including wetland delineation and pond bottom bathymetry in the project area. The survey proposed for the conceptual design was not conducted because the City of Woburn was able to provide the plans that we used to prepare Figures 1 through 3. These plans were considered suitable for purposes of this conceptual design study but a detailed site survey should be conducted for final design.

- Environmental sampling for disposal characterization in the potential inlet channel dredging area. Sediment and water samples should be collected and analyzed in accordance with a characterization sampling and analysis plan prepared by a Licensed Site Professional.

These items will allow better definition of environmental permitting requirements and excavated materials management planning and disposal costs that will be developed as part of the final design. The costs for addressing these data gaps are included in the conceptual-level estimated costs presented above.

It should be noted that evaluating the Lake Avenue culvert is outside our scope of services. We recommend that ENSR verify the culvert has sufficient capacity to discharge the anticipated flood flows following construction of the new sluice. Also, as noted above, the spillway design flood for the dam is the flood due to the 500-year storm for dam safety purposes. Accordingly, the improvements contemplated in this study may not bring the structure into compliance with current Massachusetts dam safety regulations. The dam and Lake Avenue culvert system and the Horn Pond Brook stream channel downstream of the dam should also be evaluated for the 500-year storm to evaluate the structure relative to the dam safety requirements.

Whether the improvements to the dam are based on the 100-year or 500-year storm, a Chapter 253 Dam Safety permit will be required prior to construction. Our conceptual-level estimated costs include preparation of a 253 Permit Application.

Please contact us if you have any questions or need additional information.

Attachments:
- Figures 1, 2 and 3
- Test Boring Logs

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***References:***

GEI Consultants, Inc., "Scalley Dam Phase I Inspection/Evaluation Report," dated September 7, 2006.

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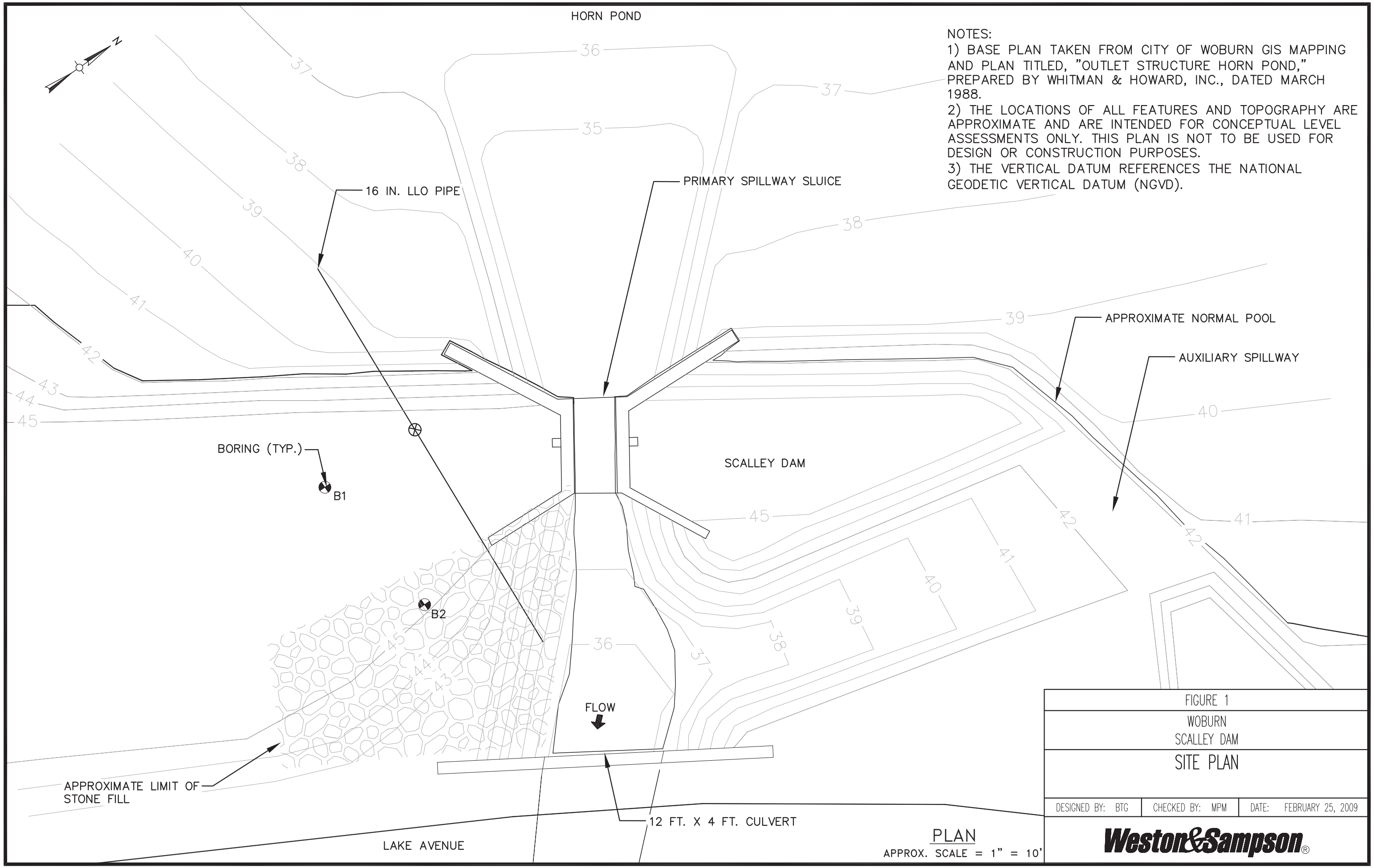
O:\ENSR\Scalley Dam\FINAL REPORT\MEMORANDUM - Scalley Dam updated 022709 (elevations MPM).doc




NOTES:
1) BASE PLAN TAKEN FROM CITY OF WOBURN GIS MAPPING AND PLAN TITLED, "OUTLET STRUCTURE HORN POND," PREPARED BY WHITMAN & HOWARD, INC., DATED MARCH 1988.
2) THE LOCATIONS OF ALL FEATURES AND TOPOGRAPHY ARE APPROXIMATE AND ARE INTENDED FOR CONCEPTUAL LEVEL ASSESSMENTS ONLY. THIS PLAN IS NOT TO BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.
3) THE VERTICAL DATUM REFERENCES THE NATIONAL GEODETIC VERTICAL DATUM (NGVD).

FIGURE 1

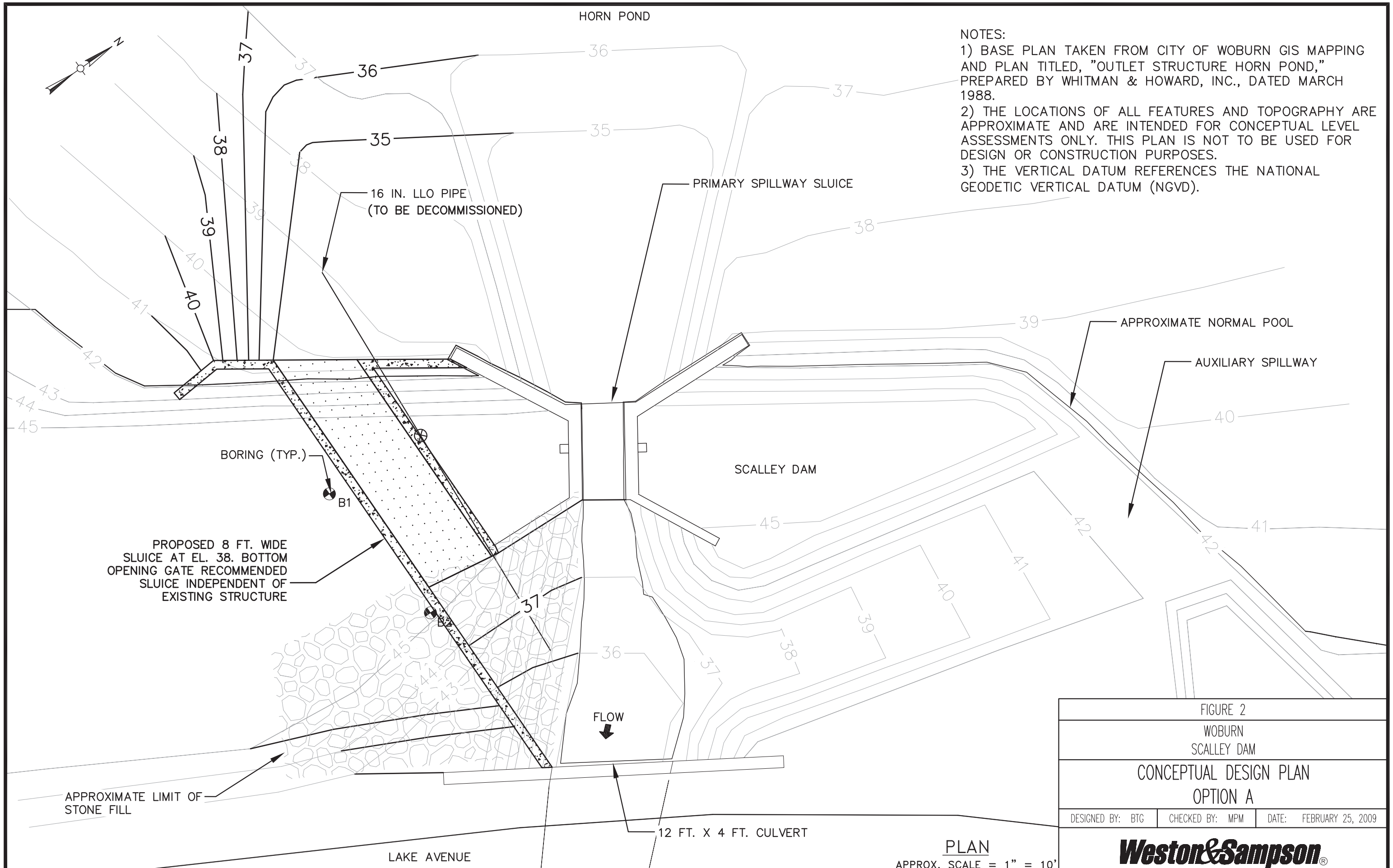
WOBURN
SCALLEY DAM

SITE PLAN

| DESIGNED BY: BTG | CHECKED BY: MPM | DATE: FEBRUARY 25, 2009 |
|---|---|---|

Weston&Sampson®

O:\ENSR\Scalley Dam\CAD\ScalleyDam – BASEPLAN – FIGURE_FINAL-8 ft. wide channel.dwg




NOTES:

1) BASE PLAN TAKEN FROM CITY OF WOBURN GIS MAPPING AND PLAN TITLED, "OUTLET STRUCTURE HORN POND," PREPARED BY WHITMAN & HOWARD, INC., DATED MARCH 1988.

2) THE LOCATIONS OF ALL FEATURES AND TOPOGRAPHY ARE APPROXIMATE AND ARE INTENDED FOR CONCEPTUAL LEVEL ASSESSMENTS ONLY. THIS PLAN IS NOT TO BE USED FOR DESIGN OR CONSTRUCTION PURPOSES.

3) THE VERTICAL DATUM REFERENCES THE NATIONAL GEODETIC VERTICAL DATUM (NGVD).

FIGURE 2

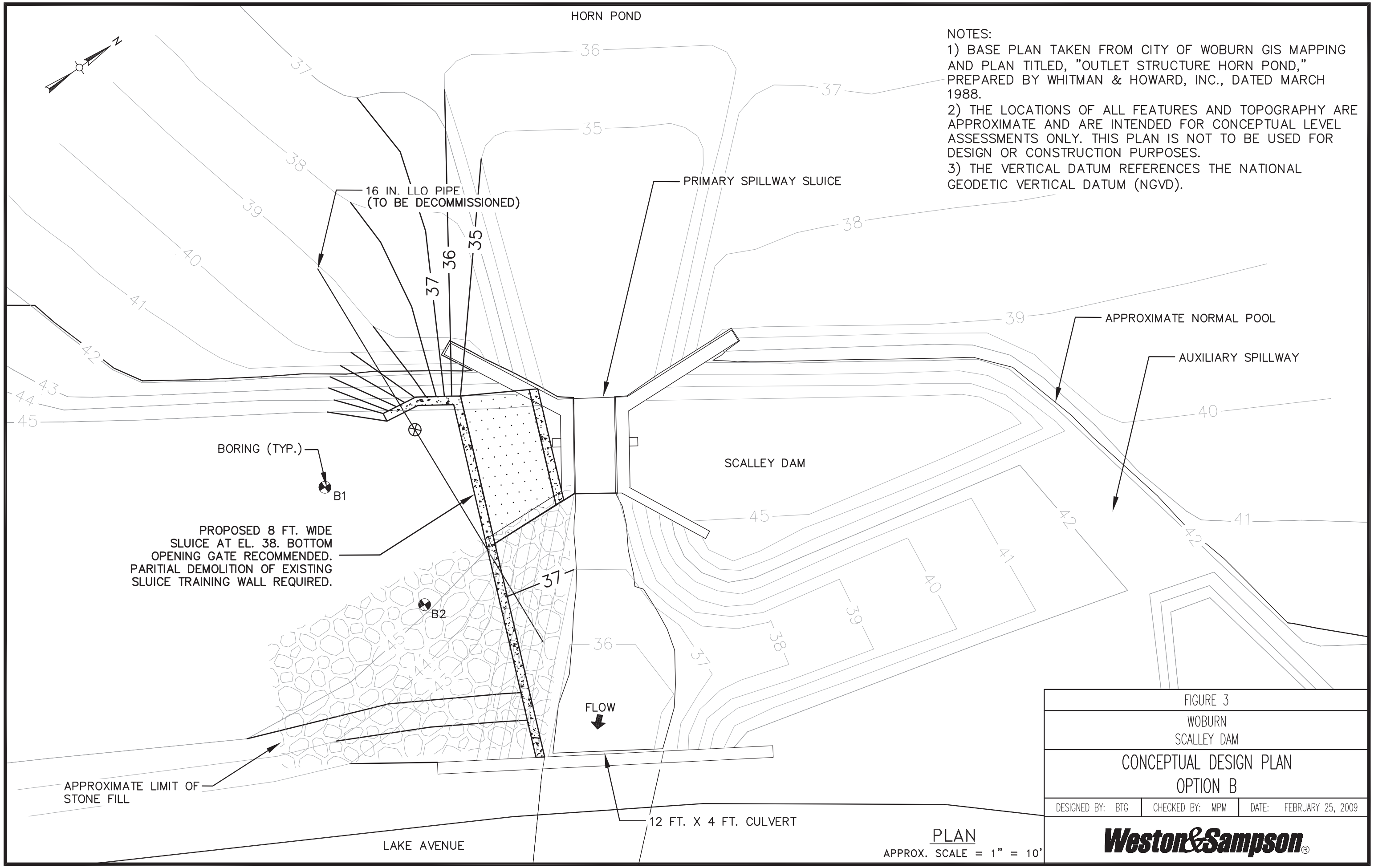
WOBURN
SCALLEY DAM

CONCEPTUAL DESIGN PLAN
OPTION A

| DESIGNED BY: BTG | CHECKED BY: MPM | DATE: FEBRUARY 25, 2009 |
|---|---|---|

Weston&Sampson®

O:\ENSR\Scalley Dam\CAD\ScalleyDam – BASEPLAN – FIGURE_FINAL–8 ft. wide channel.dwg




O:\ENSR\Scalley Dam\CAD\ScalleyDam - BASEPLAN - FIGURE_FINAL-8 ft. wide channel.dwg

| **Weston & Sampson** *ENGINEERS, INC.* | PROJECT<br>ENSR<br>Scalley Dam<br>Woburn, MA | REPORT OF BORING No. B1<br>SHEET 1 OF 2<br>Project No. 2080516.A<br>CHKD BY M. Mitsch |
|---|---|---|

| | | | |
|---|---|---|---|
| BORING Co. | Geologic Earth Exploration, Inc. | BORING LOCATION | See attached plan |
| FOREMAN | Ray | GROUND SURFACE ELEV. | El. 45 +/- DATUM USGS |
| **WSE GEOLOGIST:** | Benjamin T. Green | DATE START 10/20/2008 | DATE END 10/20/2008 |

SAMPLER: SAMPLER CONSISTS OF 2' by 1 3/8" SPLIT SPOON DRIVEN USING A 140 lb. HAMMER FALLING 30 in. (AUTO HAMMER)

CASING: DRIVEN 4" CASING USING DRIVE AND WASH TECHNIQUE CASING DRIVEN BY AUTO HAMMER (140 LB.)

CASING SIZE: 4 inch steel casing OTHER:

GROUNDWATER READINGS

| DATE | TIME | WATER AT | CASING AT | STABILIZATION TIME |
|---|---|---|---|---|
| 10/20 | 1125 | 8.3 ft. | 18 ft. | 15 minutes |
| | | | | |
| | | | | |

| DEPTH (feet) | CASING (lb/ft) | SAMPLE No. | PEN/REC (in) | DEPTH (ft) | BLOWS/6" | PID (ppm) | SAMPLE DESCRIPTION Burmister Classification | NOTES | STRATUM DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| | NA | S-1 | 24/14 | 0-2 | 3-7- | NA | Medium dense, brown/gray, | | *4 in. Topsoil* |
| | | | | | 8-10 | | coarse to medium SAND, little | | |
| | | | | | | | fine gravel, trace fine sand, silt. | | *Embankment Fill* |
| | NA | S-2 | 9/3 | 2-2.75 | 9-100<6in., | NA | Very dense, gray, coarse to fine | 1 | |
| 2.5 | | | | | (Refusal) | | SAND, little silt and fine gravel. | 2 | |
| | | | | | | | | 3 | 2.75 ft. |
| | | | | | | | | | *Boulder* |
| | 4 ft. | S-3a | 24/6.5 | 4.5-6.5 | 9-11- | NA | Medium dense, light and dark | | |
| 5 | (Spun) | | | | 14-13 | | brown coarse to fine SAND, trace | | 4.5 ft. |
| | | S-3b | | | | | silt. 3b= Medium to fine sandy | | |
| | | | | | | | SILT, little clay 2 in. top sampler. | | |
| | 4 ft. | S-4 | 24/6 | 6.5-8.5 | 5-3- | NA | Loose, brown, coarse to fine | 4 | |
| | (Spun) | | | | 5-8 | | SAND, little silt. | | |
| 7.5 | | | | | | | | | *Embankment Fill* |
| | | | | | | | | | Water - 8.3 ft. +/- |
| | 25 | S-5 | 24/8 | 9-11 | 8-9- | NA | Medium dense, dark gray, medium | | |
| | Blows | | | | 3-5 | | to fine SAND, trace coarse sand. | | |
| 10 | | | | | | | | | |
| | NA | S-6 | 24/7 | 11-13 | 17-11- | NA | Medium dense, gray, fine sandy | | |
| | | | | | 10-12 | | SILT, trace clay. | | 11 ft. +/- |
| 12.5 | | | | | | | | | |
| | Not. | S-7 | 24/9 | 14-16 | 17-17- | NA | Dense, brown, silty fine GRAVEL, | | |
| | Rec. | | | | 22-25 | | little coarse to fine sand and clay. | | *Glacial Till* |
| 15 | | | | | | | | | *(Natural Foundation Soil)* |
| | NA | S-8 | 24/12 | 16-18 | 30-34- | NA | Dense, brown, coarse to medium | | |
| | | | | | 14-10 | | SAND, trace silt and fine sand | | |
| | | | | | | | and gravel. | | |

| GRANULAR SOILS | | COHESIVE SOILS | |
|---|---|---|---|
| BLOWS/FT | DENSITY | BLOWS/FT | DENSITY |
| 0-4 | V. LOOSE | 0-2 | V. SOFT |
| 4-10 | LOOSE | 2-4 | SOFT |
| 10-30 | M. DENSE | 4-8 | M. STIFF |
| 30-50 | DENSE | 8-15 | STIFF |
| > 50 | V. DENSE | 15-30 | V. STIFF |
| | | > 30 | HARD |

REMARKS: 1) Used extended sampler, pushed with rig hydraulics to seat at sample interval.
2) Refusal at 2.75 ft. on obstruction. Appears to be a boulder.
3) Installed spin shoe on casing and spun casing using drill head to advance through boulder to 4.5 ft. One seam in rock encountered. Casing was cleared using a roller bit with carbide buttons.
4) While spinning the casing to 8 ft. drill water lost.

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B1

| **Weston & Sampson** *ENGINEERS, INC.* | PROJECT<br>ENSR<br>Scalley Dam<br>Woburn, MA | REPORT OF BORING No. B1<br>SHEET 2 OF 2<br>Project No. 2080516.A<br>CHKD BY M. Mitsch |
|---|---|---|

| | | | |
|---|---|---|---|
| BORING Co. | Geologic Earth Exploration, Inc. | BORING LOCATION | See attached plan |
| FOREMAN | Ray | GROUND SURFACE ELEV. | El. 45 +/- DATUM USGS |
| **WSE GEOLOGIST:** | Benjamin T. Green | DATE START 10/20/2008 | DATE END 10/20/2008 |

SAMPLER: SAMPLER CONSISTS OF 2' by 1 3/8" SPLIT SPOON DRIVEN USING A 140 lb. HAMMER FALLING 30 in. (AUTO HAMMER)

CASING: DRIVEN 4" CASING USING DRIVE AND WASH TECHNIQUE CASING DRIVEN BY AUTO HAMMER (140 LB.)

CASING SIZE: 4 inch steel casing OTHER:

GROUNDWATER READINGS

| DATE | TIME | WATER AT | CASING AT | STABILIZATION TIME |
|---|---|---|---|---|
| 10/20 | 1125 | 8.3 ft. | 18 ft. | 15 minutes |
| | | | | |
| | | | | |

| DEPTH (feet) | CASING (lb/ft) | SAMPLE No. | PEN/REC (in) | DEPTH (ft) | BLOWS/6" | PID (ppm) | SAMPLE DESCRIPTION Burmister Classification | NOTES | STRATUM DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| 17.5 | 212 Blows | S-9 | 24/4 | 18-20 | 11-9-10-15 | NA | Medium dense, black and brown, gravelly coarse SAND. (Drill Wash) | 5 | *Glacial Till (Natural Foundation Soil)* |
| 20 | NA | S-10 | 24/3 | 20-22 | 16-9-10-13 | NA | Medium dense, coarse to fine SAND, some fine gravel, little silt. | 6 | |
| 22.5 | | | | | | | | | 22 ft. - EOB |
| 25 | | | | | | | | | |
| 27.5 | | | | | | | | | |
| 30 | | | | | | | | | |

| GRANULAR SOILS | | COHESIVE SOILS | |
|---|---|---|---|
| BLOWS/FT | DENSITY | BLOWS/FT | DENSITY |
| 0-4 | V. LOOSE | 0-2 | V. SOFT |
| 4-10 | LOOSE | 2-4 | SOFT |
| 10-30 | M. DENSE | 4-8 | M. STIFF |
| 30-50 | DENSE | 8-15 | STIFF |
| > 50 | V. DENSE | 15-30 | V. STIFF |
| | | > 30 | HARD |

REMARKS: 5) Coarse gravel fragment in shoe.
6) Boring grouted with Quickcrete Portland Type I/II Cement (94 lb. sack) and 10-15% Cetco Super Gel X High Yield Bentonite. Patched at grade with cuttings.

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B1

| **Weston & Sampson** *ENGINEERS, INC.* | PROJECT<br>ENSR<br>Scalley Dam<br>Woburn, MA | REPORT OF BORING No. B2<br>SHEET 1 OF 2<br>Project No. 2080516.A<br>CHKD BY M. Mitsch |
|---|---|---|

| | | | |
|---|---|---|---|
| BORING Co. | Geologic Earth Exploration, Inc. | BORING LOCATION | See attached plan |
| FOREMAN | Ray | GROUND SURFACE ELEV. | El. 46 +/- DATUM USGS |
| **WSE GEOLOGIST:** | Benjamin T. Green | DATE START 10/20/2008 | DATE END 10/20/2008 |

SAMPLER: SAMPLER CONSISTS OF 2' by 1 3/8" SPLIT SPOON DRIVEN USING A 140 lb. HAMMER FALLING 30 in. (AUTO HAMMER)

CASING: DRIVEN 4" CASING USING DRIVE AND WASH TECHNIQUE CASING DRIVEN BY AUTO HAMMER (140 LB.)

CASING SIZE: 4 inch steel casing OTHER:

GROUNDWATER READINGS

| DATE | TIME | WATER AT | CASING AT | STABILIZATION TIME |
|---|---|---|---|---|
| 10/20 | 1500 | 9.67 ft. | 16 ft. | 10 minutes |
| | | | | |
| | | | | |

| DEPTH (feet) | CASING (lb/ft) | SAMPLE No. | PEN/REC (in) | DEPTH (ft) | BLOWS/6" | PID (ppm) | SAMPLE DESCRIPTION Burmister Classification | NOTES | STRATUM DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | - No surface sample taken, 1.5 in. +/- diameter Crushed Stone. | 1 | *Crushed Stone* |
| 2.5 | | | | | | | | | |
| | 4 ft. (Spun) | S-1 | 24/9 | 4-6 | 11-15-12-11 | NA | Medium dense, brown, coarse to fine SAND, little silt and fine gravel. | | 4 ft. |
| 5 | | | | | | | | | |
| | 4 ft. (Spun) | S-2 | 24/3 | 6-8 | 10-10-11-7 | NA | Medium dense, brown, coarse to fine SAND, some silt, little fine gravel. | 2 | *Embankment Fill* |
| 7.5 | | | | | | | | | |
| | 18 Blows | S-3 | 24/5 | 8-10 | 14-12-12-30 | NA | Medium dense, brown/gray, coarse to medium SAND, trace silt and fine sand. | | Water - 9.67 ft. +/- |
| 10 | NA | S-4a | 24/9 | 10-12 | 19-9-5-17 | NA | Medium dense, gray, organic SILT, some clay, trace medium to fine sand and wood. 4b= Gray fine SAND, trace silt and fine gravel. | | |
| | | S-4b | | | | | | | |
| | 357 Blows | S-5 | 24/9 | 12-14 | 25-51-26-17 | NA | Very dense, gray/black, gravelly coarse to fine SAND, some silt, frequent black silt pockets. | | 11 ft. |
| 12.5 | | | | | | | | | |
| | | S-6 | 24/12 | 14-16 | 15-26-30-16 | NA | Very dense, brown, gravelly SILT, little clay, coarse to fine sand layer. | 3 | *Glacial Till (Natural Foundation Soil)* |
| 15 | | | | | | | | | |
| | 227 Blows | S-7 | 24/3.5 | 16-18 | 7-9-9-11 | NA | Medium dense, gray/brown, coarse to fine SAND, little fine gravel, silt. | 4 | |

| GRANULAR SOILS BLOWS/FT | DENSITY | COHESIVE SOILS BLOWS/FT | DENSITY |
|---|---|---|---|
| 0-4 | V. LOOSE | 0-2 | V. SOFT |
| 4-10 | LOOSE | 2-4 | SOFT |
| 10-30 | M. DENSE | 4-8 | M. STIFF |
| 30-50 | DENSE | 8-15 | STIFF |
| > 50 | V. DENSE | 15-30 | V. STIFF |
| | | > 30 | HARD |

REMARKS: 1) Boring started by spinning casing using drill head through crushed stone layer to 4 ft. Some drill water break out noted at toe of downstream slope.
2) Spun casing to 8 ft.
3) Predrilled ahead of casing to 16 ft.
4) Fine gravel fragment observed in shoe of sampler.

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B2

| **Weston & Sampson** *ENGINEERS, INC.* | PROJECT<br>ENSR<br>Scalley Dam<br>Woburn, MA | REPORT OF BORING No. B2<br>SHEET 2 OF 2<br>Project No. 2080516.A<br>CHKD BY M. Mitsch |
|---|---|---|

| | | | |
|---|---|---|---|
| BORING Co. | Geologic Earth Exploration, Inc. | BORING LOCATION | See attached plan |
| FOREMAN | Ray | GROUND SURFACE ELEV. | El. 46 +/- DATUM USGS |
| **WSE GEOLOGIST:** | Benjamin T. Green | DATE START 10/20/2008 | DATE END 10/20/2008 |

SAMPLER: SAMPLER CONSISTS OF 2' by 1 3/8" SPLIT SPOON DRIVEN USING A 140 lb. HAMMER FALLING 30 in. (AUTO HAMMER)

CASING: DRIVEN 4" CASING USING DRIVE AND WASH TECHNIQUE CASING DRIVEN BY AUTO HAMMER (140 LB.)

CASING SIZE: 4 inch steel casing OTHER:

GROUNDWATER READINGS

| DATE | TIME | WATER AT | CASING AT | STABILIZATION TIME |
|---|---|---|---|---|
| 10/20 | 1500 | 9.67 ft. | 16 ft. | 10 minutes |
| | | | | |
| | | | | |

| DEPTH (feet) | CASING (lb/ft) | SAMPLE No. | PEN/REC (in) | DEPTH (ft) | BLOWS/6" | PID (ppm) | SAMPLE DESCRIPTION Burmister Classification | NOTES | STRATUM DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| 17.5 | NA | S-8 | 24/12 | 18-20 | 8-8-10-10 | NA | Medium dense, fine gravelly coarse to fine SAND, little silt. | 1 | *Glacial Till (Natural Foundation Soil)* |
| 20 | | | | | | | | | 20 ft. - EOB |
| 22.5 | | | | | | | | | |
| 25 | | | | | | | | | |
| 27.5 | | | | | | | | | |
| 30 | | | | | | | | | |

| GRANULAR SOILS | | COHESIVE SOILS | |
|---|---|---|---|
| BLOWS/FT | DENSITY | BLOWS/FT | DENSITY |
| 0-4 | V. LOOSE | 0-2 | V. SOFT |
| 4-10 | LOOSE | 2-4 | SOFT |
| 10-30 | M. DENSE | 4-8 | M. STIFF |
| 30-50 | DENSE | 8-15 | STIFF |
| > 50 | V. DENSE | 15-30 | V. STIFF |
| | | > 30 | HARD |

REMARKS:
1) Boring grouted with Quickcrete Portland Type I/II Cement (94 lb. sack) and 10-15% Cetco Super Gel X High Yield Bentonite. Due to breakout of drill water in crushed stone layer, grout was brought up to 4 ft. below ground surface. A plug consisting of Pure Gold Medium Bentonite Chips (2 sacks) was placed in the boring and patched at grade with crushed stone.

NOTES:
1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES. TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THIS BORING LOG. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS ARE MADE.

BORING No. B2

# Appendix I

# MHD Main Street Bridge Preliminary Hydraulic Study

**THE COMMONWEALTH OF MASSACHUSETTS**
**MASSACHUSETTS HIGHWAY DEPARTMENT**
**INTEROFFICE CORRESPONDENCE**

---

**TO:** Marie Rose, Director of Project Management

**THRU:** Alex Bardow, Director of Bridges and Structures,

**FROM:** Richard Murphy, Hydraulic Engineer

**DATE:** March 26, 2008

**SUBJECT:** Medford, Br. No. M-12-002, Route 38 (Main Street)/Mystic River, Preliminary Hydraulic Study Report, Project File No. 604716

---

**Transmitted herewith is the Preliminary Hydraulic Study Report prepared by the Hydraulic Section of Directorate of Bridges and Structures for the subject bridge. If you have questions regarding the content of this document, please contact me at (617) 973-7558.**

**RM**
**CC: Alex Bardow, Director of Bridges and Structures**

**BRIDGE NO. M-12-002**
**ROUTE 38 (MAIN STREET) / MYSTIC RIVER**
**MEDFORD, MASSACHUSETTS**

# HYDRAULIC STUDY REPORT

**26 MARCH 2008**

**PREPARED BY:** **HYDRAULIC SECTION**
**DIRECTORATE OF BRIDGES AND STRUCTURES**
**MASSACHUSETTS HIGHWAY DEPARTMENT**

## 1.0 Introduction

### 1.1 Purpose

**The purpose of this report is to present the results of a study conducted at the subject bridge site to determine the hydraulic adequacy of the existing waterway opening. This investigation was conducted in a manner consistent with American Association of State Highway and Transportation Officials (AASHTO), Federal Highway Administration (FHWA), and Massachusetts Highway Department (MHD) guidelines for preparation of hydraulic studies at bridge sites.**

### 1.2 Scope

**The scope of work for this investigation consisted of review of pertinent hydrologic and hydraulic data for the Mystic River at the project site, and a detailed hydraulic analysis. Data collected and hydraulic model computer output is presented in the appendices of this report. The appendices will be made available for review on request. A narrative discussion of the problem statement, engineering methods, results and conclusions of the hydraulic study follows.**

### 1.3 Executive Summary

**MassHighway proposes to rehabilitate the existing bridge conveying Route 38 (Main Street) over the Mystic River (aka, the Cradock Bridge) in Medford, Massachusetts. The existing bridge is a 100-foot long three span composite structure that crosses the river in a south to north orientation. It's two northerly spans are stone-arch structures that were constructed before 1880. The existing bridge's southernmost span consists of a 20-foot reinforced concrete beam structure that was constructed in 1908 to span over the Cradock Dam and Lock system- which was constructed at about the same time. MassHighway proposes to replace the existing 20-foot long southerly span and rehabilitate the northerly twin stone arch structures. Also, the widened section of bridge deck on the west side of twin arches will be replaced with a concrete twin arch structure whose overall dimensions will be similar to those of the original twin arches.**

**Project activities will not adversely affect the vertical profile of the National Flood Insurance Program Regulatory Floodway established for the Mystic River in Medford (see Reference 1).**

## 2.0 Project Description

### 2.1 Existing Structure

**The subject bridge is located in Medford, Massachusetts. It is designated as Bridge No. M-12-002 within the National Bridge Inspection Standards (NBIS) inventory. It is also known as the Cradock Bridge. The existing bridge is a 100-foot long three span composite structure. The existing roadway section (curb to curb width of approximately 50 feet including a 3 foot median and an 8-foot sidewalk on the east side and a 12-foot sidewalk on the west side). The two existing northerly spans consist of stone-arch structures that were constructed prior to 1880. The existing southernmost span consists of a 20-foot reinforced concrete beam structure that was constructed in 1908 to cross over the Cradock Dam and Locks system. Project activities in 1908 also included construction of a two cell dam**

**structure, fitted with tide gates just upstream of the two northerly arch structures. This dam structure was vertically and horizontally configured to serve as a foundation for an upstream extension of bridge superstructure spanning the twin arches.**

**The watershed of the Mystic River is a collection of rivers, streams, lakes and ponds that drain an area of approximately 76 square miles north of Boston. The drainage area of the River at the crossing site is about 49 square miles. The watershed's overall gradient is quite low. It's main channel loses about 80 ft of altitude in 16 mi from the headwaters of the Aberjona River in Reading to the confluence of the Malden and Mystic Rivers between Everett and Somerville, an average bottom gradient of only 5 feet/mile.**

**Prior to the turn of the nineteenth century, the Mystic River was tidally influenced from Boston Harbor to the Lower Mystic Lake in Winchester. In 1908, the Cradock Dam and Lock system was constructed just downstream of the subject bridge along the southern bank of the river. The purpose of this dam system was to eliminate tidal flooding between Medford Square and the Lower Mystic Lake.**

**After the severe flooding that accompanied the storm surge of Hurricane Diane in August, 1955, investigations were begun to determine a more comprehensive means of tidal flood control for the Mystic's lower reaches. A 1962 study by C.E. Maguire and Associates eventually led to the design and construction of the Amelia Earhart Dam and Pumping Station in 1966. This structure is located just downstream of the confluence of the Malden and Mystic Rivers. After the Earhart Dam was complete, the Cradock Dam and Locks were taken offline. In 1978 a portion of this relict dam system was demolished and removed from the crossing location.**

**Presently, daily manually controlled releases at the Earhart Dam during outgoing tides control the rate of flow from the Mystic River watershed into Boston Harbor. The dam is equipped with three large, diesel-powered pumps, which serve to prevent flooding by pumping storm runoff flows into the Harbor's tidal zone before and during significant precipitation events. During storms occurring at high tide, three pumps can pump water across the dam at a rate of approximately 1400 cfs each or a total of 4200 cubic feet per second. As a consequence of the Earhart Dam construction, the hydraulics of the Mystic River through the crossing site are primarily controlled by operations at the dam location.**

**Route 38 (Main Street) is functionally classified as an Urban Minor Arterial. The average daily traffic (ADT) conveyed on Route 38 at the crossing site is estimated to be has been estimated to be 42,000 vehicles per day. Approximately 6% are estimated to be trucks.**

**Land use near the bridge is mostly commercial and residential.**

**The bridge is located within the National Flood Insurance Program (NFIP) Regulatory Floodway established for the Mystic river in Medford (see Reference 6).**

## 2.2 Proposed Action

**The principle project objectives are to upgrade the existing load and service capacity to current highway standards along Route 38 at the crossing site.**

**MassHighway proposes to replace the existing bridge's 20-foot long southerly span, and rehabilitate it's northerly twin stone arch structures. The widened section of bridge deck appended to the western side of the bridge along with it's appurtenant tide gate and trash structures will be removed. The portion of the widened deck upstream of the bridges two arch spans will be replaced with a concrete twin arch system that will have similar dimensions to the twin masonry arches downstream of them. The river and canal walls upstream and downstream of the bridge location along the southern bank of the river will rehabilitated as required. The existing highway profile over the bridge will be maintained on the renovated bridge.**

**For the purposes of this preliminary hydraulic study, the existing composite bridge waterway opening and a probable future three-span opening, unobstructed by upstream appurtenant dam, tide gate, and trash grate structures, were evaluated for hydraulic adequacy and scour safety. Alternative waterway opening configurations will be evaluated at the request of MassHighway's Director of Bridges and Structures.**

## 3.0 Data Collection

### 3.1 Sources and Applications

**Primary information sources for this report are listed below.**

| Reference No. | Title |
|---|---|
| 1 | Camp, Dresser and McKee, Mystic River Comprehensive Hydrology Study, 1981 |
| 2 | Camp, Dresser and McKee , Mystic River Hydrologic and Hydraulic Study Report , 2003 |
| 3 | ENSR, Massachusetts Environmental Policy Act, Draft Environmental Impact Report, EOEA File No. 13046, Aberjona River Flood Mitigation Program, Town of Winchester, MA, 2006. |
| 4 | Federal Highway Administration, Hydraulic Engineering Circular, HEC-18 Evaluating Scour at Bridges, November 2005 |
| 5 | MHD NBIS Bridge Inspection File, Br. No. M-12-002 |
| 6 | Middlesex County Flood Insurance Study (FIS), September 2007 |
| 7 | US Army Corps of Engineers (COE), Hydrologic Engineering Center, HEC-HMS Watershed Modeling System |
| 8 | US Army Corps of Engineers (COE), Hydrologic Engineering Center, HEC-RAS River Analysis System |

**Hydrologic and hydraulic data from References 3 and 6 were used in conjunction with field observations and project survey data to assemble and calibrate a hydraulic model of the bridge crossing site. NBIS inspection reports were used to corroborate field observations. Pertinent data entries are included in the appendices to this report.**

## 4.0 Engineering Methods

### Introduction

**The Federal Emergency Management Agency updated the 10-, 50-, 100- and 500-year flood profiles for the Mystic River and it's main tributaries- the Aberjona River and Alewife Brook- during the preparation of the Middlesex County FIS (see Reference 6). ENSR was contracted to perform the hydrologic and hydraulic analysis required for those flood profile updates. Their analysis included assembly and calibration of a ACOE HEC-HMS hydrologic model and an unsteady state ACOE HEC-RAS hydraulic model (see Reference 8) of the watershed's linkage of streams and rivers. MassHighway's Hydraulic Section obtained an electronic copy of both model data decks directly from ENSR. Extracted portions of that data deck serve as the basis for this project's unsteady state HEC-RAS model of site flood hydraulics.**

### 4.1 Hydrologic Analyses

**Discharge peaks used in this hydraulic study were adopted from ENSR's HEC-HMS hydrologic model of the Mystic River watershed. See Reference 6 for computational procedures. Peak discharges for the 10-, 50-, 100-, and 500-year floods applicable to the project location are presented in Table 1.**

**Table 1. Summary Of Existing Peak Flood Discharges**

| Mystic River | Drainage Area (Square Miles) | 10 - Year (CFS) | 50 – Year (CFS) | 100 – Year (CFS) | 500 – Year (CFS) |
|---|---|---|---|---|---|
| At Route 38 (Main Street) | 49.0 | 1020 | 1928 | 2165 | 3520 |

### 4.2 Hydraulic Analysis

**Backwater analyses were performed for the 10, 50, and 100 year return events in a manner consistent with the standards used to develop water surface profiles under the National Flood Insurance Program. River channel and bridge cross section geometry , as well as the related spatial distribution of surface roughness coefficients (Manning's "n"), used in the project hydraulic model were extracted directly from ENSR's HEC-RAS Mystic River watershed hydraulic model and from the project base plan. Flood discharge data used in the model was extracted from the electronic output of ENSR's previously referenced HEC-HMS model of the Mystic River watershed.**

**Pertinent computational results of 10- thru 500-year unsteady flow flood simulations performed for the existing bridge and a probable replacement bridge type are presented in Tables 2 and 4.**

**Table 2. Summary Of Hydraulic Performance**

| Alternative | Return Period (Yrs) | Flow (Cfs) | U/S Stage (Feet, NAVD) | D/S Stage (Feet, NAVD) | US/DS Stage Elevation Differential (Feet) |
|---|---|---|---|---|---|
| Existing Composite Bridge Opening | 10<br>50<br>100<br>500 | 1020<br>1928<br>2165<br>3520 | 1.15<br>2.3<br>5.7<br>10.6 | 0.4<br>1.55<br>5.0<br>10.0 | 0.75<br>0.75<br>0.7<br>0.6 |
| Proposed Three-Span Bridge Opening | 10<br>50<br>100<br>500 | 1145<br>1996<br>2200<br>3520 | 0.65<br>1.98<br>5.58<br>10.3 | 0.62<br>1.92<br>5.5<br>10.1 | 0.03<br>0.06<br>0.08<br>0.2 |

### 4.3 Scour/Stability Analysis

**Scour potential at the crossing site was analyzed using the guidelines set forth in the FHWA's Hydraulic Engineering Circular No.18 (HEC-18), Evaluating Scour at Bridges. In this evaluation process, long term stream bed elevation changes (aggradation or degradation), contraction (conveyance reduction) scour, and local (vortex induced) scour depths are summed to estimate total potential depth of scour. We assumed that the river bed would not measurable degrade over the next 100 years. The 100-year return frequency discharge peak, 2000 cfs) at the bridge site, was used as the design event in this analysis.**

**Pertinent computational results are presented in Table 3.**

**Table 3. Summary Of Calculated Scour**

| Alternative | Return Period (Yrs) | Flow (Cfs) | Contraction Scour (Feet) | Local Abutment Scour (Feet) | Local Pier Scour (Feet) | Total Abutment Scour (Feet) | Total Pier Scour (Feet) |
|---|---|---|---|---|---|---|---|
| Existing Composite Bridge Opening | 100 | 2165 | 0.0 | 4.8 | 5.1 | 4.8 | 5.1 |
| Proposed Three-Span Bridge Opening | 100 | 2200 | 0.0 | 4.7 | 5.0 | 4.7 | 5.0 |

## 5.0 Conclusions and Recommendations

### 5.1 Conclusions

1. **The existing bridge opening constitutes a minor restriction to Mystic River flood flows. The maximum headwater/tailwater elevation differential calculated at the bridge location within the 10- to 500-year flood simulations was 0.75 feet during the 10- and 50-year return frequency flood events.**

2. **The probable replacement bridge opening assessed in this study offered virtually no restriction to the Mystic's flood flows.**

3. **The calculated scour depths in Table 3 are not supported by notations of observed scour in the NBIS inspection data base for the existing bridge- and should be considered conservative results.**

### 5.2 Recommendations

1. **This preliminary hydraulic study report should be updated as required as the design of the replacement bridge evolves.**

2. **Installation of scour countermeasures around the replacement bridge's abutments and piers may be warranted due to pump capacity improvements proposed at the Amelia Earhart Dam and Pumping Station by the Massachusetts Department of Conservation and Recreation. Coordination with Mass DCR during the preliminary design phase of the replacement bridge is strongly recommended.**

**Table 4**

**Br. No. M-12-002**
**Route 38 (Main Street)/ Mystic River**
**Medford , Massachusetts**

**Provisional Hydraulic Data**

| | |
|---|---|
| **Drainage Area** | **49.0 Square Miles** |
| **Design Discharge** | **1020 Cubic Feet per Second** |
| **Design Frequency** | **10 years** |
| **Design Flood Stage** | **1.15 Feet, NAVD** |
| **Design Velocity** | **2.5 Feet per Second** |

**Basic Flood Data**

| | |
|---|---|
| **100 Year Flood Discharge** | **2165 Cubic Feet per Second** |
| **100 Year Flood Stage** | **5.7 Feet, NAVD** |

**Flood Of Record**

| | |
|---|---|
| **Discharge** | **Unknown** |
| **Stage** | **Unknown** |
| **Date** | **August 1955** |

**History Of Ice Floes: None documented in NBIS database**

**Evidence Of Scour Or Erosion: None documented in NBIS database**

# Appendix J

# Craddock Locks Evaluation Study

**AECOM Environment**
2 Technology Park Drive, Westford, Massachusetts 01886-3140
T 978.589.3000 F 978.589.3100 www.ensr.aecom.com

February 20, 2009

Margaret White
Town Planner
Town of Winchester
Town Hall
71 Mount Vernon Street
Winchester, MA 01890

Re: Evaluation of Craddock Locks, Main Street, Medford
Aberjona River Flood Mitigation Program Element

Dear Ms. White:

Per the scope of our June 27, 2008 contract with the Town we have completed the evaluation of the feasibility of making modifications to Craddock Locks. The structural evaluation, development of options, and cost estimating for this assignment was accomplished by our sub-consultant Weston and Sampson Inc. We have enclosed a copy of their report. AECOM evaluated the hydraulics of the options prepared by Weston & Sampson.

*Project Background*

The objective of the engineering study was to evaluate the feasibility of improving the hydraulic opening of the Main Street Bridge in Medford as an interim measure to the bridge replacement planned by the City and MassHighway. The bridge replacement project is in its early stages and may not occur for a decade. Modeling of flood flows as part of the Aberjona River Flood Mitigation Program showed that the remnants of the Craddock Locks at the Main Street Bridge Impede flood flow. Improving flow through this reach is a key mitigation measure for the improvements proposed in Winchester.

Portions of the lock mechanisms were previously removed by the DCR, but what remains act as an impediment to flow under the bridge. As shown by HEC-RAS modeling during the DEIR and SDEIR phases of the Aberjona River Flood Mitigation Program, enlarging the hydraulic opening under the bridge will help reduce backwater flooding upstream of the locks (including in the Alewife Brook area). What was not known is if the remnants of the lock are at all structural to the bridge. This is what was evaluated in the current study.

*Findings*

Based on our hydraulic evaluation of the options presented in the Weston & Sampson Report, we recommend presenting Option 2 (removal of portions of the remaining concrete panels) to the City of Medford, MassHighway, and the DCR. The proposed solution will expand the hydraulic openings under the bridge but will not result in the need for additional structural measures. Option 2 provides hydraulic improvements similar to the more aggressive Option 3, but Option 3 is likely to be significantly more expensive because of the need to provide additional structural support to the bridge after removing the piers.

AECOM evaluated all three Options using the FEMA HEC-RAS model refined for the SDEIR for the Mystic River watershed. Profile plots of the resulting reduction in the 100-year floodplain upstream and downstream of the structure for Options 2 and 3 are attached, figure 1 and 2 respectively. Option 1 is the baseline condition. Since changes to the Main Street Bridge do not affect predicted peak water levels upstream of Mid Lakes Dam profile plots upstream are not included.

Based on the results of our analysis for the design flood (100 year) AECOM performed additional HEC-RAS model runs for the 2, 5, 10, 50, and 500 year storm events for Option 2. Table 1 presents a comparison of predicted water surface elevations upstream and downstream of the Main Street Bridge for Option 1, the preferred Alternative from the SDEIR (Alternative 7), and Option 2. Table 2 presents a comparison of predicted velocities upstream and downstream of the Main Street Bridge for Option 1, Alternative 7, and Option 2. Option 2 model runs included the upstream improvements of the Winchester Flood Mitigation Program, including in-Town improvements and improvements to Scalley Dam in Woburn. The new configuration of the mid-lakes dam was also taken into account. From a project timing standpoint it is important that Scalley Dam be constructed first as this project helps to offset the effects of increased flow from the in-Town projects at the Craddock Locks. No change in pumping capacity or protocol at the Amelia Earhart Dam was assumed as part of the analysis of Craddock Locks, therefore the recommended improvements of Option 2 are independent of any proposal to increase pumping capacity at the AE Dam.

**Table 1: Option 1 (Baseline Conditions), SDEIR Alternative 7, and Option 2 Water Surface Elevation Comparisons (feet NAVD)**

| | Upstream[1] | | | Downstream[2] | | |
|---|---|---|---|---|---|---|
| **Event** | **Baseline** | **Alternative 7** | **Option 2** | **Baseline** | **Alternative 7** | **Option 2** |
| 2-Year | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 | -0.2 |
| 5-Year | 0.6 | -0.1 | -0.1 | 0.0 | -0.1 | -0.1 |
| 10-Year | 1.1 | 0.3 | 0.3 | 0.3 | 0.2 | 0.2 |
| 50-Year | 2.3 | 1.5 | 1.7 | 1.5 | 1.4 | 1.4 |
| 100-Year | 5.7 | 4.8 | 4.9 | 4.9 | 4.8 | 4.8 |
| 500-Year | 10.0 | 9.9 | 10.0 | 10.0 | 9.9 | 9.9 |

*1. Predicted peak water surface elevations just upstream of Main Street Bridge (Craddock Locks), model cross-section Mystic, 2, 14991.31.*

*2. Predicted peak water surface elevations just downstream of Main Street Bridge (Craddock Locks), model cross-section Mystic, 2, 14715.68.*

**Table 2: Option 1 (Baseline Conditions), SDEIR Alternative 7, and Option 2 Velocity Comparisons (fps)**

| | Upstream[1] | | | Downstream[2] | | |
|---|---|---|---|---|---|---|
| **Event** | **Baseline** | **Alternative 7** | **Option 2** | **Baseline** | **Alternative 7** | **Option 2** |
| 2-Year | 0.7 | 0.6 | 0.6 | 0.9 | 0.6 | 0.6 |
| 5-Year | 0.9 | 0.9 | 0.9 | 1.1 | 0.9 | 0.9 |
| 10-Year | 1.2 | 1.2 | 1.2 | 1.4 | 1.3 | 1.3 |
| 50-Year | 1.8 | 2.0 | 2.0 | 2.2 | 2.1 | 2.1 |
| 100-Year | 1.4 | 1.5 | 1.5 | 1.6 | 1.6 | 1.6 |
| 500-Year | 1.6 | 1.6 | 1.6 | 1.6 | 1.6 | 1.6 |

1. *Predicted model velocities taken just upstream of Main Street Bridge (Craddock Locks), model cross-section Mystic, 2, 14991.31.*
2. *Predicted model velocities taken just downstream of Main Street Bridge (Craddock locks), model cross-section Mystic, 2, 14715.68.*
3. *The velocities reported are associated with the velocity at the peak stage from the model run; the value is not necessarily the peak velocity.*

The analyses show that implementation of Option 2 is not predicted to have a negative impact on either water surface profiles or on velocities upstream or downstream of the Main Street Bridge.

*Permitting Considerations*

Work under the Main Street Bridge on the remnants of Craddock Locks will require filing a Notice of Intent under the Massachusetts Wetlands Protection Act with the City of Medford Conservation Commission. As currently envisioned, the project does not appear to require permitting by the MADEP and USACE under the Clean Water Act (Sections 401 and 404, respectively) but this could change based on a constructability review during the design stage of the project. Amending or applying for a Waterways (MGL Chapter 91) License may also be required depending on the final design. A Request for Determination of Applicability (under MGL Chapter 91, the Public Waterfront Act) to the MADEP is recommended. Permitting costs are estimated to be approximately $7500 (higher if 401 and 404 permits are required).

Please contact me with any questions about this report. We are prepared to meet with you, DCR, MassHighway, and the City of Medford at your earliest convenience. Please let us know when you have arranged for this meeting through Mayor McGlynn's office.

Thank you again for asking AECOM to assist you with this project.

Sincerely,

Michael J. Toohill, PWS, CE
Project Manager

Jacob San Antonio, PE, CFM
Senior Engineer

Cc: Mark Twogood, Assistant Town Manager, Winchester
David Elmer, Project Manager, Weston & Sampson

**Figure 1: 100-Year Flood Profile, Option 2 vs. Option 1 (Baseline Conditions)**

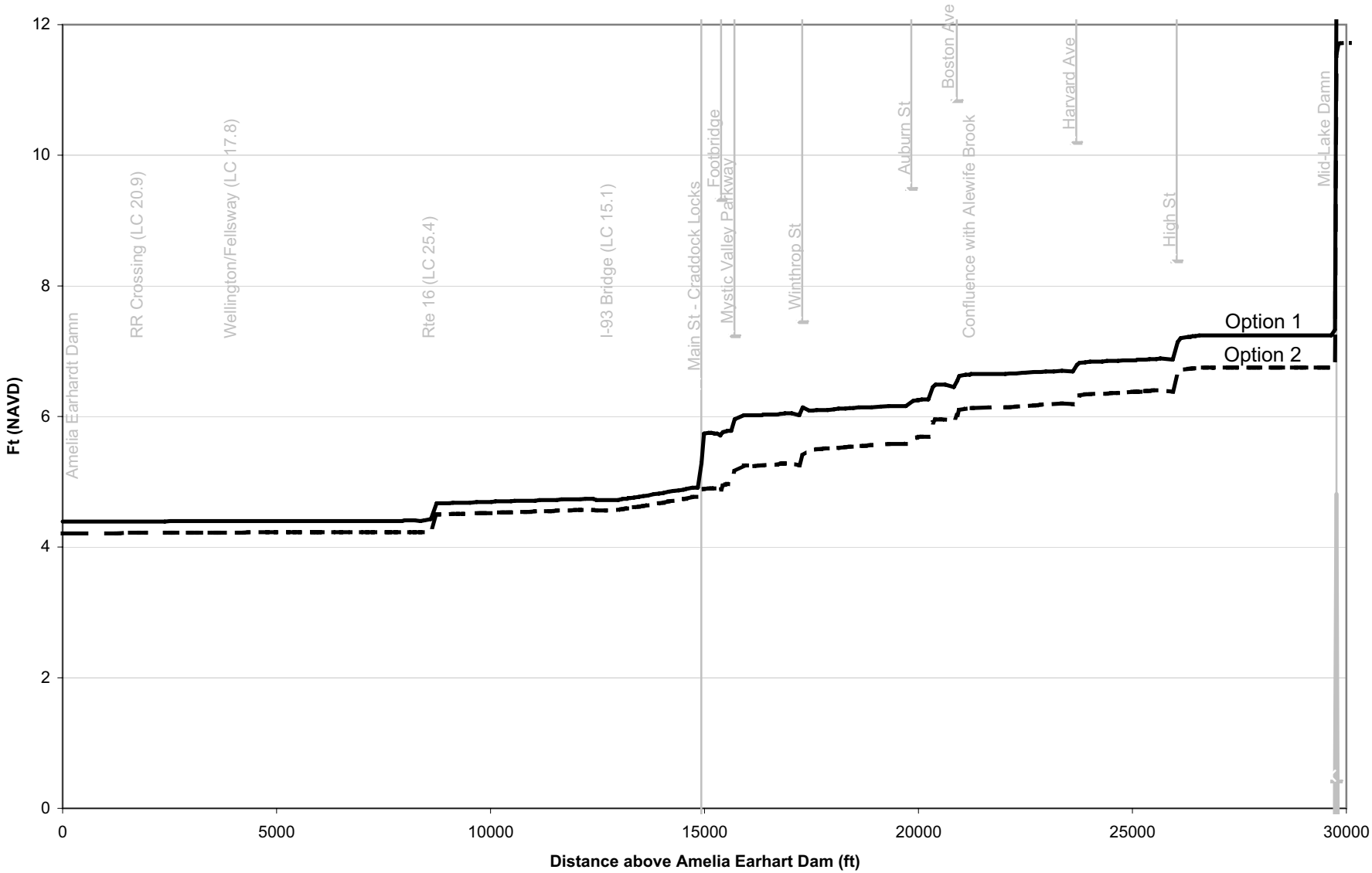



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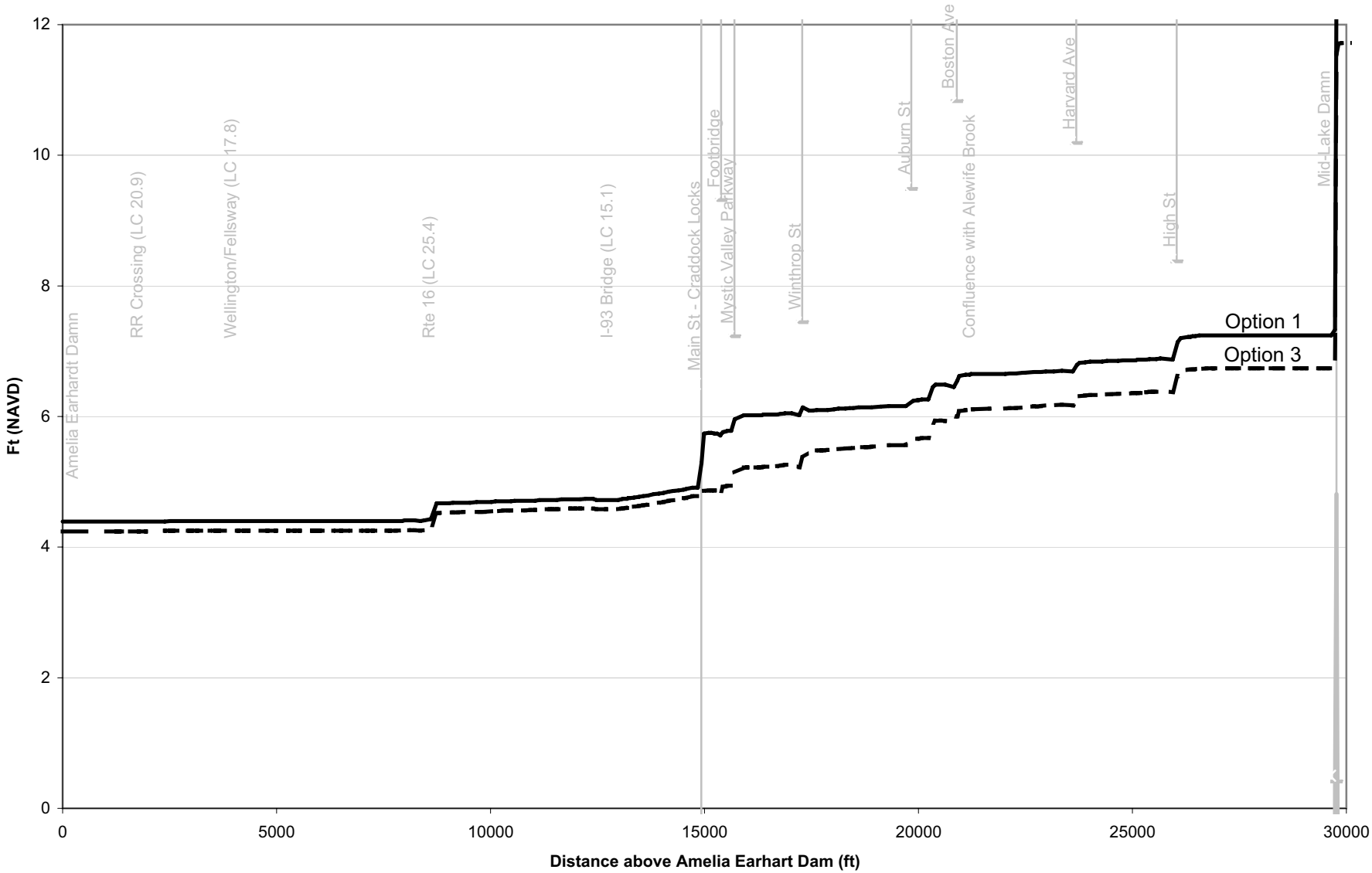
**Figure 2: 100-Year Flood Profile, Option 3 vs. Option 1 (Baseline Conditions)**

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**ENSR/Winchester, Massachusetts**
**WSE Job No. 2080517.A**

February 17, 2009

Mr. Michael J. Toohill
Project Manager
ENSR Corporation
Two Technology Drive
Westford, Massachusetts 01866

Re: Aberjona River Flood Mitigation Program
Evaluation of Removal of Flow Restrictions at Cradock Locks (Main Street Bridge) in Medford, Massachusetts

Dear Mr. Toohill:

Weston & Sampson appreciates the opportunity to provide you with this report summarizing the findings of our visual inspection and evaluation related to the potential removal of flow restrictions beneath the Cradock (Main Street) Bridge in Medford, Massachusetts (also known as the Cradock Locks).

Background

As a result of computer modeling conducted by ENSR Corporation (ENSR) during preparation of the Draft Environmental Impact Report (DEIR) and the Supplemental Draft Environmental Impact Report (SDEIR) for the Aberjona River Flood Mitigation Program, existing structural components under the Main Street Bridge create an impediment to flood flows. The purpose of the visual inspection and evaluation conducted by Weston & Sampson was to ascertain the feasibility of removal of the identified flow obstructions without impacting the overall structural integrity of the bridge and appurtenances.

As part of the evaluation Weston & Sampson reviewed construction plans obtained from ENSR and the Massachusetts Department of Conservation and Recreation (DCR), formerly the Metropolitan District Commission (MDC), consisting of the following:

- "Construction Plans for Widening Cradock Bridge", MDC Parks Division, dated August 11, 1921.
- "Demolition of the Cradock Dam, Medford", MDC Engineering Division, dated January 1979.

Based upon review of the above plans a majority of the hydraulic structures and components of the old Cradock Lock, including concrete weirs, sluice gates, tide gates, and stop logs were removed under the 1979 construction contract. However, there are five concrete wall panels still in place which extend vertically from underneath the bridge deck down towards the Mystic River water surface. It is these concrete wall panels that pose the flow restriction identified by ENSR in the DEIR and SDEIR.

Weston & Sampson conducted a visual inspection of the bridge structure on October 30, 2008, to assess possible improvements that may decrease restrictions to the waterway caused by the old abandoned lock system beneath the bridge. In addition, Weston & Sampson discussed the status of the planned future bridge improvements with Mr. Brian Chapman, P.E., Project Manager with the Massachusetts Highway Department (MHD or MassHighway).

Conditions Observed During Visual Inspection

The visual observations were conducted by Weston & Sampson using a watercraft to view the bridge superstructure as well as the existing concrete panels and the concrete pier abutments. The existing concrete wall panels are positioned closer to the upstream edge of the bridge and reside in the bays previously used for the tide gates, sluice gates, stop logs and appurtenances (refer to Figures 1 and 2). Referring to Figure 2 from left to right are two bays each approximately 12-feet-wide that formerly contained tide gates and appurtenances on the upstream side and stop logs on the downstream side; one bay approximately 10-feet-wide that formerly contained a sluice gate on the upstream side and stop logs on the downstream side; and two bays each approximately 12-feet-wide formerly containing tide gates on the upstream side and stop logs on the downstream side. The five bays are shown on a reduced copy of the MDC August 1921 plans as Figure 3.

The existing concrete wall panels are each roughly 2-feet-thick and extend down from the top of the piers approximately 7 to 8 feet (refer to Figures 4 through 7). The clearance between the water and bottom of the walls at the time of inspection (October 30, 2008) was approximately 2 to 3 feet. Unfortunately, the MDC plans do not show the panels in section view; therefore we are unable to confirm the elevation of the bottom of the wall panels. The concrete pier abutments appear to have been part of a larger locks system at one time and the remaining portions of these piers have been patched. It is evident that the remaining portions of these concrete pier abutments on the upstream side of the bridge are supporting those beams that were added during the last widening of the bridge.

It appears that portions of the concrete wall panels may be removed without critical impact to the structural stability or integrity of the existing bridge piers and/or overlying

bridge and sidewalk features (refer to Figures 4 through 8). Further, it does not appear that temporary support for the bridge and/or sidewalk would be needed to remove portions of the walls. This observation would need to be confirmed during the design of the improvements. It appears that the four existing galvanized steel boat racks on the upstream side of the bridge could also be removed without affecting the structural integrity of the bridge (refer to Figure 9), however this would need to be confirmed with other parties (DCR, MHD, City of Medford) as the racks may be present for safety purposes and/or operational considerations (i.e. trash racks) of the Amelia Earhart Dam located further downstream of the Main Street Bridge.

Options for Improvement

Four options were reviewed to modify the existing configuration of Cradock Locks. It is important to note that the structural evaluation for each of these options does not take into consideration upstream and downstream hydraulic impacts, which would need to be looked at separately. The options are as follows:

1. Do nothing.
2. Remove portions of one or more of the existing concrete wall panels.
3. Remove selected piers and one or more of the existing concrete wall panels.
4. Wait for the bridge improvements planned by MassHighway.

*Option 1 – Do Nothing*

This option would leave the existing Cradock Bridge in its present configuration. There is no construction cost associated with this option.

*Option 2 – Remove Portions of One or More Existing Concrete Panels*

This option consists of selective removal of portions of one or more of the existing concrete wall panels for the purpose of reducing the restriction to flow. It should be noted that per our discussions with MassHighway (Mr. Chapman), it was indicated that MassHighway recommends that no increase in water flow beneath the Cradock Bridge occur until a fourth pump is added at the Amelia Earhart Dam. However, for the purpose of this option, we have considered removal of as many as five feet from the lower portion of the walls to provide more clearance between the bottom of the old lock wall structure and the water level (refer to Figures 4 through 8). The selection of five feet is arbitrary and additional (slight) removals could be considered if hydraulic evaluations by ENSR indicate they are needed. The table below provides a summary of the width of each bay and the maximum wall area that could be removed to reduce flow restrictions.

| Bay | Bay Width (feet) | Max. Height of Curtain Wall Removal (feet) | Max. Area of Curtain Wall Removal (sq. ft.) |
|---|---|---|---|
| #1 | 12 | 5 | 60 |
| #2 | 12 | 5 | 60 |
| #3 | 10 | 5 | 50 |
| #4 | 12 | 5 | 60 |
| #5 | 12 | 5 | 60 |
| | | **Total:** | **290** |

We envision that the removal of these concrete sections can be accomplished using crew(s) working from a barge or boat type arrangement; coffer damming of the river does not appear to be necessary. The concrete wall sections could be removed by saw cutting and the Contractor would need to employ suitable means to capture the concrete debris and control dust so that impacts to the river and other resources do not occur. Following removal, the concrete debris would be transported off-site and disposed of in accordance with applicable regulations.

For budget-level estimating purposes of this conceptual option, our opinion of probable construction cost is $395,000. This cost includes removal of approximately five feet of concrete vertically from all five of the wall bays (estimated as approximately 25 cubic yards of concrete); Contractor labor and equipment (such as dump truck); allocation for Contractor environmental controls; allocation for Contractor access to work area (barge or boat); allocation for transport and disposal of concrete debris; Contractor overhead and profit (assumed as 25 percent of estimated construction cost); and allocations for engineering design and permitting, construction observation/administration and contingency (assumed as 20 percent, 10 percent and 30 percent of estimated construction cost respectively). We anticipate that the work could be completed in 2 to 4 weeks following the award of a construction contract. This budget does not include the cost of providing temporary flow restrictions as may be requested by MHD.

*Option 3 – Remove Selected Piers and Portions of One or More Existing Concrete Panels*

Option 3 would include removing not only selected portion of the walls as outlined in Option 2, but also piers #1 and #3 (refer to Figure 3). Although removal of these piers would provide a slightly less impeded flow path for water passing under the bridge, the additional area generated is not substantial when compared to the area associated with the curtain wall removal. Although the exact height of the piers is unknown, assuming a height of 10-feet, at two-feet wide, only a 20 sq.ft. area for additional flow would be created with the removal of each pier. This additional 40 sq. ft. area is insignificant when compared with the area generated by removal of portions of the walls, which is estimated at 290 sq.ft.

In addition to the minimal flow area created, there are considerable construction and cost implications associated with this option. In order to remove the piers, additional support will need to be added to the bridge structure in the form of steal beams. The existing steel beam currently supported by these piers is not sufficient for carrying the bridge loads if the piers are removed. Option 3 will also require coffer damming during construction to dewater the area around the piers. Dewatering will need to be done in two phases, as the entire river may not be blocked off at any point during construction. It is our recommendation that piers #2 and #4 (refer to Figure 3) stay in place as they will keep the spans of the new beams to optimal length. During removal of piers #1 and #3, the contractor will need to employ suitable means to capture the concrete debris and control dust so that impacts to the river and other resources do not occur. The concrete debris will need to be transported off-site and disposed of in accordance with applicable regulations. The construction cost for this option will be significant due to the required coffer damming and the additional structural support required for the bridge.

MassHighway recommends that no increase in water flow beneath the Cradock Bridge occur until a fourth pump can be installed at the Amelia Earhart Dam. Given its complexity, this option was not given additional consideration. As such, a construction cost estimate was not prepared.

*Option 4 – Wait for Bridge Improvements Planned by MassHighway*

The fourth option that should be considered is to wait for the bridge renovations that will be performed by MassHighway. Our discussions with MassHighway Project Manager Mr. Brian Chapman have revealed that the renovations are currently listed on the 2015 Statewide Transportation Improvement Program (STIP), with a planned advertisement date in October 2014. Mr. Chapman indicated that there is a possibility that the advertisement date could be moved up (possibly to October 2011), but that is uncertain since the project is very early in the design process.

Presently, the bridge replacement/renovations that MassHighway is planning are as follows. For the DCR single span (which is the southern span) section of the bridge, MassHighway plans to replace the superstructure with new bridge seats, deck, sidewalks and railings. They are also planning on repairing the south abutment wall and the exterior surface of the northern side pier. For the City of Medford owned arch spans, MassHighway plans on removing the widened (upstream) section including the trash racks, concrete substructure and superstructure. They are also planning on removing the dam structure and replacing it with a temporary curtain beam system until a fourth pump is installed at the Amelia Earhart Dam. According to MassHighway this is required in order to maintain the current flow constriction beneath the bridge. After the fourth pump

is installed at the Amelia Earhart Dam, the widened section is to be replaced with two concrete arch sections that will match the original downstream arch sections.

Conclusion

Option 2 appears to be the most technically-feasible means of removing hydraulic restrictions to the Mystic River identified at the Cradock (Main Street) Bridge. However, this option does not take into consideration upstream and downstream hydraulic impacts. The final selection of an option to proceed should be coordinated with DCR and MHD, as well as other concerned parties.

Please feel free to contact Kent Nichols, P.E. at 978-532-1900, ext. 2408 or me at ext. 2280, if we can provide any additional information or be of further assistance.

Very truly yours,

WESTON & SAMPSON ENGINEERS, INC.

David M. Elmer, P.E.
Team Leader

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# Figures

Figure 1 – Downstream Elevation View

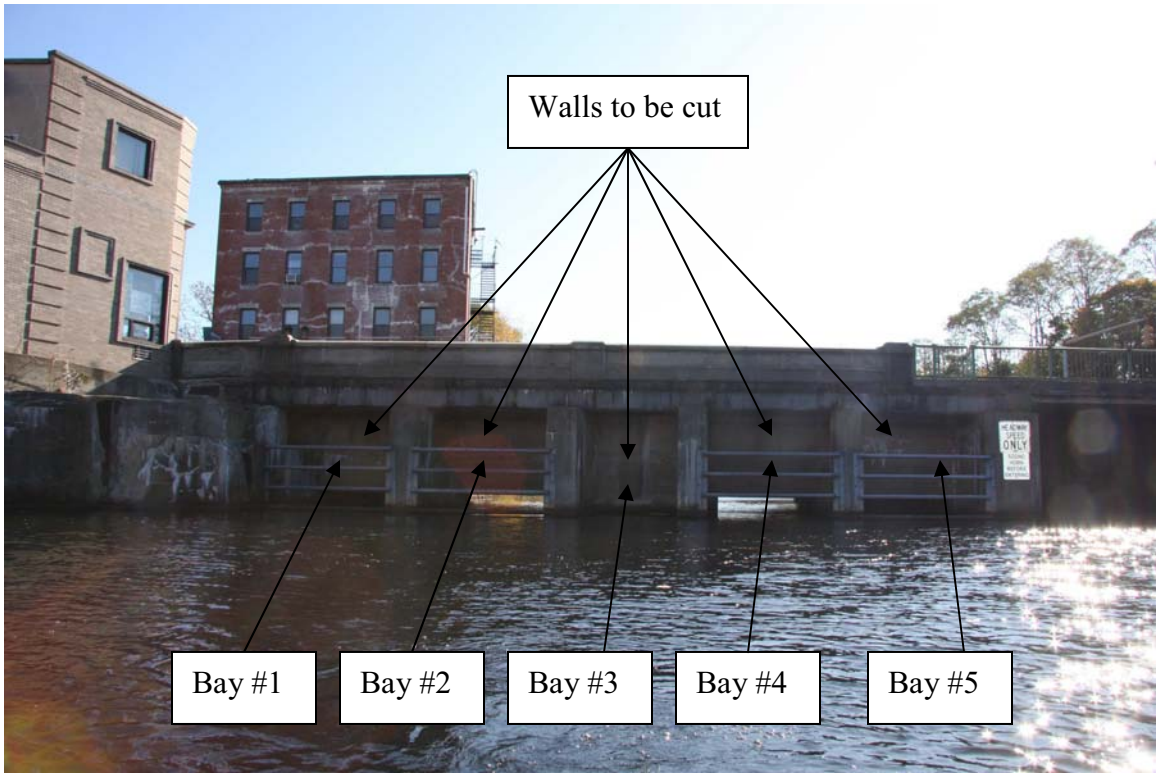


Figure 2 – Upstream Elevation View

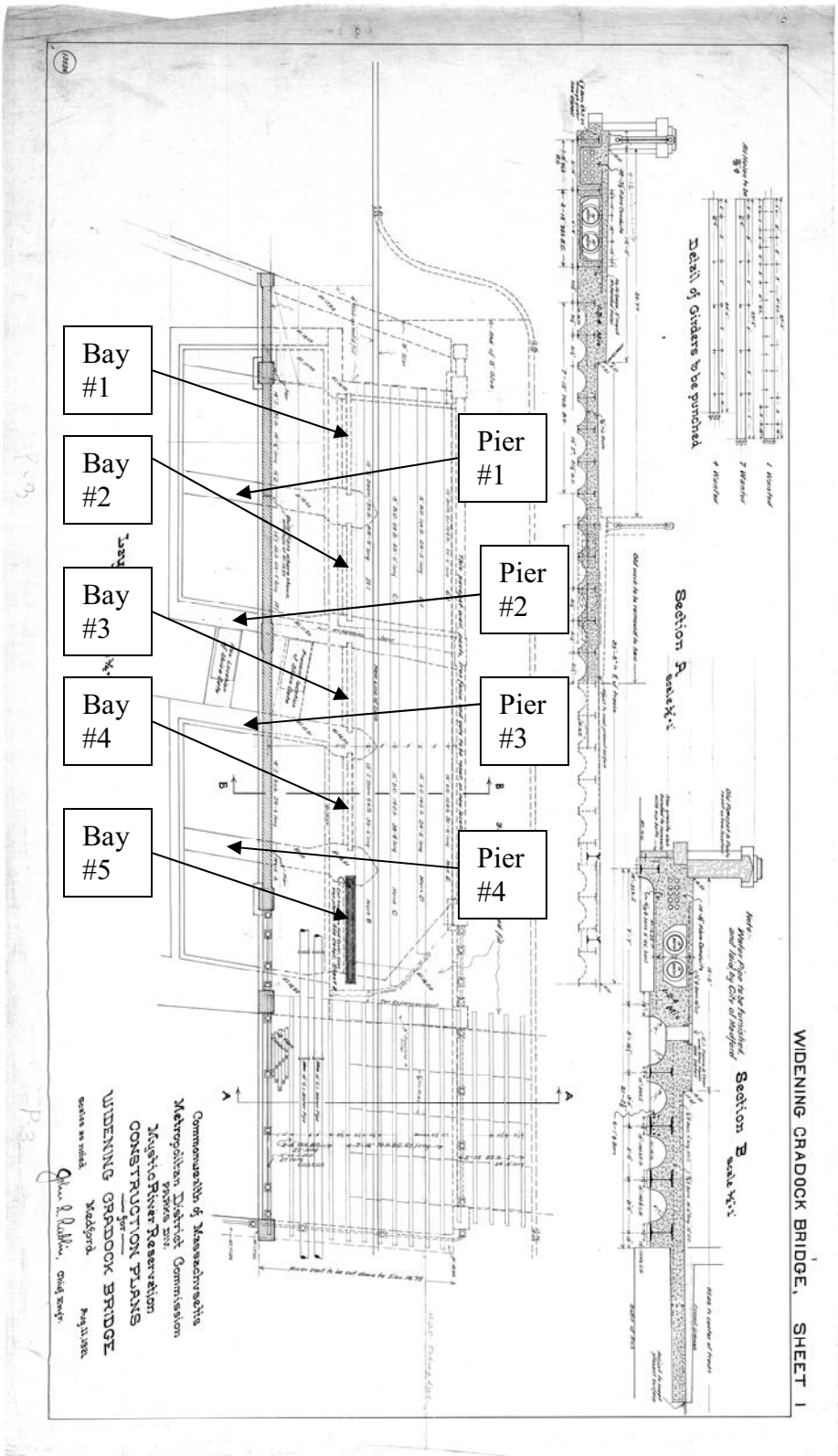


Figure 3 - 1921 Construction Plans for Widening of Cradock Bridge.

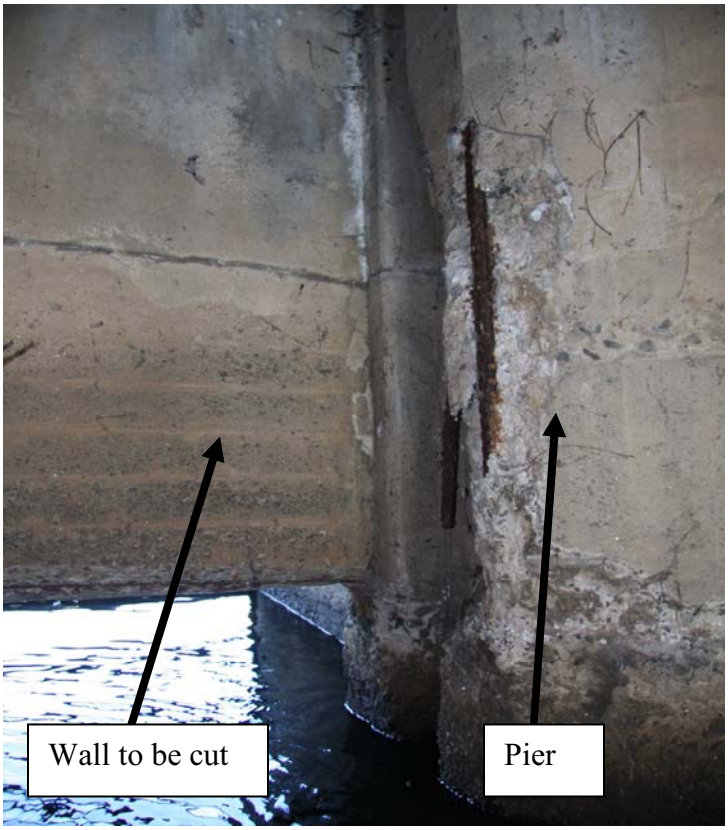

Figure 4 – Walls Perpendicular to Piers

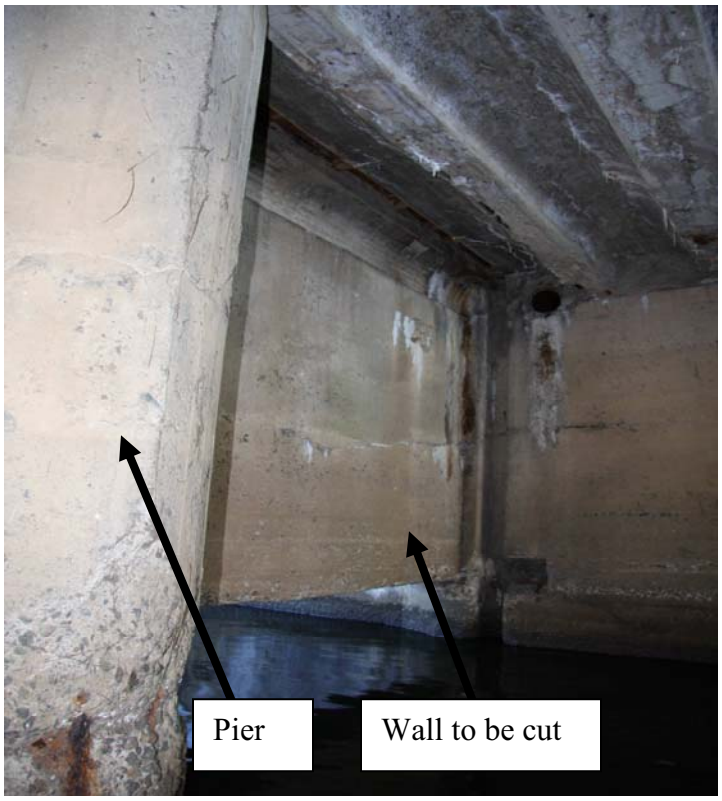

Figure 5 – Walls Perpendicular to Piers

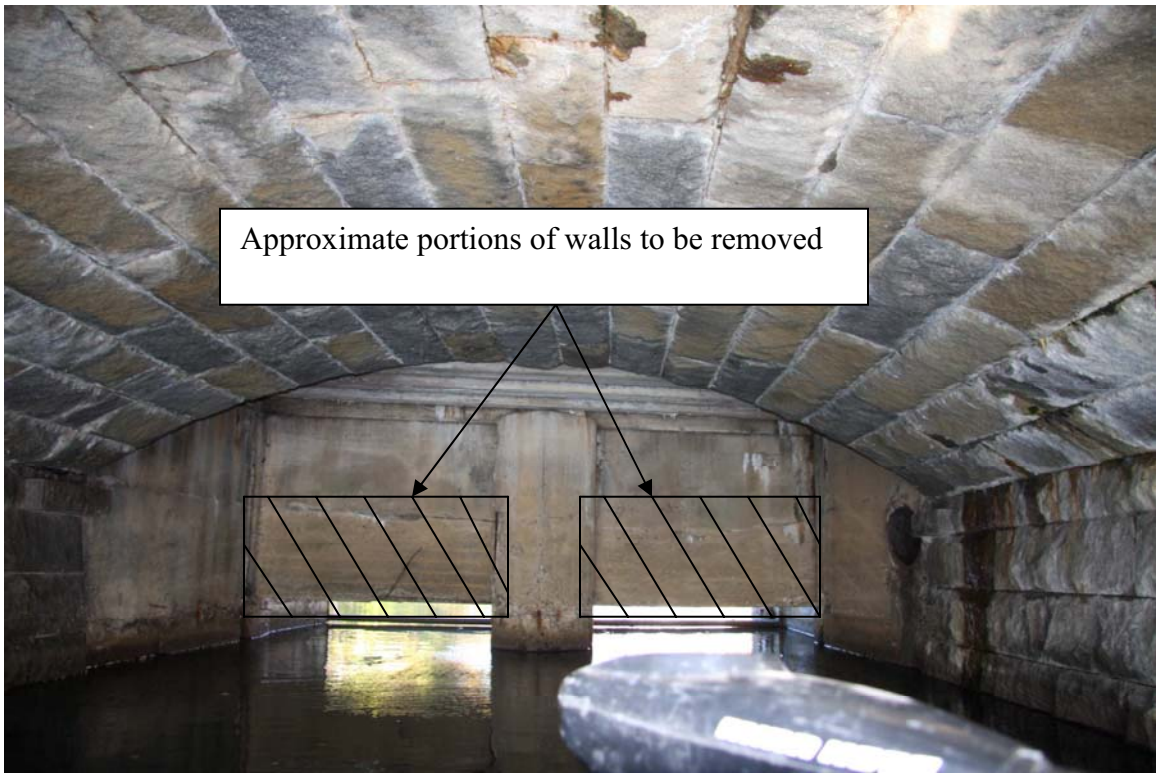


Figure 6 – Elevation View of Walls

Figure 7 – Elevation View of Walls

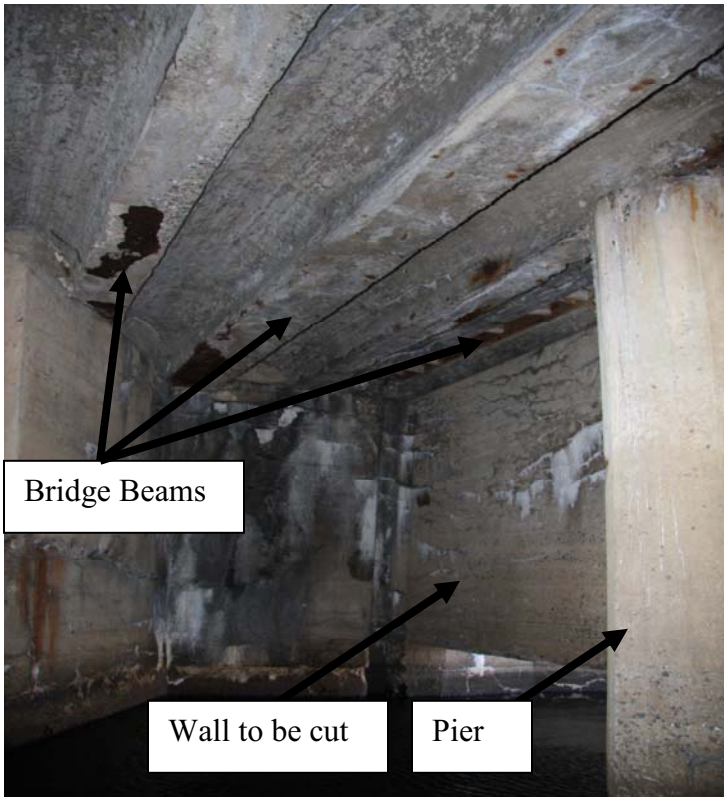

Figure 8 – Pier Supporting Bridge Beam Beneath Sidewalk

Figure 9 – Close-up of Boat Racks on Upstream Side