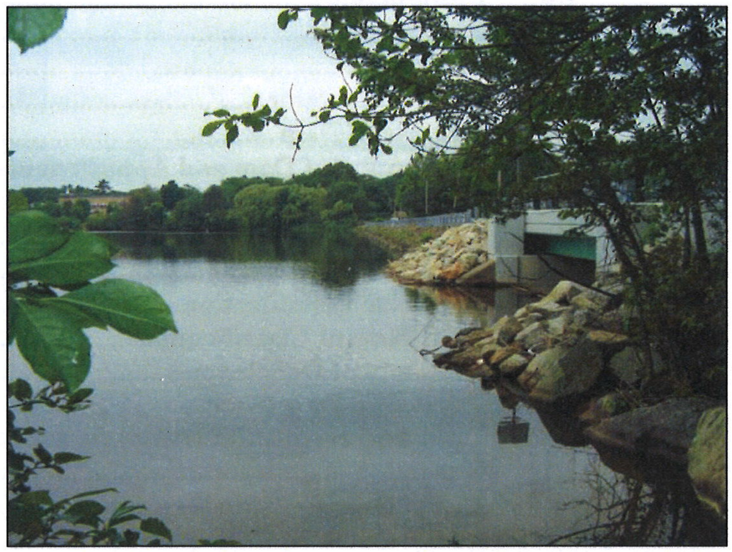
Draft Final Report
4/25/11

Willett Pond Dam
Phase II Investigation

Neponset River Land Holding Association

Norwood, MA

April 25, 2011



Dam Name: Willett Pond Dam

State Dam ID#: 6-11-220-02

NID#: MA00169

Owner: Neponset River Land Holding Association

Owner Type: Private

Town: Norwood/Walpole

Consultant: Fuss & O'Neill, Inc.

Date of Completion: July 28, 2010

FUSS & O'NEILL
Disciplines to Deliver

78 Interstate Drive
West Springfield, MA 01089

Project No. 20051323.A20

FUSS & O'NEILL

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1 Description of Project

1.1 Introduction

Fuss & O'Neill, Inc. has been retained by the Neponset River Land Holding Association (NRLHA) to perform a Phase II assessment of the dam at Willett Pond in Norwood/ Walpole, Massachusetts. The State Inventory ID number is 6-11-220-02 and the National Inventory of Dams ID number is MA00169.

1.2 Purpose

The purpose of this investigation is to assess the current condition of the dam, the options available for its repair, the costs for those options and the approach to its repair, as required in the Massachusetts Department of Recreation and Conservation Office of Dam Safety (ODS) Certificate of Non-Compliance and Dam Safety Order dated November 13, 2007, amended in a letter dated June 24, 2008.

1.3 Location

Town:	Norwood and Walpole
County:	Norfolk
Street Address/Nearest Intersection:	Brook Street, Norwood and Bullard Street Walpole
Nearest Intersection:	Nichols Street, Norwood

1.4 Owner/Operator

Owner:	Neponset River Land Holding Association
Operator:	Neponset River Watershed Association 2173 Washington Street Canton, MA 02021

1.5 Purpose of Dam

Current:	Conservation and Recreation
Historic:	Constructed circa 1913 to supply water to downstream mills

1.6 Description of Dam and Appurtenances

1.6.1 Dam

Dam Length:	900 feet

Dam Structural Height:	25 feet
Dam Hydraulic Height:	22 feet
Type of Dam:	Earth Embankment with concrete core wall
Type of Spillway:	Concrete Broad Crested Weir
Low-Level Outlet:	Dual 20-inch cast iron culverts with operable knife gates

1.6.2 Dike

Dike Length:	1,900 feet
Dike Structural Height:	14 feet
Dike Hydraulic Height:	11 feet
Type of Dike:	Earth Embankment with concrete core wall
Type of Spillway:	NA
Low-Level Outlet:	NA

1.7 Identification

Name:	Willett Pond Dam
State Inventory ID #:	6-11-220-02
National Inventory of Dams ID #:	MA00169

1.8 DCR Size Classification

Per Current Regulations (302 CMR 10.06(2)):Large
Latest DCR Classification (1998 Department of Environmental Management Inspection): Large

1.9 DCR Hazard Classification

Per Current Regulations (302 CMR 10.06(2)):High Hazard (Class I)
Latest DCR Classification (1998 Department of Environmental Management Inspection): High Hazard (Class I)

1.10 Summary of Deficiencies

Visual inspections of Willett Pond Dam were completed in 1979, 1998, 2005, 2007, 2008, and 2009 concurrent with this report. During each of these inspections, Willett Pond Dam was found to be in poor condition. The primary deficiencies identified during these inspections include:

- Brush and vegetation on upstream and down stream dam embankments
- Rodent holes in dam
- Wet areas at down stream toe of dam
- Water in the gate house

- Scoured spillway training walls
- Deteriorated area in down stream channel
- Parshall flume deteriorated
- Gatehouse roof deteriorated
- Small trees on dike
- Eroded locations on dike
- Sand beach on dike
- Patches of dead grass on dike
- Large trees at toe of dike
- A few wet areas at the down stream toe of the dike
- Possible instability based on one leaning power pole
- Iron staining/seepage in spillway concrete outlet apron
- Inadequate spillway capacity

Most of these deficiencies have been corrected. Some were corrected during the bridge replacement; some were corrected by NRLHA. Still others are being corrected by NRLHA currently. Some of the past deficiencies are no longer evident, while some were never deficiencies in the first place, but were listed due to inadequate inspection or misunderstanding of the observations. It appears that much progress has been made on improving the dam and dike. The most significant remaining deficiency is the spillway capacity, which is discussed in detail below.

At this time, the only deficiency that causes this dam and dike to be in Poor condition is the inadequacy of the spillway to accommodate the SDF. Please refer to the recent Visual Inspection performed by Fuss & Neill dated June 11, 2009 for a more detailed description of inspection findings and the current condition of the dam.

2 Hydrologic and Hydraulic (H&H) Assessments

2.1 General

A hydrologic and hydraulic analysis has been performed for the contributing drainage basin to Willett Pond Dam for the Spillway Design Flood (SDF). The SDF for an existing dam with a size classification of "Large" and a hazard classification of "High Hazard (Class I)" is the half of the probable maximum flood (½ PMF). The analysis was performed using the "Hydrologic Modeling System" (HEC-HMS) software, Version 3.4, released by the U.S. Army Corps of Engineers (USACE)[1]. Development of the hydrologic models is discussed in greater detail in the following sections.

2.1.1 Drainage Basin Description

The total drainage area of Willett Pond Dam is approximately 4.8 sq. mi. In general, development in the watershed is characterized by mixed residential and rural areas. The greatest amount of development is in the immediate vicinity of Willett Pond. There are a variety of small impoundments and undersized stream culverts in the watershed; however, none of these, aside from Willett Pond itself, provide significant attenuation of flood flows. The primary inflows to the impoundment are from Bubbling Brook and Mill Brook.

2.1.2 Outlet Hydraulics

The primary outlet structure is the spillway, which is located near the right abutment and is 28.3 feet long with an invert elevation of 137.75 feet (NAVD 88). Three bays of timber stop logs, with a crest elevation of 139.91 feet, are located immediately upstream of the bridge. The stop logs remain installed under normal operating conditions. Therefore, the normal water surface elevation (WSE) is approximately 139.9 feet.

Flow is trained through the spillway by vertical concrete wing walls and training walls. The bridge opening is approximately 5 feet high and 28.3 feet across. The channel through the bridge opening is lined with concrete. Downstream of the bridge, the discharge is conveyed to the downstream portion of Bubbling Brook through a 200-foot long channel that is lined primarily with stone; in some areas concrete paving is present.

A low-level outlet is located near the mid-point of the dam embankment. It consists of two 20-inch cast-iron conduits, both with a downstream invert elevation of 119.64 feet. The outlets are manually operated by knife gates.

The discharge capacity of the spillway was calculated in accordance with guidance provided in "Design of Small Dams", issued by the U.S. Bureau of Reclamation[2]. The discharge calculations for each of the outlets are summarized in Appendix A. This includes example calculations and elevation vs. discharge curves.

2.1.3 Willett Pond Impoundment Volume

The storage volume in Willett Pond was calculated by first defining an elevation-area curve and then applying that data to calculate the volume using the conic sections method. USGS topographic mapping was used to define the elevation-area curve above the normal WSE. Bathymetric mapping is not available for water elevations above normal pool; therefore, the elevation-area curve above the normal WSE was estimated based upon the slope of the surrounding terrain and the elevation at the toe of the downstream slope of the dam.

2.2 ½ Probable Maximum Flood

The ½ PMF is derived by reducing the ordinates of the Probable Maximum Flood (PMF) hydrograph 50%. The PMF represents the most severe runoff that would result from the occurrence of the probable maximum precipitation coincident with saturated soil conditions.

2.2.1 Probable Maximum Precipitation

The rainfall distribution for the probable maximum precipitation was obtained from Hydrometeorological Report No. 51 and No. 52, issued by the National Oceanic &

Atmospheric Administration[3]. Although the likelihood that a storm producing the probable maximum precipitation would be oriented directly over the contributing drainage area is extremely remote, this assumption was conservatively applied to the analysis. As such, the spatial distribution of rainfall was assumed to be centered over the Willett Pond watershed. The rainfall distribution over time is summarized in Table 1.

Table 1: Probable Maximum Precipitation

Time (hours)	Incremental Precipitation (inches)	Time (hours)	Cumulative Precipitation (inches)
12	2.25	42	2.25
24	2.25	48	5.5
36	2.50	54	7.0
42	4.50	60	11.5
48	24.5	66	36.0
72	1.00	72	37.0

2.2.2 Runoff Production & Hydrograph Routing

Losses from infiltration, surface detention, and transpiration were assumed to be at a constant rate of 0.1 inches per hour. This loss rate is consistent with minimum losses determined in previous studies for New England[4]. The results of the analyses are discussed in detail in Section 2.3.

To account for the effect of flood storage in Willett Pond the impoundment was modeled assuming level pool routing.

2.2.3 Existing Conditions

Two scenarios were run for the existing condition: weir boards installed to elevation 139.91 feet; with no weir boards installed (crest elevation 137.75 feet). The starting WSE in each scenario was assumed to be at the elevation of the respective spillway crest. Additionally, discharge through the low-level outlets was not considered. The results of the analysis for each flood event at Willett Pond are summarized in Table 2.

Table 2: Existing Conditions

Flood Event	Initial WSE (feet)	Max. WSE (feet)	Peak Inflow (cfs)	Peak Outflow (cfs)	Freeboard* (feet)
Weir Boards Installed	139.91	144.02	4,883	4,830	-1.82
No Weir Boards Installed	137.75	143.95	4,883	4,811	-1.75

* Measured from lowest embankment elevation = 142.2 feet

2.3 Spillway Alternatives Assessment

At a water surface elevation (WSE) of 142.2 feet, the maximum elevation before overtopping of the dike embankment occurs; the existing spillway has capacity to convey 250 cfs with weir boards installed and 650 cfs with the weir boards removed. The low level outlet, which was omitted from the calculations above, has the capacity to release approximately 70 additional CFS.

As part of this alternatives assessment a variety of spillway configurations were considered in combination with reducing the normal WSE and/or raising of low portions of the dike embankment crest. However, it was found that increasing the capacity of the spillway alone will not be sufficient to safety convey the Spillway Design Flood (SDF) downstream. The existing bridge opening area is too small to convey the SDF without causing overtopping. Due to the close proximity of the spillway to the bridge, backwater will inundate the spillway opening; the bridge itself is the controlling hydraulic section. Therefore, the bridge opening must be increased to realize the benefit of providing greater discharge capacity of the spillway.

Options for increasing the bridge opening cross sectional area include maintaining the current opening width and reducing the invert elevation (deepening the spillway), adding more spans adjacent to the existing bridge to increase the total width of the bridge opening, or a combination of both. Any of these approaches poses challenges. The downstream channel, which conveys the discharge to Mill Brook, is relatively narrow, having a width of approximately 23.8 feet. Furthermore, in its existing configuration it is too shallow to safely pass the SDF. Flood waters would overtop the left side of the channel and flow down the abutment contact of the dam creating the likelihood of severe erosive damage to the embankment.

Due to the considerations listed above, providing the discharge capacity to safely convey the SDF downstream must include a combination of increasing the capacity of the spillway, the bridge opening, and channel downstream of the bridge.

For all the alternatives considered, the bridge opening cross sectional area was assumed to be increased by maintaining the existing bridge span of 28.3 feet and lowering the invert elevation of the channel through it. The reason the option of widening the spillway was discarded was that the cost of expanding the bridge length is substantial, and the water that flows through a widened spillway must still be channeled back to the downstream channel somehow, requiring additional structures and disruption of the earth embankment and wetland areas on the downstream side. Furthermore, the width of the channel downstream of the bridge was assumed to be unchanged, so the capacity of the proposed channel was increased by making it deeper. It should be noted that other potential configurations, such as widening of the existing bridge or the downstream channel, could also be utilized.

Four alternatives are described in detail below. In addition, a number of other alternatives were considered but found to be infeasible and were therefore not examined in detail. These discounted alternatives are also discussed very briefly below.

2.3.1 Alternative 1

For this alternative the crest elevation of the proposed spillway was assumed to be 139.91 feet; therefore, the existing normal WSE would remain unchanged from the present condition in this proposed condition. The required spillway length is 140 feet assuming an ogee crest is used, which is an efficient crest design. The proposed spillway is horseshoe shaped to provide a smooth transition from the spillway into the bridge opening while providing the additional length required to accommodate the SDF. A conceptual sketch and profile of Alternative 1 is attached as Figures 3A and 3B.

The cross sectional area of the proposed bridge opening is increased by reducing the existing invert elevation of 137.75 feet to 131.0 feet. Likewise, the invert of the downstream channel also needs to be reduced at the upstream end (immediately downstream of the bridge) from 137.0 feet to 129.0 feet. The downstream invert elevation (at the confluence with the downstream portion of Bubbling Brook) will remain unchanged at elevation 125.0 feet.

To obtain the required bridge opening area, the maximum WSE for the SDF was assumed to be 144.0 feet. As such, the crest of the dike embankment would have to be raised to prevent overtopping of the dike during the SDF. This could be accomplished by placing earth fill or constructing a concrete parapet wall. The lowest point on the dike is approximately 142.2 feet, so the crest would have to be raised a maximum of 1.8 feet to prevent overtopping.

2.3.2 Alternative 2

For this alternative, the proposed modifications to the spillway and downstream structures provide sufficient discharge capacity such that the maximum allowable WSE (142.0 feet) will not require raising the crest of the dike embankment. A conceptual sketch and profile of Alternative 2 is attached as Figures 4A and 4B.

To accomplish this, the proposed spillway crest elevation is reduced to 137.75; the spillway length would still be 140 feet assuming a horseshoe shaped ogee crest. The depth of the proposed bridge opening would be increased by reducing the existing invert elevation of 137.75 feet to 129.0 feet. Likewise, the invert of the downstream channel will also be reduced at the upstream end (immediately downstream of the bridge) from 137.0 feet to 128.0 feet. The downstream invert elevation (at the confluence with Bubbling Brook) will remain unchanged at elevation 125.0 feet.

2.3.3 Alternative 3

A third alternative that makes it possible to maintain the current normal elevation of the pond without raising the dam or dike is to design a spillway of such length that the SDF storm could flow over the spillway without overtopping the dike. This alternative still requires the bridge opening to be enlarged to allow the flow under the bridge.

Therefore, to restrict the SDF spillway overtopping to 2 feet or less, the height required to prevent overtopping of the dike, would require a spillway length of 365 feet. The bridge opening would need to be enlarged by lowering the apron under the bridge by 7 feet 9 inches. The opening could also be enlarged by widening the bridge, or a combination of widening and lowering the apron. A conceptual sketch and profile of Alternative 3 is attached as Figures 5A and 5B.

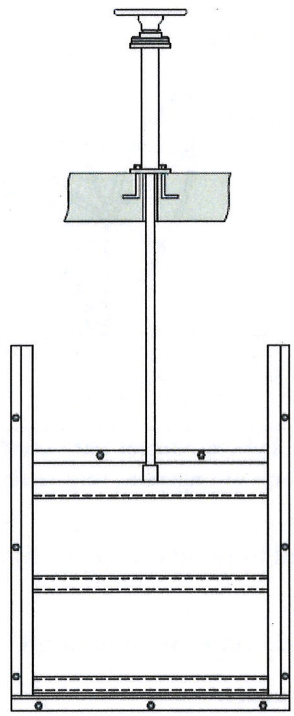
2.3.4 Alternative 4

Flood gates were considered as an additional alternative that might allow maintenance of the existing normal pool elevation without changing the elevation of the dike. The primary drawback of this approach is that it would require the active intervention of personnel, which is problematic given that the facility is not normally staffed. Ideally these personnel would also need to be aware of weather forecasts to assure their availability during times of anticipated heavy rainfall and flooding. An additional consideration is that the operation of flood gates during certain storm events could result in rapid and dramatic changes in downstream discharge rates, which could adversely impact public safety downstream. There would need to be a public education campaign and the use of an ample warning system to inform people of anticipated increases in flow as a result of anticipated need to open the gates. Lastly, operable mechanical equipment will increase annual Operation and Maintenance costs. The bridge cross sectional area would still need to be increased under this option, to allow the SDF to pass under the bridge. The following paragraphs discuss the various gate alternatives.

Flood gates are typically used on larger dams that function as flood control or hydroelectric power generation dams. They are most frequently used where there is qualified manpower available on a regular basis to operate, maintain, and repair them. They can be automated to react to changing water levels. This feature would be useful at Willett Pond dam to maintain sufficient cross sectional area under the bridge to pass the SDF where there is no full-time operating staff. However, compared to stationary spillway structures, such as the concrete ogee weir in the other alternatives, gates require maintenance and occasional repair by trained technicians. They also have a design life, and will eventually need replacement. These issues are discussed in more detail below following descriptions of the major gate types.

Three general gate types (radial, hinged crest, vertical lift) were considered for use at the Willett Pond Dam and are summarized in the following sections. Sources utilized in preparing the information provided include Army Corps Engineering Manuals 1110-2-2607, 1110-2-2701, 1110-2-2702, and product data sheets from the Rodney Hunt Company.

The current method for controlling water level in the lake is through the use of weir boards in the spillway channel, which are manually installed and removed as needed. Additionally there are two low-level outlet pipes with manually-operated vertical slide gate valves for lowering the lake level if needed. These are not effective for use as flood gates for flow rates of the magnitude of the SDF at Willett Pond. Manually operated vertical slide gates of the size needed here (*Figure below*) would not be practical.

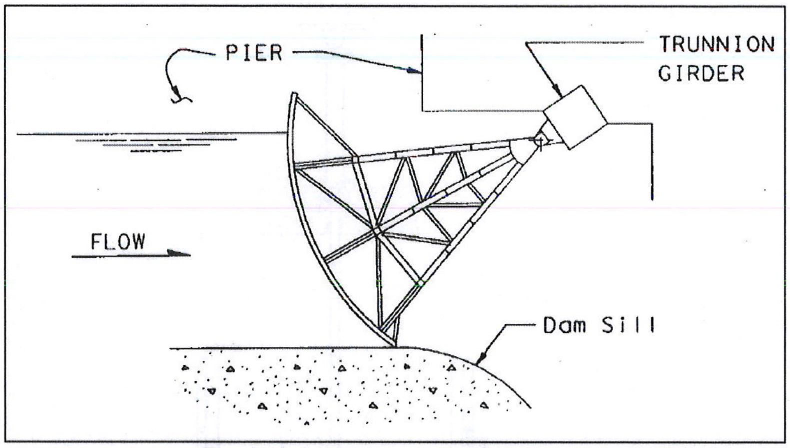


Vertical Slide Gate (Source: Rodney Hunt Company)

A slide gate valve consists of a sliding gate mounted on a seating surface and raised or lowered manually by means of a stem or rod. (A slide gate could be operated with an electrically or hydraulically powered hoist, but this then closely approaches the vertical slide gate described below.) It is typically placed at the up- or down stream end of a flow channel, or covering an opening in a wall or barrier. Sizes can vary but are limited by the unseating pressure of the water and the weight of the gate that must be lifted. Due to the size of the spillway opening required to safely pass the SDF, it would likely be necessary to install multiple gates at Willett Pond and construct intermediate gate channels to hold them, restricting flow through the outlet. In addition, the time required to open and close the gates would be significantly longer than the other gate options. For these reasons, a manually operated vertical slide gate option is not further included in this discussion.

2.3.4.1 Radial (Tainter) Gate

A tainter gate (figure below) is a type of floodgate that is convex on the upstream side and is mounted on radial arms that rotate about an axis on the downstream side. The gate disc is typically raised or lowered through the use of an electrically powered cable drum hoist, although hydraulic actuation units can also be used. Because the gate arms are angled toward a common axis of rotation, the moment that must be resisted by the gate supports due to water pressure is greatly reduced. Tainter gates come in a variety of sizes and can be quite large.



Typical tainter gate configuration
(Source: U.S. Army Corps of Engineers, Engineering Manual 1110-2-2607, July 1995)

Tainter gates are frequently used in flood control or hydropower projects, and provide a number of advantages:

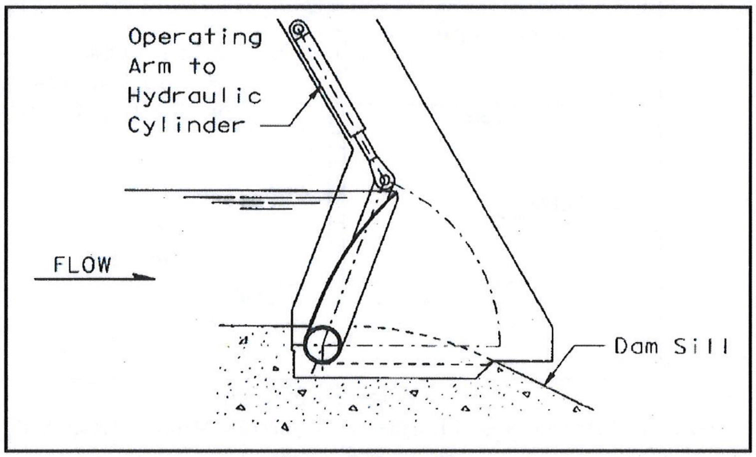
- Efficient transfer of hydrostatic loads through the trunnion, due to its radial shape
- Requires a low hoist capacity
- Relatively fast operating speed
- Gate slots are not required (side and bottom seals are used)
- Geometry provides favorable hydraulic discharge characteristics
- Can release a large quantity of water in a short time, since the entire spillway width is used

There are, however, a number of disadvantages to using a tainter gate:

- More structural concrete may need to be constructed relative to other gate types, due to the location of a trunnion downstream of the gate and support hoist.
- Ends of support members may interfere with high flows. Long strut arms may be necessary to clear high flood levels.
- Because water typically flows under rather than over the gate, precise control of impoundment levels may be difficult.
- Gate cannot be opened in the event of a power failure

2.3.4.2 Hinged Crest Gate

A hinged crest gate (*Figure below*) acts as a spillway weir that can be raised or lowered by hydraulic cylinders which pivot the gate about its hinged base. In this way the gate can retain impoundment pool, or be lowered to pass flood flow or lower the pond's water surface. The gate is typically sealed at the base and edges when raised to its upright position. Hydraulic controls can be automated to raise or lower the gate in response to impoundment level, and the gate can be configured to either open or remain closed in the event of a power failure. Maximum gate dimensions for the simplest type of hinged-crest gate, which utilizes a hydraulic arm at one end, is approximately 8 feet in height and 35 feet in width. A torque-tube style hinged crest gate, with a torque tube along the invert rotated by an actuator enclosed in the concrete abutment, can be slightly larger with a height of 10 feet and a length of 35 feet. Hinged-crest gates with hydraulic cylinders underneath can be up to 200 feet long, although these require a drop downstream of the spillway crest to provide room for mounting the cylinders.



Hinged Crest Gate
(Source: U.S. Army Corps of Engineers, Engineering Manual 1110-2-2607, July 1995)

Hinged crest gates provide the following advantages:

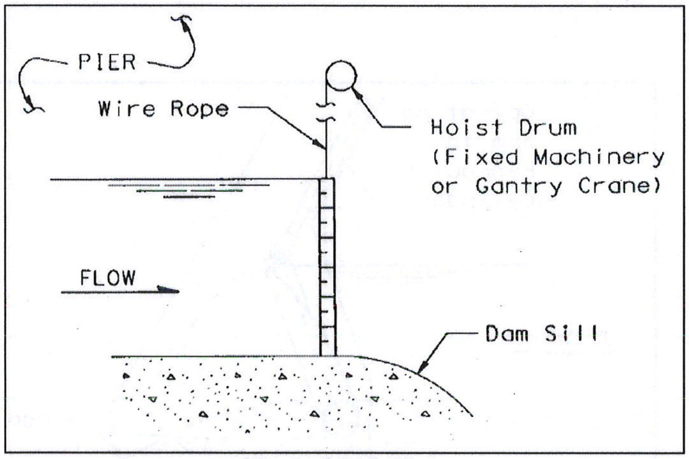
- Precise control of pond level and outflow. "Normal pool" elevation easily regulated by adjusting the gate.
- Gate disc can be configured to fit down into the spillway invert to minimize flow obstruction
- Can be automated based on pond level, to open in times of flood
- Can release a large quantity of water in a short time, since the entire spillway width is used

Disadvantages of the hinged crest gate include:

- Deeper excavation required than for tainter gate, to accommodate gate disc when open
- More expensive than tainter gate, although concrete work may be less expensive
- Hydraulic arms may interfere with flow at gate end(s)
- There is some added maintenance in keeping the gate seat clear of debris

2.3.4.3 Vertical Lift Gate

Vertical lift gates consist of a gate that can be raised vertically, sliding up and down via roller guides at both ends. They typically utilize a horizontal rather than a vertical framing system, to effectively transfer loads to the side bearing surfaces. Vertical lift gates are hoisted at both ends by cables using a fixed hoist drum or, for several gates, a hoisting system mounted on a gantry crane.



Vertical Lift Gate
(Source: U.S. Army Corps of Engineers, Engineering Manual 1110-2-2607, July 1995)

Vertical lift gates provide the following advantages:

- Concrete support dimensions are reduced in upstream and downstream directions
- Simple design
- Provides clear and unobstructed opening
- Can have relatively fast operating speed, depending on hoist mechanism
- Geometry provides favorable hydraulic discharge characteristics
- Can release a large quantity of water in a short time, since the entire spillway width is used

Disadvantages of the vertical lift gate include:

- Because water typically flows under rather than over the gate, precise control of impoundment levels may be difficult.
- Regular maintenance required to keep gates operating and roller guides free of debris
- Side piers must be extended upward to a considerable height above high water to accommodate guide slots for the gate in fully open position
- Relatively large hoist capacity required for heavy lifting load
- Gate slots can lead to cavitation and debris collection

2.3.4.4 Applicability to Willett Pond

A brief discussion of issues surrounding design and construction, operability, and maintenance of three of the gate alternatives presented above: tainter gate, hinged-crest gate, and vertical lift gate is presented in the following sections.

To provide sufficient hydraulic capacity to convey the spillway design flood (SDF) without overtopping the dam or dike, the existing spillway section must be deepened and/or widened considerably. This will be required regardless of what improvement alternative is ultimately selected. The spillway discharge channel will likewise need to be excavated, resulting in a more gradual slope for this waterway. A spillway or gate will be installed to maintain existing pond elevation above a deepened discharge channel during non-flood periods.

The spillway section, approach area, and discharge channel appear to be founded on bedrock. This is expected to make excavation more difficult and expensive, but will ultimately provide a sound underpinning for the spillway and associated appurtenances. It is expected that the hinged-crest gate would require more excavation than the tainter gate or vertical lift gate options, since the gate, supports, and possibly hydraulic arms must fit into the spillway channel invert.

The tainter gate and vertical lift gate would both require additional structural design and structural concrete than the hinged-crest gate. This is due to the overhead structures required to support the trunnion of the tainter gate, or the piers needed to raise the vertical slide gate up above the flood elevation. For similar reasons, the structures associated with these alternatives would be more imposing and obvious, which could be an aesthetic concern. The hinged-crest gate would maintain a lower visual profile.

2.3.4.5 Operability

One of the drawbacks of the gate systems is that they must be operated, which means some entity must assume responsibility for responding to emergencies at any time. The operation would be formalized and incorporated into the Emergency Action Plan for the dam, so this responsibility would be made clear in that document. The operational procedures should be incorporated into the dam Operation and Maintenance Manual.

The three gate options would each be fitted with hydraulic, pneumatic, electric power or some combination to open and close the gates. The response time of all three of these gate options is expected to be relatively short. Depending upon the hoist equipment selected for the vertical lift gate, this option may have a longer response time than the other two gate options. Since discharge flows under the tainter gate or vertical lift gate, rather than over the top, controlling the level of the impoundment with either of these options may prove difficult relative to the hinged-crest gate.

The hinged crest gate was therefore evaluated in more detail. The hinged-crest gate could be automated to control the pond level, i.e. to increase or decrease outflow based on the current water surface elevation of the impoundment. The gate can be programmed in a variety of ways to assure that adequate capacity is provided to convey the SDF, while not dropping the water level pre-maturely below a desired level (normal pond level). A hinged spillway gate can drop through its full swing in 10 to 15 minutes. The gate can be set to maintain a pre-determined normal pool level and the gate can be adjusted continuously or it can adjust at some specified time interval.

To avoid false alarms, such as the hinged crest gate adjusting due to wave action, a higher level threshold can be set so the gate won't begin to adjust until it gets to that higher threshold level. This would also allow the existing hydrologic response of the impoundment to be maintained up to a pre-determined return frequency storm, such as the 50-year or 100-year flood event for instance. This would allow the impoundment to store flood waters as it currently does, and maintain current peak flow characteristics for these flood events. After the pond level rose to the higher level threshold the gate would begin to adjust. The system could then be further programmed to initially adjust modestly to bring the water level down to the normal pond level gradually as opposed to one huge gate adjustment which could release a lot of water at one time. If the inflow was very large, as with the design flood event, the gate would adjust progressively more dramatically as necessary to keep the water level dropping.

To make the system more reliable, in addition to pond level monitors, inflow measuring monitors could be installed that measures the inflow from the main tributary or some representative tributary. This would allow the gate to be adjusted to accommodate the rate of inflow from the contributing watershed. It would give more warning of the anticipated water level condition in the pond, whether that is an expected rise in water surface elevation because of increases in inflow, or reduction of water surface elevation due to reduction in rate of inflow. The inflow monitor could be a water level monitoring device related to flow capacity at a channel section or a measuring weir structure. The specific programming of the gate is based upon the inflow hydrograph of the Spillway Design Flood and setting the gate response thresholds such that the gate has adequate time to react to the rate of inflow. There is great benefit in programming the gate to not increase flood flows up to the 100 year flood event if possible, so that the flood potential for this regulatory flood event does not change downstream.

Any of these systems would require an electrical power source and ideally they would run on 3 Phase AC power. Since the SDF for the dam is an extremely large storm event, it should be assumed there is a high likelihood of a power outage during the storm.

A relatively inexpensive way to provide backup capacity is by use of UPS (uninterrupted power source (battery backup)) to power the sensors and the opening of the solenoid valves to allow the hydraulic cylinders of the hinged gate to drain as the hydrostatic pressure pushes the spillway gate down. This backup power system would not have enough power to raise the spillway gate though. So the UPS would provide enough power to assure the full capacity of spillway is available for the big event, but would also allow more water to drain out of the impoundment subjecting downstream areas to more flow. This is still better than the dam breaching due to overtopping and failure, although not as good as a system with full backup power that would allow the gates to be raised as the peak of the flood event passes through the impoundment.

Gates can be fitted with gas cylinders that will perform a "one-shot" operation during a power loss, such as opening or closing the gate to a preset level. This is referred to as a hydraulic accumulator which stores hydraulic energy to move the gate. These are not inexpensive. There would still be the need to have electrical power backup to operate the sensors and gate solenoid valves, whether UPS and/or generator. This equipment, as with the electronic controls for the overall gate system, ideally should be stored in a weather tight enclosure like a gate house.

The most complete system would be the full power backup electrical generator which would provide sufficient power to allow full system capability. This would be the most expensive option. For a gate of the size required at Willett Pond, a portable gasoline-powered generator with a capacity of approximately 4,000 to 6,000 watts will be needed.

Any system would involve an alarm system that would notify of power loss and need for manual intervention.

2.3.4.6 Maintenance

Gates with electrical/hydraulic components add some maintenance tasks that are not necessary with purely mechanical systems. This adds to the time and effort required for maintenance, and as with operations, it must be made clear who is responsible for maintenance. Maintenance might require a skill set not possessed by local DPWs. There may be other entities, such as the MDC for example, who have this capability and who could be contracted to provide these services. Clearly there are specialty contractors who can provide this service. The maintenance procedures and frequency should be incorporated into the dam Operation and Maintenance Manual.

Any type of gate or other equipment that is installed will require maintenance to stay in good working order and avoid shortening its design life. The gate option that is expected to be the most maintenance-intensive is the vertical lift gate. While all three gate options have some type of hoist or hydraulic actuator(s), the vertical lift gate also has rollers along both sides that must be kept free of debris and in working condition. In addition, debris will likely collect in or around the vertical side slots and would need to be removed. Both the vertical slide gate and the tainter gate would be more prone to collecting debris at the base of the gate when partially open. This debris would be more difficult to clear than debris that might collect at the surface of the hinged crest gate.

Any mechanical gate needs to be exercised periodically to be sure the gate is operational and to identify additional maintenance that might not be evident during visual inspection. As part of the maintenance routine, we would expect exercising of the gate through its full range of motion should be performed at least once per year. This means opening the gate to its full operational capacity. Because there would only be one gate at Willett Pond, opening the gate completely would likely release an unacceptable quantity of water downstream. For this reason, weir board slots should be cast in place during the construction of the gate support structure, allowing weir boards to be inserted prior to exercising the gate to its full capacity.

2.3.4.7 Costs

The costs associated with purchase and installation of a gate are considerably less expensive compared to the expense of constructing a concrete spillway in the pond, as in the three weir alternatives, especially with the Long Spillway Alternative 3. On the other hand, gates require additional maintenance and have a design life that would require replacement after a certain number of years. We estimate the annual maintenance costs would range from approximately $3,000 to $5,000. The life of the gates depends on the degree of regular maintenance, but a design life of 50 years is a reasonable estimate. This is far less than a properly constructed stationary concrete weir, which could last twice as long.

An opinion of cost table is attached indicating the costs of the gate options. Much of the work required for preparing the site for the gates is the same as for implementation of the weir alternatives. Other costs, such as supporting the bridge and increasing the cross sectional area beneath the bridge would still pertain. The table has been organized to separate up stream costs from down stream costs, with the work required beneath the bridge part of the down stream costs.

In summary, the installed gate costs are approximately $175,000 for each of the tainter and vertical lift gates. The hinged bottom gate is more expensive at approximately $325,000. These costs assume the concrete support structure at the spillway is already in place. Automating the hinged bottom gate can range from $15,000 to $65,000 according to Rodney Hunt. We have assumed a cost of $50,000. Total costs for the project employing a gate instead of a fixed spillway range from approximately $1,134,000 to $2,430,000 (slightly lower for a tainter gate).

2.3.5 Discarded Alternatives

As mentioned above, a variety of additional alternatives were considered at a conceptual level but discarded as they were considered infeasible for one or more reasons. These discarded alternatives are discussed very briefly here.

2.3.5.1 Dam Removal

The benefits of dam removal would potentially include the restoration of cold water fishery habitat, a limited area of bordering vegetated wetlands, and historic upland habitats. Dam removal would also permanently eliminate flood hazard and maintenance responsibilities associated with the dam. A dam removal alternative would require substantial excavation of the dam back to the original stream bed and installation of very large culverts or a long bridge. Culverts would likely be more cost effective than a large bridge and would minimize the need for permanent disposal of the existing dam embankment material.

This approach would, however, substantially disrupt the road and would result in the loss of the existing recreational benefits of the existing pond. It would require the removal and disposal of approximately 20,000 cubic yards of soil and concrete making up the dam, as well as potentially thousands of yards of accumulated sediment trapped behind the dam. If the sediment were to be found to be contaminated, the sediment removal costs alone could be on the order of hundreds of thousands to millions of dollars. Although the roadway and bridge are not owned by the NRLHA, the cost to the State of Massachusetts to replace the road and bridge for the full length of the existing dam would cost millions of dollars.

2.3.5.2 Draining the Pond without Dam Removal

Another option that was considered was draining the pond without removing the dam, as either a permanent measure or an interim condition while awaiting implementation of another alternative. The low level outlet structure would allow Willett Pond to be entirely drained. However, a preliminary hydrologic analysis of this alternative indicated that even if the pond were fully drained, the impoundment lacks sufficient storage capacity to prevent overtopping of the dike during the design flood given the existing spillway configuration, rendering this alternative infeasible. As an interim measure, it could be argued that flooding from storms less severe than the SDF would be less likely to overtop due to the increased storage capacity. As a permanent measure, this alternative does not address the problems, since it would still be a jurisdictional High Hazard dam.

2.3.5.3 Breakaway Flashboards

Another alternative that operates similarly to flood gates would be the use of breakaway flashboards or soil plugs. This approach would allow water releases to be initiated without the same level of active intervention associated with flood gate. However, the method may not be as reliable as the other alternatives considered here. Their use still results in rapid water level change downstream that is not as controllable as flood gates would be. Additional inspection and maintenance would be required as intermediate size storms may break the boards or wash out the plugs (partially or completely) and they would need to be replaced periodically. We have discounted this option due to its lack of reliability and additional operation ad maintenance requirements.

2.3.5.4 Armored Crest

This alternative would entail lowering the roadway across a section of the dam and armoring the downstream face of the dam to withstand overtopping flow during large storms. In this way the roadway itself would function as an auxiliary or emergency spillway.

To limit the height by which the roadway would be lowered (necessary to reduce the frequency of overtopping during smaller storm events, which would close the road each time it was overtopped), the length of the lowered area should be maximized. The extent of the lowered area, however, is constrained to the distance between the right (west) side of the dam by the bridge over the spillway and the left (east) side by the dam abutment. The available length of the lowered section of road would be further limited by the length required for a gradual transition in grade (vertical curve) of the public roadway at each end. After taking these constraints into account, it was found that to pass the required spillway design flood the roadway would need to be lowered below the elevation of the current spillway. Under these circumstances the road would, in effect, become the primary spillway for the pond with constant flow overtopping the road. This, of course, is unacceptable.

In addition to the limitations on the auxiliary spillway dimensions, it was found that a relatively large shear stress (resulting from the water falling over the crest at a high velocity) would act upon the downstream face of the dam during design flood flows. Given the overtopping flow depths and velocities that would result from this configuration, riprap would not be sufficient to protect the downstream face of the dam from erosion and head cutting. A more substantial protection measure, such as roller compacted concrete, would be required. The disposition of the water downstream is another concern. While the appropriate armor might prevent dam failure, the wetlands downstream of the dam would be severely damaged during a design storm event. For these reasons, the armored dam alternative was deemed infeasible.

2.4 References

[1] U.S. Army Corps of Engineers, Hydrologic Engineering Center. (April 2008). Hydrologic Modeling System, Version 3.2. Davis, California.

[2] U.S. Department of the Interior, Bureau of Reclamation. (1987). "Design of Small Dams", 3rd Edition.

[3] U.S. Department of Commerce, National Oceanic & Atmospheric Administration. (June 1978). "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian". Washington, D.C.

[4] U.S. Army Corps of Engineers, New England Division. (June 20, 1958; Revised May 1, 1959). Northeast Flood Studies, "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River, Connecticut". Boston, Mass.

3 Dam Break Model and Inundation Modeling

Dam break and inundation modeling was performed previously by Fuss & O'Neill as part of the Emergency Action Plan prepared in 2007. The alternatives presented in this report should not affect the inundation mapping previously prepared and was not included as part of the scope of work for this investigation.

4 Geotechnical Assessment

4.1 General

The dam was assessed for seepage and structural stability for the purpose of determining whether structural improvements to the dam are necessary and, if so, what remedial measures should be taken. Soil borings, slug tests, and investigations of gate house seepage were performed to provide information for modeling and analysis.

Four borings on the dam crest and three borings on the dike were performed between May 11 and May 13, 2009 by GeoSearch Drilling of Fitchburg, Massachusetts, and logged by a

Fuss & O'Neill engineer. The locations are indicated on the attached plan Figures 2A, 2B, and 2C and labeled B-1 through B-7. Samples were collected from the borings for subsequent testing and review.

4.2 Subsurface Explorations

Four borings on the dam crest and three borings on the dike were completed as indicated on the attached plan. The borings were advanced with hollow stem augers. Boring depths ranged from 22 to 27.5 feet on the dam and from 8 to 18 feet on the dike. In each boring, Standard Penetration Testing (ASTM D 1586) and split spoon sampling were performed. Continuous split spoon sampling was performed at borings B-1, B-5, B-6, and B-7. At borings B-2, B-3, and B-4, split spoon sampling was performed at 5 foot intervals. Soil samples were classified in accordance with the Unified Soil Classification System (USCS). After drilling was completed, the boreholes were backfilled with the soil cuttings. On the dam, two borings were completed on each side of the concrete core wall located as follows:

- Boring B-1: located approximately 255 feet left of the spillway and 7 feet from the upstream face
- Boring B-2: located approximately 150 feet left of the spillway and 17 feet from the upstream face
- Boring B-3: located approximately 260 feet left of the spillway and 17 feet from the upstream face
- Boring B-4: located approximately 140 feet left of the spillway and 7 feet from the upstream face
- Boring B-5: located on the dike south of an existing concrete stairway and landing
- Boring B-6: located on the dike crest near the northern end of the dike
- Boring B-7: located on the dike crest near the southern end of the dike

Observation wells were installed in Borings B-1, B-3, B-6 and B-7. The wells were installed to obtain stabilized groundwater elevations in the dam and dike embankments. The wells each had 10 feet of 2-inch PVC screen backfilled with sand and centered on the approximate water level within the dam/dike as determined during the boring process. The bottom of the screen was set at depths of 12 feet, 25 feet, 8 feet, and 12 feet for Borings B-1, B-3, B-6, and B-7, respectively.

4.3 Subsurface Conditions

4.3.1 Dam Embankment

In general, the soil profile through the dam consisted of approximately 22 to 26 feet of embankment fill underlain by natural sand, gravel, and bedrock. The embankment fill generally consisted of sand with varying amounts of silt. Boring logs are included in Appendix B. General findings per boring were as follows:

- Boring B-1: Beneath 22 feet of the silty sand fill material, a thin layer of weathered bedrock was encountered at approximately 23 feet. The weathered bedrock was augered an additional 4-feet before encountering very hard rock and the boring was terminated at 27 feet. The dam embankment fill was generally very loose to loose at this location. Groundwater was encountered at approximately 5 feet below the dam crest.
- Boring B-2: Beneath 26 feet of silty sand fill material, weathered bedrock was encountered. The boring was advanced to 27.5 feet before reaching auger refusal. The dam embankment fill was generally loose at this location. Groundwater was encountered at approximately 22 feet below the dam crest.
- Boring B-3: Beneath 25 feet of silty sand fill material, a thin layer of silty clay was encountered on top of weathered bedrock. Split spoon refusal was encountered at 26.3 feet. The dam embankment fill encountered was generally loose to very loose with a 1-foot layer of dense material on top of the weathered bedrock. Groundwater was encountered at approximately 21 feet below the dam crest.
- Boring B-4: Fine silty sand was encountered to a depth of 21 feet, with a layer of sandy gravel at the base of the bore hole. The boring was not advanced to refusal. The dam embankment fill was generally very loose. Groundwater was encountered at approximately 5 feet below the dam crest.

4.3.2 Dike Embankment

Depth of fill encountered in the borings on the dike crest ranged from 0 to 15 feet. The dike fill primarily consisted of silty sand down to natural grade. In borings B-5 and B-6, gravel and rock caused spoon refusal. In boring B-7, a location where the dike is 3 to 4 feet high, glacial till was encountered at 8 to 10 feet and the borehole was advanced to a depth of 14 feet without spoon refusal. At each location, two additional probes (without sampling) were performed for the purpose of directly confirming core wall presence and confirming core wall functionality by checking groundwater levels upstream of the core wall. The core wall was encountered at each location at approximately 2 to 3 feet below the surface. At boring B-5, groundwater upstream of the wall was measured at 5 feet below the dike crest and the water level downstream of the wall was measured at 13.5 feet below the dike crest. At borings B-6 and B-7, water levels upstream of the wall were measured at 3.5 to 4 feet, and water levels downstream of the core wall were measured at approximately 6 feet.

4.4 Seepage Analysis

Several areas of seepage were observed at the toe of the dike during the Phase II investigation. No apparent seepage was observed at the dam. Primary areas of suspected seepage in past inspections included a generally wet area along the toe of the dam; an iron-stained seep in the downstream concrete apron of the spillway; seepage into the gate-house structure; two small wetland areas adjacent to the dike, and a report of persistent basement flooding near the south end of the dike. Of these, the water at the toe of the dam appears to be standing water of the wetland immediately downstream of the dam. Anecdotal evidence revealed that there were wetlands and "springs" at this location before the dam was constructed in its present location. The iron-stained seepage in the concrete spillway apron was not observed during our inspections.

Seepage was evaluated several different ways, including: phreatic surface modeling; measuring actual water levels upstream and downstream of the core wall at five boring locations; performing slug testing at two monitoring wells installed on the dam and dike; and performing an investigation of the gate house structure.

The inspections and analyses indicate that the concrete core wall is functioning as designed and that there are no apparent excessive gradients within the dam. The wet areas at the toe of the dam and along the dike are likely related to external causes and do not adversely affect the safety of the dam and dike. The gate house seepage was found to be closely related to the water surface elevation within the downstream wetland, with all observed inflow resulting from the drain pipe connecting the gatehouse to the wetland (see Section 4.5.2).

4.4.1 Main Dam

Four borings were completed on the dam to assess the materials of construction and observe the water surface elevations within the dam. Two borings in the dam were located upstream of the dam core wall and two were located on the downstream side of the same core wall. Water level observations were made during the course of the drilling. The water surface upstream of the core wall was consistent between the upstream holes at an elevation 5 feet below the road surface. The water level depths in the downstream hole were observed to be approximately 22 to 23 feet at the time of the drilling operations. We chose the highest point of the dam for analysis (located near borings B-1 and B-2). In addition to being the highest point of the dam, these locations are in close proximity to the low level outlet and gate house.
Monitoring wells were installed in two locations (borings B-1 and B-3) near the center of the dam where the embankment was highest. Water level measurements were obtained immediately after the wells were installed and approximately one month following well installation. Impoundment water surface elevation was approximately 2 inches over the dam stop logs (elevation 140.0 NGVD) during all measurements. The water levels upstream of the core wall were 5 feet below the road surface (elevation 138.3 NGVD). The water levels in the downstream well were measured at approximately 20.5 feet below the road surface (elevation 122.5 NGVD). These results corroborate the phreatic surface predicted in the model. We believe that the model and field investigation results show that the dam core wall is functioning as designed and that the apparent seepage along the dam toe is actually backwater from the wetlands and pond downstream.

Slug testing was used to estimate the hydraulic conductivity of the dam soils. Based on the sandy soil found in both the dam and dike and their expected rapid water level recovery, the slug test was performed using a transducer for water level measurement and a data-logger. Excel spreadsheets and summary tables are included in Appendix C. The hydraulic conductivity at the dam, based on the analytical method by Bouwer and Rice (1976), is approximately 17.9 feet per day at boring B-2.

With respect to dam operating and safety conditions, the borings indicated that the core wall is functioning as designed to reduce the elevation of the phreatic surface of the flow through the dam and with it any potential destabilizing seepage pressures. Without a core wall, the phreatic surface upstream of the wall would be expected to be lower than we observed in the field. Conversely, the phreatic surface at the downstream location in the absence of the core wall would be expected to be higher than the condition we found in the field. Although the soils making up the dam were loose, this is not a concern for dam safety, because the core wall prevents high water levels and resulting seepage pressures on the downstream side of the dam embankment. The core wall is the seepage control mechanism, and the downstream embankment slope is stable at a 2:1 slope. Wet areas at the toe of the dam do not appear to be related to seepage through the dam. Rather, these areas appear more likely to be related to historical wetlands or springs, runoff patterns, and backwater from the downstream pond and wetlands.

Using the SEEP2D model (GMS Software) to assess the seepage conditions, it was found that the phreatic surface under normal pool conditions (elevation 140) is lower than the core wall, so there is no flow. In reality there is likely a minor amount of water getting under or past the core wall, but the measured groundwater levels in the piezometers indicate the amount is negligible. Under full pond conditions (elev. 145), the phreatic surface barely clears the top of the core wall, and the seepage over the wall is so small there is no significant pore pressure buildup down gradient of the core wall.

4.4.2 Gate House

Seepage investigation at the gate house included visual inspection of the gate house sump and a methodical review of seepage into the gate house related to water surface elevation in the wetland immediately downstream of the gate house. The gate house is located on the down stream slope of the dam. The two 20-inch cast iron outlet pipes run through the dam to the gate house with a knife valve on each pipe. The gate house has a concrete sump where water is often observed to be standing in the bottom of the sump. The sump is equipped with a 6-inch diameter cast iron and PVC drain to let water out of the sump. The twin 20-inch outlet pipes and the 6-inch drain all daylight in a concrete basin downstream of the gate house, where water is routed through the basin to a Parshall flume before being released into the wetland at the toe of the dam. It has been suggested in past inspection reports that the standing water in the bottom of the gate house might be seepage from the dam embankment, perhaps traveling along the outlet pipes or seeping in from outside the gate house. In all our inspections of the gate house, we have never observed water seeping in along the low level outlet pipes or leakage from the pipe valves. We have not witnessed any visual evidence of flow in any direction in the gate house sump.

The procedure for assessing the source of the water in the gate house sump was to alter the water levels inside and outside the gate house and observe the response of water flow in the gate house and the area down gradient of the gate house. The water levels outside the gate house were controlled by adjusting the flow through the twin 20-inch outlet pipes to the concrete basin down gradient of the gate house. By opening the gate valves, the water level in the concrete basin and in the wetland around the outlet structure could be raised. The water level inside the gate house could be lowered by pumping water out of the gate house sump using a one-inch submersible pump (approx. capacity 21 gpm). Water levels in the gate house sump were measured from the floor grate (known elevation) in the gate house. Outside water elevations were measured from the top of the concrete wall of the down gradient basin. Flow through the gate house PVC sump drain was monitored visually for flow direction.

The water level elevations were determined by subtracting measured values from elevations of known points established by field survey of the dam using GPS/RTK and conventional field survey techniques. The known points included top of grate in the gate house (elev. 127.80) and top of wall at Parshall flume (elev. 123.54). Measurements to the water surface from these points were made by directly reading a metal tape. The results of the investigation are presented in the table below.

Action or Condition at Time of Measurements	Water Surface Elevation in Gate House	Water Surface Elevation in Wetland	Response
Field Survey 3/31/2009	120.7	120.6	
Dam Observations 5/28/2009	120.6	120.5	
Seepage Investigation 6/11/2009			
Initial condition - 8:15 am	120.25	120.27	Measurements taken upon arrival at dam. No flow into or out of gate house sump was noted.
Pump water from the gate house sump - 8:30 a.m.	120.21	120.27	Water level decreases in the gate house sump.
Outlet valves closed, pump kept running - 8:50 a.m.	120.13	120.18	Water noted flowing through PVC pipe into sump from down gradient concrete basin.
Flow into sump from concrete basin partially blocked - 9:20 a.m.	120.11	120.18	Water level in gate house sump decreases
Gate house sump pump shut off. Valve opened to ½ of initial setting - 9:40 a.m.	120.17	120.17	Water flows from concrete basin back through PVC pipe into gate house sump.
Valve opened to arrival position 10:00 a.m.	120.25	120.23	Water rises outside and flows back through PVC pipe into gate house sump.
Valve opened 3 additional turns to raise water level in the wetland 10:20 a.m.	120.28	120.27	Outside water level rises further; water flows through the PVC sump drain pipe back into the gate house sump.

Conditions during our topographic survey and subsequent measurements indicate the elevation of the water in the gate house was very close to the elevation of the water in the downstream elevation. The data in the table above also indicate that water level changes in the gate house are very closely related to water levels in the wetland below the dam. Every time the water level outside the gate house was higher than inside, flow was toward the gate house sump to equilibrate the inside and outside water levels. No seepage flow was observed from any part of the gate house floor, walls, or outlet pipes during our investigation. We believe that due to the close correspondence of water elevations in the gate house to the down gradient water levels outside the gate house, the water in the sump results from back flow into the sump and not from seepage through the dam.

4.4.3 Dike

Three areas of interest were identified along the dike. In two areas (Boring B-5 and Boring B-6), small wetlands have formed at the down gradient toe of the dike. At the third location near the south end of the dike (Boring B-7), an adjacent homeowner is experiencing significant seepage into the basement of his home. Boring locations were selected in the vicinity of the areas of interest to evaluate the functioning of the dike. At each of these three locations, a boring was drilled on the downstream side of the wall and split spoon sampling was performed. The borings were advanced to refusal or until the hole was 5 feet into the natural grade below the base of dike fill. Two additional borings without split spoon sampling were advanced in each location. At each drilling location, one boring was located upstream of the wall for the purpose of determining the phreatic surface in the dam and a second boring was advanced to confirm presence of the core wall.

The phreatic surface in the dike upstream of the core wall ranged between 3.5 and 5 feet below the dike crest, or at the approximate water level of the pond. The water level in the downstream holes was 6 feet below the dike crest at Borings B-6 and B-7 and 13.5 feet at Boring B-5. The seepage at the toe of the dike at B-5 is also approximately 13.5 feet below the dike crest, indicating a flat gradient from the core wall to the seepage area. This means there may be a natural low area at this location where water collects or that water may be migrating beneath the core wall. However, the slope appears stable, so it is unlikely that any vertical seepage pressure is being exerted on the downstream side of the dike embankment at this location.

Monitoring wells were installed at Borings B-6 and B-7, because these holes exhibited a smaller differential between the upstream and downstream water levels than at boring B-5 during drilling. This allowed water levels to stabilize for future measurements. The water levels upstream of the core wall were 3.5 to 5 feet below the dike crest, which translates to elevations of 138.3 to 139.8 feet. The water levels in the downstream wells were measured at approximately 6 feet below the dike crest (elevation 137.3 NGVD), similar to that observed during drilling.

Slug testing was performed to evaluate the hydraulic conductivity of the dike fill soil at boring B-6 near the north end of the dike. Based on the sandy soil found in the dike and its expected rapid water level recovery, the slug test was performed using transducer measurement and a data-logger. Results are included in Appendix C. The hydraulic conductivity at the dike fill soil, based on Bouwer and Rice (1976) methodology, is 19.2 feet per day at boring B-6.

Although minor seepage was noted at two locations along the dike down stream toe, no signs of dike instability were noted in these areas. We recommend that these areas be monitored periodically for increases in flow. If these seeps worsen, it may be necessary to investigate these areas by exposing the subsurface conditions of the dike and core wall. These areas are located down gradient of borings B-5 and B-6, and are labeled on the attached Figures 2B and 2C.

4.5 Stability Analysis

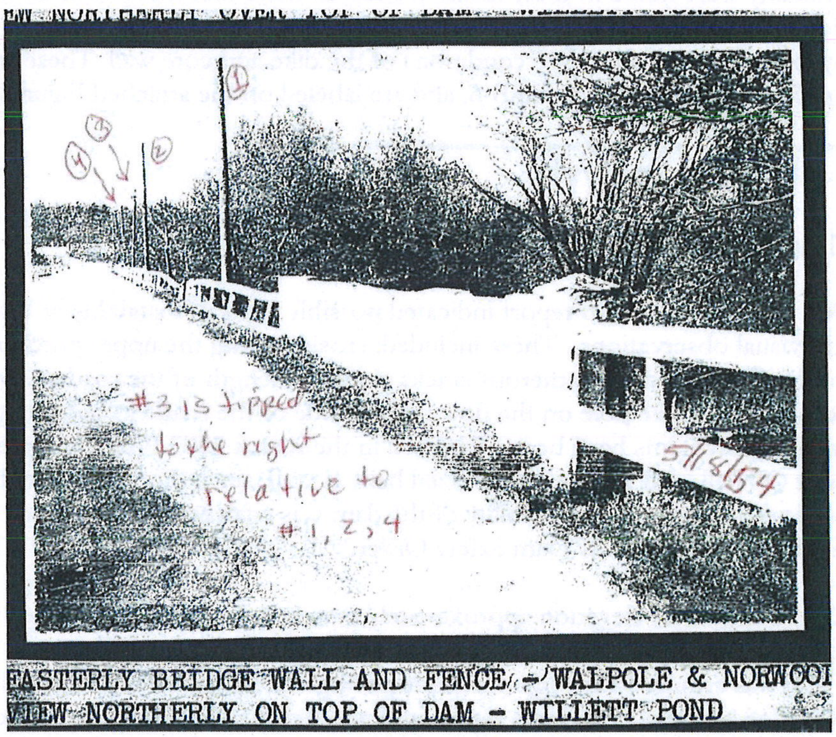
4.5.1 Visual Evidence

The 2007 Phase I inspection report indicated possible signs of instability at Willett Pond Dam based on visual observations. These included: erosion along the upper portion of the upstream slope of the dam and dike; numerous cracks along the length of the roadway on the dam crest; and a leaning telephone pole on the downstream side of the roadway indicating possible slope movement. These items have been addressed in the recent 2009 Phase I inspection performed by Fuss & O'Neill, and will be summarized here as well, since they can all be dispelled on the basis of visual inspection. The stability of the dam was analyzed in accordance with the requirements of the Phase II Dam Safety Order.

In 1913, the dam crest elevation approximately matched the crest elevation of the dike according to a plan filed with county commissioners. The upstream slope was approximately 2H:1V and was armored with smooth stone paving. Sketches from a 1979 National Program of Inspection for Non-Federal Dams report indicate that the dam crest was raised approximately two feet and a road was constructed across the crest. The road construction resulted in the uppermost portion of the upstream face having a slope of 1H:1V or steeper, out of necessity to maintain an adequate road width for two-way traffic. This area appears to be one of the areas of concern reported in the 2007 Phase I Inspection report. While the upstream face of the roadway base material located above the original dam crest appears to be eroding, the original dam itself appears to be in good condition with the original stone paving intact.

The pavement cracking on the crest of the dam is classic alligator cracking, likely caused by inadequate subgrade compaction during construction. The areas of longitudinal cracking on the dam appear to be precursors of alligator cracking due to inadequately compacted subgrade as well. Since the regulatory road width required for a two-way road appears to be close to the original dam crest width, the raised roadway had to be built with near vertical slopes of the base and subbase at the road edges. There was no room to widen the base and subbase to support the pavement. Therefore compaction efforts during road construction were likely limited, at the edges, leading to the existing conditions where the road base and subbase materials are sliding from beneath the pavement.

All evidence indicates that the telephone pole cited in the 2007 Phase 1 report as leaning was installed at approximately the current angle. The photo below shows the pole with a traffic sign circa 1954. In the photo, the sign is plumb and level, but at an angle to the axis of the pole, indicating that the pole was leaning when the sign was installed many years ago. Currently the sign is still level and plumb, and the pole is leaning at the same approximate angle. Adjacent guard rail shows no signs of movement, and the slope itself shows no signs of movement. Therefore there is no visual evidence of slope movement.



Power pole leaning in 1954

Note old rusted sign – plumb and level and
guard rail – plumb and straight

4.5.2 Stability Evaluation

Embankment stability analysis was performed using Galena Version 4.0 software and Bishop's Simplified methodology with circular failure surfaces. Global stability was evaluated using data collected from our survey data, boring data, groundwater elevations, and slug testing. Static stability was evaluated at full pond conditions. This dam has no capability of sudden drawdown, so stability of the upstream embankment was not considered. Seismic stability was also evaluated.

Parameters assigned to the soil layers included an angle of internal friction of the loose sand embankment soil of 32 degrees and a moist unit weight of 120 pounds per cubic feet (pcf). The underlying layer of weathered rock was assigned a unit weight of 140 pcf and friction angle of 40 degrees. The sound bedrock was treated as a confining layer through which no failure would occur. In addition, the concrete core wall was also treated as a no-failure boundary, and only failure surfaces initiating on the downstream side of the core wall were considered. To evaluate seismic stability, a pseudo-static lateral force factor of 0.05 was employed to add a lateral force to approximate the additional force resulting from earthquake shaking. The pseudo-static force factor was estimated based on the maximum acceleration of 0.10 g based on the 1.0 second earthquake duration.

The concrete core wall has a significant effect on stability. This is due in part to the inherent strength of the concrete interrupting potential failure surfaces forming through the embankment, and also by cutting off water flow through the embankment, which reduces potential seepage pressures in the down stream embankment to a negligible level. As discussed above, the core walls at the dam and dike appear to be functioning well based on piezometer readings and groundwater levels observed during drilling.

To confirm the stability of the dam using the data collected in the borings and piezometers, slope stability analysis was performed using Bishop's Simplified method of slices under full pond conditions. The results indicated the critical factors of safety occurred at shallow depths on the 2:1 slope. Figure B-3 in Appendix B shows a shallow sloughing failure with a factor of safety of 1.3. However, with good vegetative cover, 2:1 slopes generally perform satisfactorily against shallow sloughing, especially if groundwater seepage is not a concern as in this case. To test for global stability through the embankment toe area, where it is expected the most critical slope failure circle would be located, we forced the analysis to limit the failure circles through the area near the toe of the embankment. Results of stability modeling under this condition indicate a minimum factor of safety of 1.5 against failure under full pool conditions, as indicated in Appendix B Figure B-2. Applying a pseudo static force factor to the estimate of earthquake forces on the dam under full pond conditions indicated a critical factor of safety of 1.16, as shown in Appendix B Figure B-4. All the factors of safety are acceptable, especially considering there is no visual indications of instability are present.

Recent repair activity at the dam has resulted in slopes clear of woody vegetation and slopes have been seeded. We anticipate no problems with the dam embankment as long as maintenance is regularly performed. The road surface will likely continue to deteriorate due to its poor construction, but this should not affect embankment stability as long as future road repairs do not disturb the dam embankments or the underlying dam crest.

5 Evaluation of Repair Alternatives

5.1 Spillway Capacity Improvement Alternatives

All alternatives have the costs associated with bridge reconstruction, enlarging the cross sectional area of the downstream channel, and a relatively large spillway structure. The costs of these items are substantial, making the remaining factors more a consideration of ease of construction, or lack of disruption, rather than cost-related.

5.1.1 Alternative 1

Alternative 1 has the advantage of employing a combination of increasing the bridge opening area and adjusting the dike height to achieve the objective of accommodating the SDF. Some disruption of the residential properties abutting the dike would be necessary, but a raise in height of less than 2 feet is not an extreme measure.

5.1.2 Alternative 2

Alternative 2 also requires that the spillway be lowered to increase the bridge opening area, and to a depth 2 feet deeper than Alternative 1. However, in combination with lowering the water level in the lake by 2 feet, no construction would be necessary on the dike. The advantage of not having to raise the dike must be weighed against the impact to all the lakeside residences of having a shallower lake.

5.1.3 Alternative 3

Alternative 3 maintains the current normal pond level while avoiding the need to increase the height of the dike. However, this alternative would be very expensive, requiring a very large spillway, replacement of the bridge, and enlargement of the opening beneath the bridge. The attached schematic of this alternative shows one possible configuration. An ogee spillway of this size would be very expensive to build, considering the cost of water control, concrete forming and pouring, and the size of the spillway pad channeling flow to the outlet under the bridge. Other configurations could include extending the spillway out away from the dam into the pond more, or using a labyrinth configuration. However, it is likely these configurations would be more expensive than the configuration shown on the schematic.

5.1.4 Alternative 4

The gates present a lower initial cost than the concrete weir alternatives, and the tainter gate and vertical lift gate are the least expensive of the gate types. However, there are additional monetary costs to be considered as well as non-monetary costs associated with floodgates.

The hinged bottom gate, despite being more costly than the other gate types, is attractive due to the fact that it allows easier control of lake water levels, and it can be automated more easily due to the fact it operates as a type of weir. The hinged bottom gate also does not have the vertical superstructure that the other gates require and will be less visually obtrusive.

One non-monetary cost to be considered with the use of gates is the responsibility of operation of the gates. Like all mechanical equipment, gates could fail to operate properly, and there is the possibility of harm to down stream residents and infrastructure should this happen. It makes sense that an organization with the financial ability to retain maintenance and repair personnel, as well as operations personnel would be more suited to owning and operating a mechanical gate structure. Perhaps a maintenance agreement could be entered into with an entity with the appropriate staff.

Despite the increased cost of purchasing and installing the hinged-bottom gate, the benefits appear to outweigh the monetary costs in ease of operation and maintenance compared to the other gate types. Compared to the total cost of rehabilitating the dam, the costs between gate types are relatively minor and should not be the sole determining criterion for choosing a gate. We feel the relative ease of operation, maintenance, and ability to control lake levels more precisely with the hinged gate outweighs the capital cost differences.

6 Other Considerations

6.1 Embankment Repair

Recent clearing and reseeding of the downstream embankment has been performed by NRLHA (except for property owned by Norfolk County). No additional repair appears to be necessary at this time. NRLHA has also repaired some minor erosion gullies from road run-off on the upgradient embankment. No additional work other than routine inspection and maintenance is necessary at this time.

It should be noted again that the road and its base material, which were constructed on top of the dam crest, are not the responsibility of the NRLHA, but are the responsibility of the Massachusetts Highway Department, as the road is not part of the original dam, nor is it necessary for the safe operation of the dam.

The dike has been modified on the down gradient side in several locations. Most of these areas are stabilized, but there is still ongoing construction of steps and other structures. The NRLHA has notified the residents along the dike that no more modifications of the dike can be made, and no additional plantings of woody vegetation will be permitted on the dike. We recommend that any areas of disturbance on the dike crest or embankments be stabilized with non-woody vegetation.

6.2 Seepage Control

Seepage is not currently a major concern at the dam. Suspected seepage in the gate house has been determined to be backwater from the down gradient wetland. The wetland also extends to the toe of the dam, but no instability has been noted at the dam toe, probably due to the presence of the core wall that reduces the pore water pressures within the dam embankment, and thus increases the stability.

There are areas with apparent seepage on the down gradient side of the dike. These areas do not appear to be causing instability of the dike, but should be monitored for increases in flow or the presence of soil being carried through the dike. Again, the core wall in the dike appears to be working well and these areas could represent minor seepage beneath the core wall, or low areas where water naturally collects behind the dike.

6.3 Property Considerations

6.3.1 Dam

A large portion of the down gradient left abutment slope is not owned or controlled by the NRLHA, but is owned by Norfolk County. The presence of large trees within 20 feet of the abutment is considered a deficiency under the Dam Safety Regulations. However, NRLHA has no right or obligation to perform maintenance on County owned property. The Massachusetts Highway Department (MHD) recently rebuilt the Bullard Street Bridge, work that included installing new wing walls at the spillway, rebuilding a portion of the spillway walls (aka bridge abutments) and removing portions of the intermediate pier walls that divide the spillway in three. MHD executed an emergency taking order before performing this work. However, now that the bridge work is complete, MHD has failed to respond to repeated requests by the NRLHA that they finalize the taking order and specify exactly what was taken. For the time being, the NRLHA is operating under the presumption that it does not own the bridge, the wing walls added to the spillway by MHD, the portion of the spillway sidewalls that were demolished and rebuilt by MHD, or the portion of the dam embankment behind the spillway walls that (aka bridge abutments) that were excavated and reconstructed by MHD.

6.3.2 Dike

The NRLHA owns the dike as far as the down gradient toe. Individual residences own the property down gradient of the toe. There has been good cooperation among the property owners to clear trees within twenty feet of the embankment toe. However, it should be noted that NRLHA has no control over the maintenance activities on abutter's properties. Also, many of the property owners have filled against the down gradient dike embankment to expand their yards and ease the slope steepness for access to the pond. This has made it difficult to distinguish where the toe of the embankment is located. It also means the dike has been buttressed and may result in increased stability of the dike where fill has been placed. Some permanent structures have been constructed on the down gradient embankment, such as steps and stairs, to access the pond. There are also a few small sand beaches with no vegetation that have been created that could be a source of erosion.

We recommend that the dike be cleared of trees to the extent possible and areas with no or thin vegetation be seeded with grass. The upstream embankment should be inspected for missing or mis-aligned riprap and repaired where applicable.

7 Anticipated Permits Required for Construction

7.1 Local Permits

- *Order of Conditions to be issued by the Town of Norwood and Town of Walpole Conservation Commissions*

  This permit is required for work within wetlands and the wetland buffer downstream of the Dam and within Willett Pond and the Willett Pond wetland buffer.

7.2 State Permits

- *MGL Chapter 253 - Dam Safety Permit issued by Massachusetts DCR*

  This permit is required for the construction, repair, material alterations, breach, or removal of a dam.
- *Clean Water Act Section 401 Water Quality Certification issued by Massachusetts DEP*

This permit is required for projects that involve more than 100 cubic yards of dredging and management and disposal of dredged materials, and for disturbance of more than 5,000 square feet of bordering or isolated wetlands or land under water, and for state review of an activity that requires a permit from the Army Corps of Engineers related to wetlands and waterways. Alternative one and three are likely to trigger the dredging threshold. Alternative two is also likely to trigger the BVW threshold by permanently lowering the water level.

- *Massachusetts Environmental Policy Act Environmental Notification Form*

  This review process must be undertaken for projects that involve alteration of more than 5,000 square feet of bordering or isolated vegetated wetlands or ½ acre of other wetlands (such as land under water). Please note that a mandatory Environmental Impact Report would be required if construction would increase the impoundment capacity by 20 percent or more or cause any decrease in the impoundment capacity. Alternatives one and three may trigger an ENF requirement for alteration of land under water. Alternative two would trigger a mandatory EIR.

7.3 Federal Permits

- *Clean Water Act Section 404 Programmatic General Permit issued by the Army Corps of Engineers*

  Required for excavation and dredging in wetlands and waterways. The category of the permit will depend on quantity of disturbance, and may trigger a state 401 permit.

8 Summary

8.1 Stability

The dam and dike both appear substantially stable with no major rehabilitation effort required. Minor seepage should be monitored in a few locations along the dike. Analyses discussed earlier indicate the core walls are functioning and seepage and slope stability are satisfactory.

8.2 Monitoring and Maintenance

The NRLHA is committed to maintaining the dam and dike in compliance with the Dam Safety Regulations. An EAP has been prepared as well as an Operation and Maintenance Manual. The NRLHA has done extensive clearing, stabilization and reseeding during the past year, as well as complying with inspection requirements. They interceded when they saw incorrect construction methods being applied to the reconstruction of the bridge over the spillway, which resulted in correction of the construction in accordance with dam safety design practices.

Some areas are not within the control of NRLHA and should be resolved by the parties involved. These include the responsibility of the owners of abutting properties along the dam and dike to keep the areas within 20 feet of the dam and abutments clear of woody vegetation. Another concern is the poor condition of the road on top of the dam. This road is failing and attempts to correct road problems could negatively impact the dam. NRLHA is not responsible for road maintenance or repair, but road maintenance and repair must comply with dam safety regulations.

8.3 Opinion of Construction Costs

An order-of-magnitude estimate of construction cost for each alternative has been prepared for planning purposes. An order-of magnitude cost estimate is assumed to be accurate to within a range of -30% to +50% of the estimated value, and are based on our experience with construction projects of a similar magnitude, as well as some more detailed estimates of quantities and unit costs. Appendix D includes cost estimation sheets for the three alternatives considered here. As can be seen, there are no inexpensive alternatives for bringing this dam up to dam safety standards. The estimates for each fixed concrete spillway alternative range from roughly $600,000 to $5,700,000.

8.3.1 Long Spillway Alternative Cost

Fuss & O'Neill subcontracted with New England Infrastructure, Inc. (NEI) to perform a more detailed cost estimate of Alternative 3, which consists of constructing a long spillway (370 feet) to limit the height of the water over the spillway to below the elevation of the dam and dike crests during the Spillway Design Flood (SDF). This alternative would allow the lake to be maintained at its current normal elevation without increasing the elevation of the dike to prevent overtopping during the SDF.

Fuss & O'Neill was originally tasked with preparing opinions of cost for the various spillway alternatives at a very broad order of magnitude. Order-of-magnitude estimates are typically based on knowledge of similar construction efforts through previous experience. Our estimates were based on experience as well as some itemization and research into construction materials and tasks that we are fairly confident would be included in the design of each alternative. The cost estimates prepared were a little more detailed than typical order of magnitude estimates, but should still be considered broad in range. It is difficult at this time to make estimates much more detailed than this due to the lack of detail at this conceptual level of design; there are no design plans to reference for a detailed construction cost estimate. However, contractors such as NEI who are experienced in work with dams and structures in water environments, and with access to the most current cost data, are able to provide somewhat more detailed estimates, although they are also constrained by the lack of detail available in the conceptual drawings. It should be noted that NEI's cost estimating was confined to the construction aspects of the Long Spillway alternative. Additional cost items affecting the overall project cost, such as permitting and engineering costs, were provided by Fuss & O'Neill.

The long spillway alternative is an attractive alternative because it is the least disruptive of the fixed concrete spillway alternatives to residents living on the lake shore. It is also a relatively expensive alternative, due primarily to the size of the concrete weir structure and the need to provide water control in deep water over a large area during construction. NRLHA requested

Alternative 3 be the focus of the cost estimate performed by NEI in conjunction with Fuss & O'Neill.

During the conceptual review of this alternative, it was decided to locate the proposed concrete ogee weir in a perpendicular direction to the long axis of the dam. This takes advantage of shallow bedrock in the weir location that would reduce the quantity of concrete required to construct the weir. It likely means less sediment will need to be excavated, and water control in shallower water will be less expensive.

The attached Order of Magnitude opinion of cost shows that the base cost would be $3,234,000. The range of costs for this Alternative 3 is $2,260,000 to $4,850,000. This broad range (-30% to +50%) is typical for order of magnitude costs, and is based on some assumptions, the validity of which are uncertain at this time. The attached spreadsheet has been organized to separate up stream costs from down steam costs, with the work required beneath the bridge part of the down stream costs.

In an effort to look at the costs associated with different uncertainties, the second table at the bottom of the attached spreadsheet tabulates the change in the base cost estimate if some of the assumptions turn out to be invalid. For instance, the base cost assumes that the sediment in the pond will not need to be removed and disposed of at a lined landfill, either because it is not contaminated with organic chemicals or heavy metals, or because the sediment could be redistributed in the lake and stabilized. The small table at the bottom accounts for the possibility that sediment is contaminated and needs to be removed. As can be seen, contaminated sediment is an expensive item. Both the quantity of sediment and the presence of contaminants are uncertain at this time.

Likewise, water control during construction is a relatively high cost item. The ability to draw down the lake during construction has a relatively large effect on the total cost. The more the lake can be drawn down, the easier and less expensive the control of water would become. In addition to comparing three high cost items alone, the table also examines combinations of the items.

8.4 Preferred Alternative

At this time the NRLHA is unable to declare a specific preferred alternative, though the three alternatives that were examined in depth appear to represent the best balance of costs and benefits.

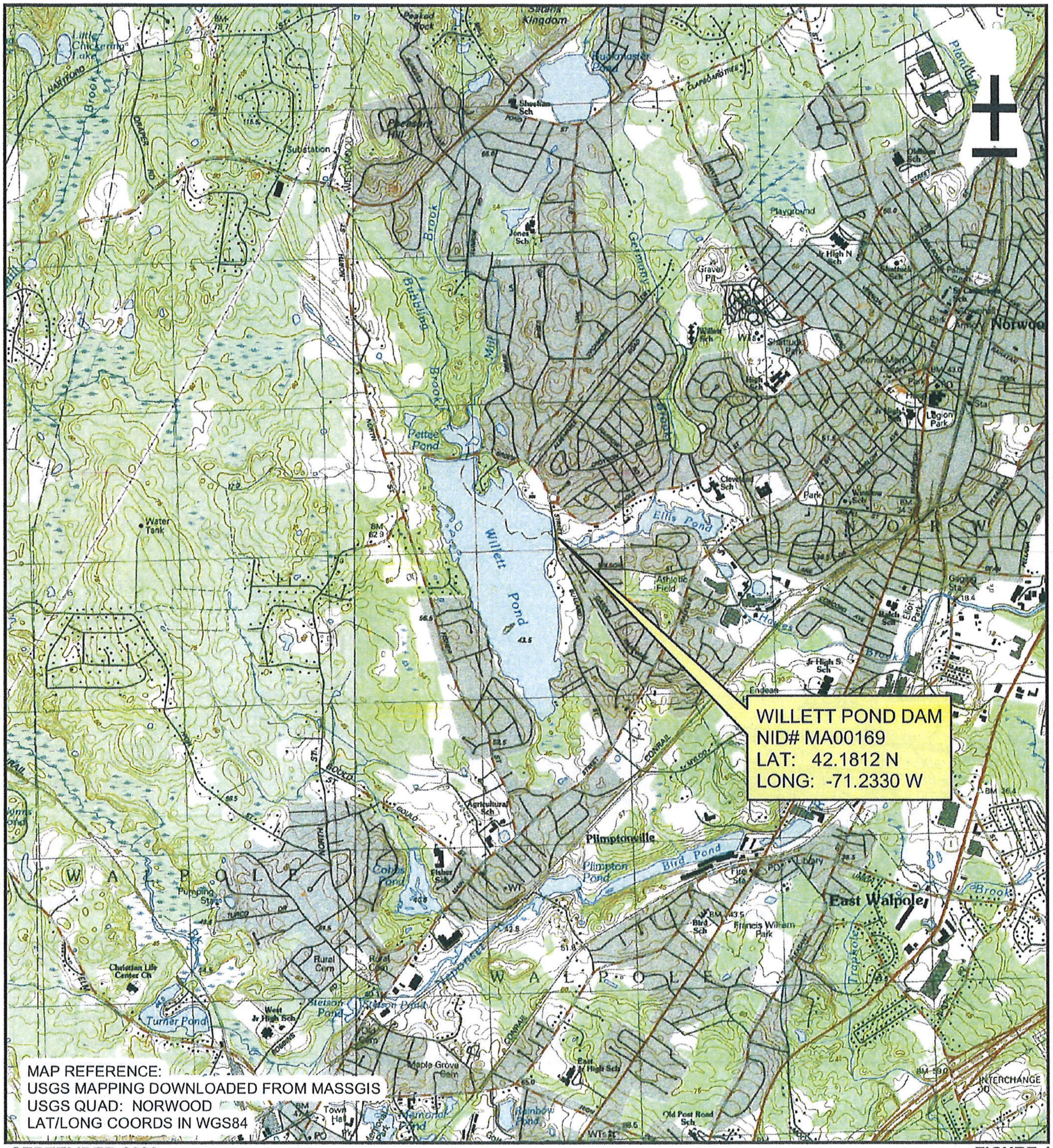
All of the feasible alternatives for improving the spillway capacity would involve closing Bullard Street, demolishing the recently reconstructed bridge, deepening the spillway, rebuilding the bridge abutments and rebuilding the bridge. As discussed above, the NRLHA does not own the bridge and appurtenant structures which are also integral components of the overall spillway structure. Furthermore, the bridge was recently reconstructed at considerable expense and effort to the Commonwealth under the auspices of special legislation which was specifically designed to circumvent Office of Dam Safety rules which would have otherwise required MHD to bring the spillway into compliance with the Dam Safety Regulations as a condition of replacing the bridge.

In addition to the fact that the NRLHA lacks sufficient property rights to implement any of the feasible spillway remediation alternatives, the NRLHA also lacks the financial resources to implement any of the feasible alternatives on its own.

Now that the Phase II investigation has been completed, the NRLHA intends to work with the Office of Dam Safety, local and state officials, legislators and abutters from around the pond in an attempt to resolve property rights conflicts, secure consensus from key stakeholders on a preferred alternative, and to secure the funding needed to implement that preferred alternative.

FUSS & O'NEILL

Figures



SEPTEMBER 2009

FIGURE 1

NEPONSET RIVER LAND HOLDING ASSOCIATION

3,000 1,500 0 3,000 Feet

SCALE
HORZ: 1 INCH = 3,000 FEET
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DATUM
HORZ:
VERT: NGVD29 (3-METER CONTOURS)

LOCUS MAP

WILLETT POND DAM (MA00169)

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NORWOOD/WALPOLE, MASSACHUSETTS

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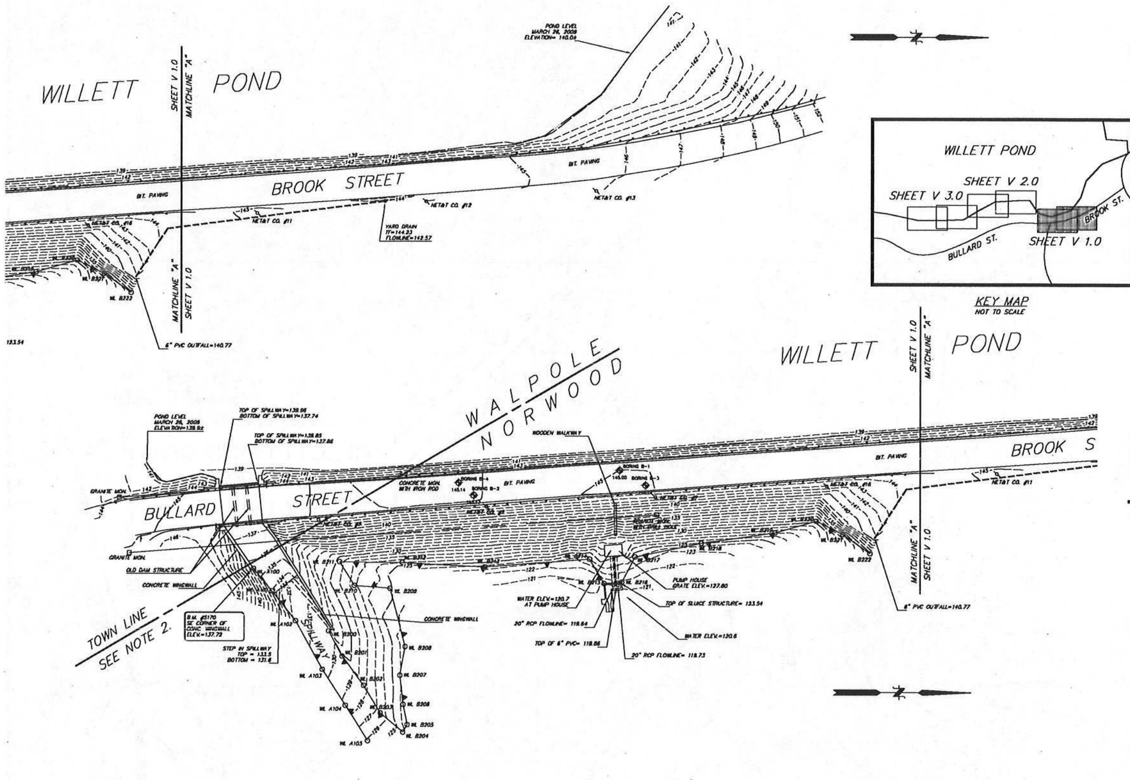

KEY MAP
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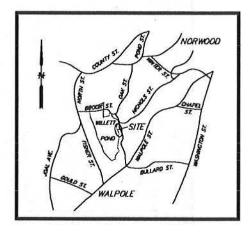

LOCUS MAP
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LEGEND

UTILITY POLE			PROPERTY LINE
BOLLARD			EDGE OF WATER
SIGN			WETLANDS
SOIL BORING			EXISTING CONTOUR
DECIDUOUS TREE			INDEX CONTOUR
EXISTING MONUMENT			STOCKADE FENCE
LIGHT			GUIDE RAIL
WETLANDS FLAG			STORM DRAINAGE PIPE
BORING B-3 145.14	BORING LOCATION AND SURFACE ELEVATION		

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MAP REFERENCES:

1. "MAP OF LAKE AND VICINITY, WINSLOW BROS. AND SMITH CO., NORWOOD, WESTWOOD AND WALPOLE, MASSACHUSETTS," PREPARED BY A. L. MADDOX ENGINEER, FEBRUARY 1912.

2. PLAN SHOWING LAYOUT OF BROOK STREET & BULLARD STREET IN THE VICINITY OF THE TOWN LINE BETWEEN NORWOOD AND WALPOLE, MASSACHUSETTS. SHEET NO. 3 OF 3, RECORDED IN THE NORFOLK REGISTRY OF DEEDS, DEEDHAM, MASSACHUSTETTS FILED AS 213 1967, PLAT BOOK 222.

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WILLETT POND DAM & DIKE
PHASE II INVESTIGATION
NORWOOD/ WALPOLE MASSACHUSETTS

PROJ. No.: 20061323A20
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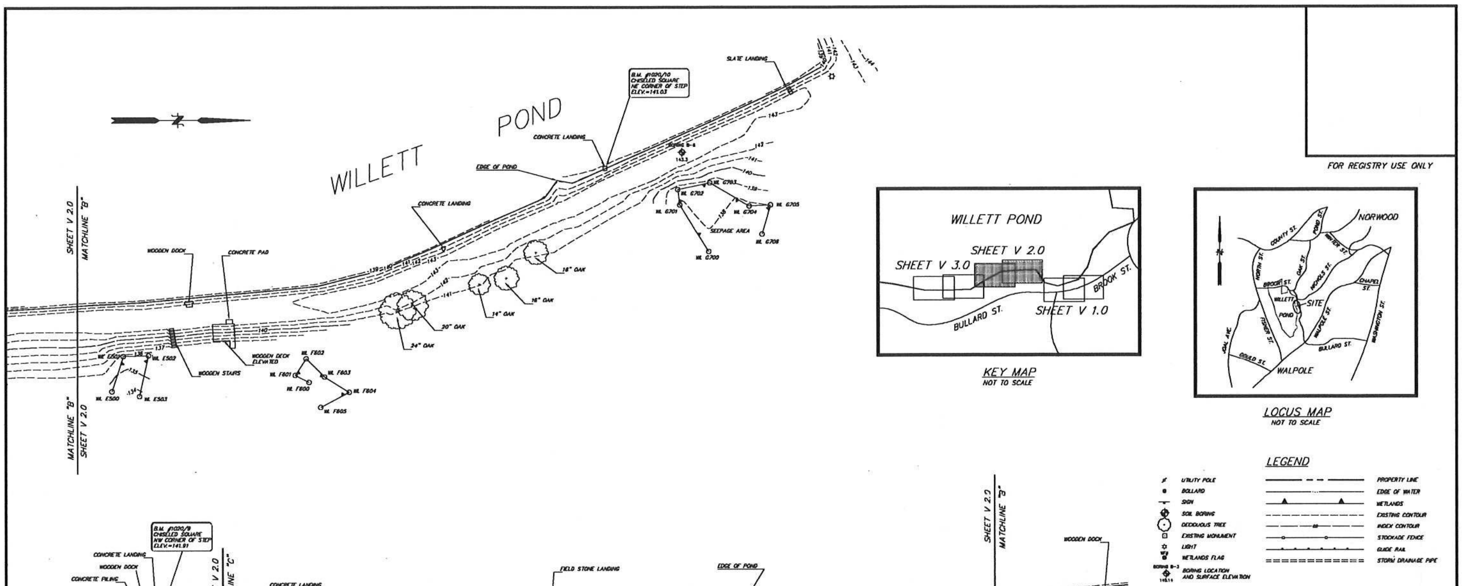
FIG. 2A



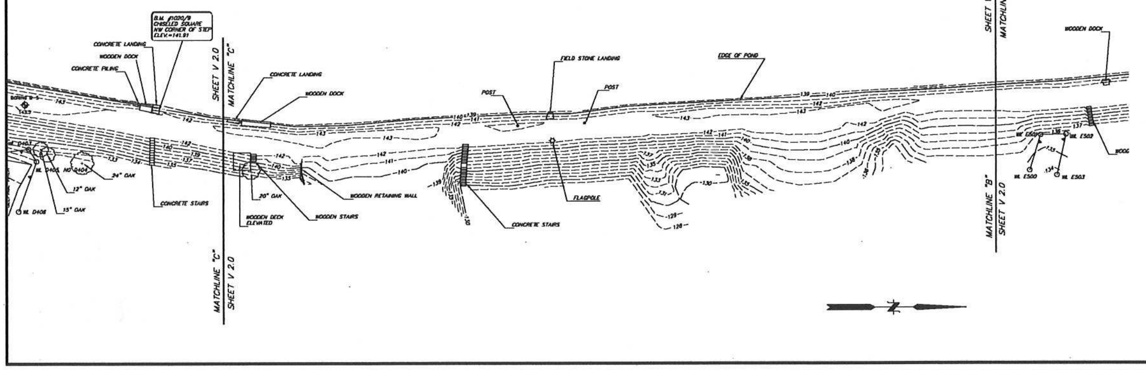


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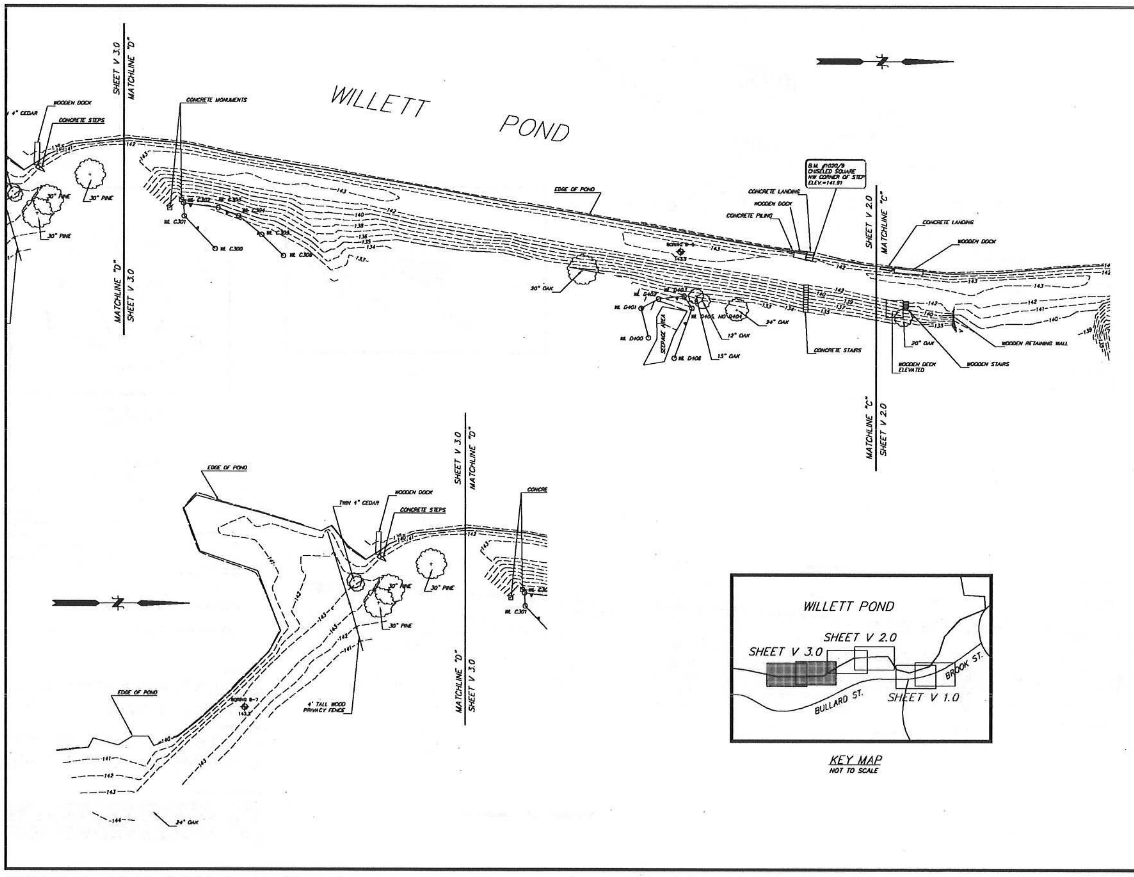
FIG. 2B



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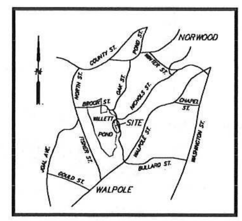

LOCUS MAP
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LEGEND

	UTILITY POLE		PROPERTY LINE
	BOLLARD		EDGE OF WATER
	SIGN		WETLANDS
	SOIL BORING		EXISTING CONTOUR
	DECIDUOUS TREE		INDEX CONTOUR
	EXISTING MONUMENT		STOCKADE FENCE
	LIGHT		GUIDE RAIL
	WETLANDS FLAG		STORM DRAINAGE PIPE
BORING B-3	BORING LOCATION AND SURFACE ELEVATION		

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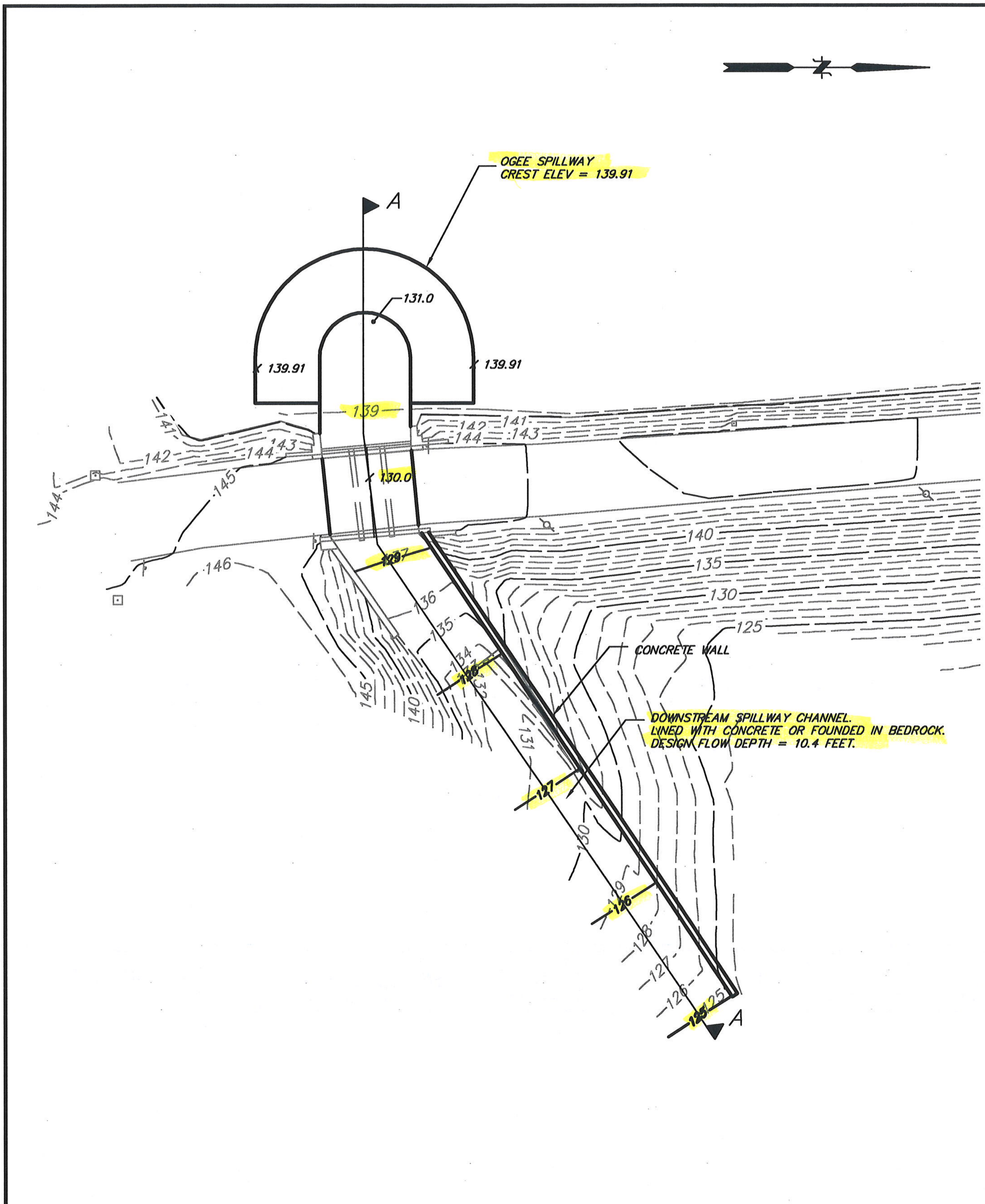
FIG. 2C



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ALTERNATIVE 1
PROPOSED SPILLWAY MODIFICATIONS
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NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323.A20
DATE: NOVEMBER 2009

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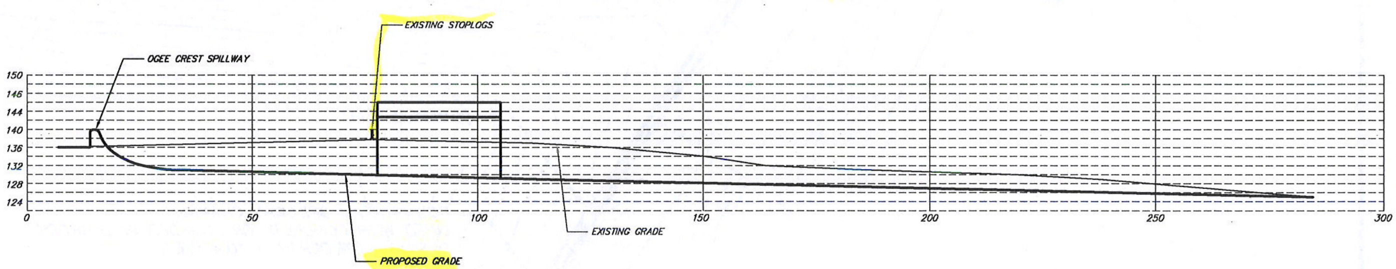
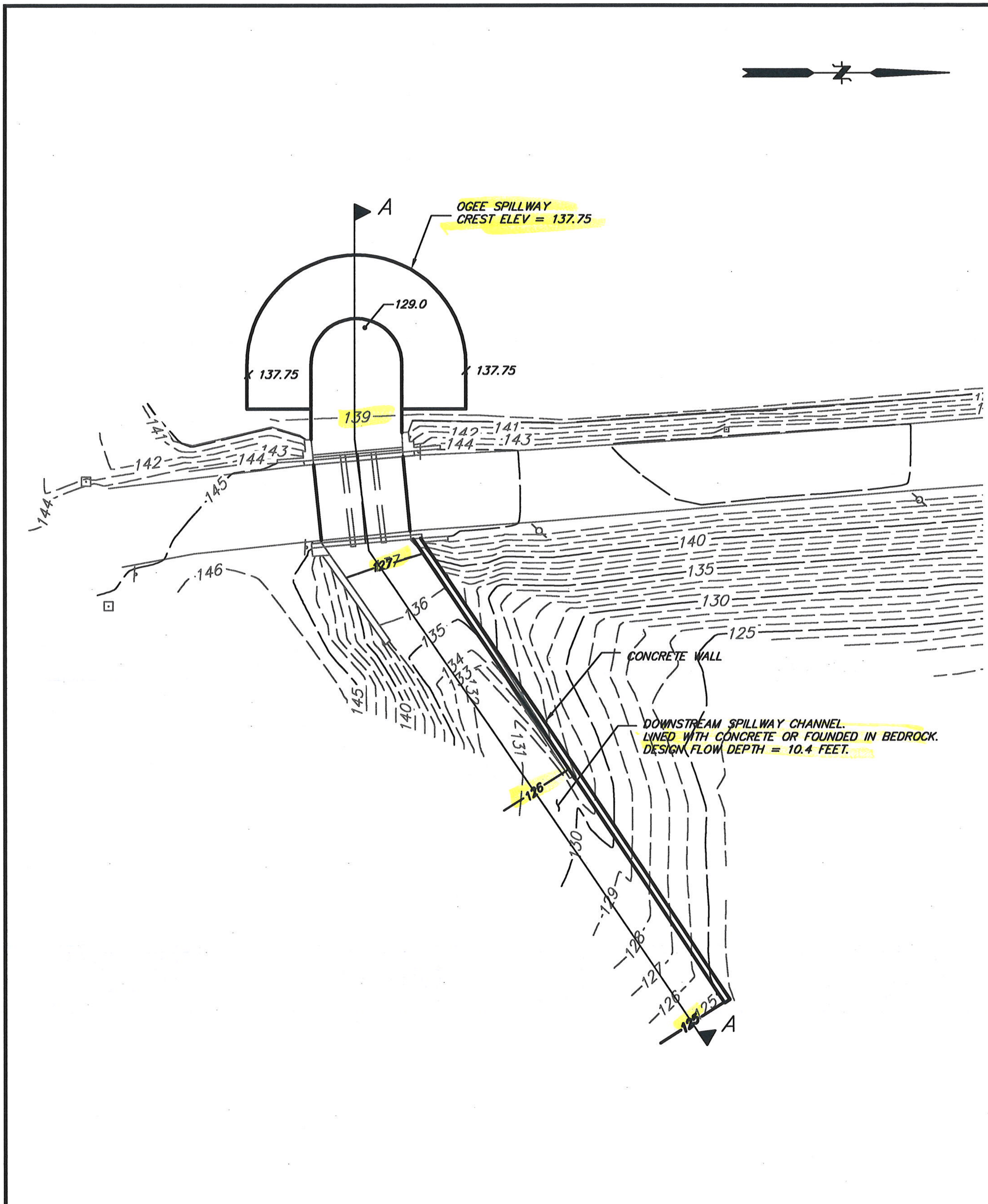


SECTION A – A

1				PROJ MANAGER:	SCALE: HORZ.: 1" = 20' VERT.: 1" = 20'	WWW.FandO.COM	NEPONSET RIVER LAND HOLDING ASSOCIATION	PROJ No.: 20051323.A20
No.	DATE	DESCRIPTION	BY	CHIEF DESIGNER:	DATUM: HORZ.: VERT.: NAVD 88	FUSS & O'NEILL *Disciplines to Deliver*	ALTERNATIVE 1 - SPILLWAY PROFILE SPILLWAY ALTERNATIVE ASSESSMENT	DATE: NOVEMBER 2009
REVISIONS				REVIEWED BY: DATE	0 10 20 GRAPHIC SCALE	78 INTERSTATE DR WEST SPRINGFIELD, MA 01089 413.452.0445	WILLETT POND DAM PHASE II INVESTIGATION NORWOOD/WALPOLE MASSACHUSETTS	3B



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UCS: | MS VIEW: | LMAN: | CTB:

SCALE:	
HORZ.:	1" = 40'
VERT.:	
DATUM:	
HORZ.:	
VERT.:	NAVD 88

0 20 40
GRAPHIC SCALE

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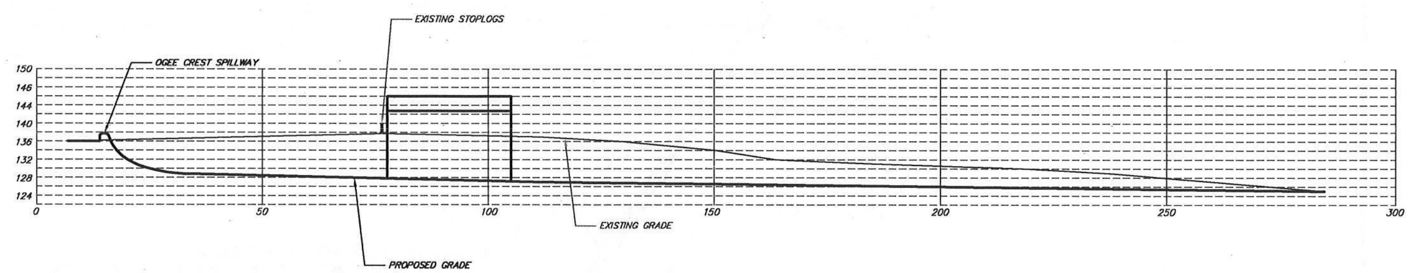
FUSS & O'NEILL
Disciplines to Deliver
78 INTERSTATE DR WEST SPRINGFIELD, MA 01089 413.452.0445

NEPONSET RIVER LAND HOLDING ASSOCIATION
ALTERNATIVE 2
PROPOSED SPILLWAY MODIFICATIONS
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323.A20
DATE: NOVEMBER 2009

4A



SECTION A – A

1				
No.	DATE	DESCRIPTION	BY	
REVISIONS				

PROJ MANAGER:
CHIEF DESIGNER:
REVIEWED BY: DATE

SCALE:
HORZ.: 1" = 20'
VERT.: 1" = 20'
DATUM:
HORZ.:
VERT.: NAVD 88
0 10 20
GRAPHIC SCALE

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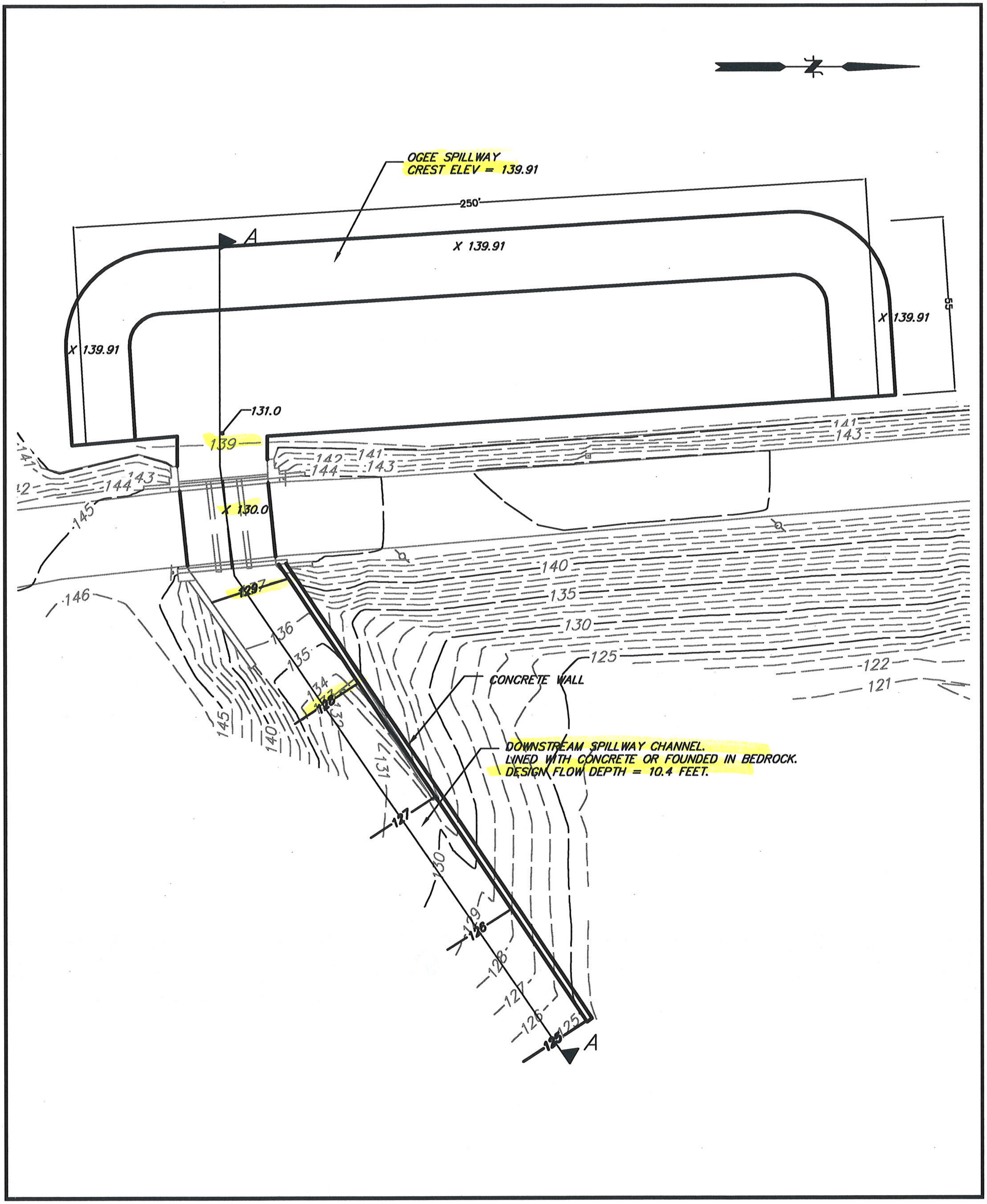
78 INTERSTATE DR WEST SPRINGFIELD, MA 01089 413.452.0445

NEPONSET RIVER LAND HOLDING ASSOCIATION
ALTERNATIVE 2 - SPILLWAY PROFILE
SPILLWAY ALTERNATIVE ASSESSMENT
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

PROJ No.: 20051323.A20
DATE: NOVEMBER 2009

4B



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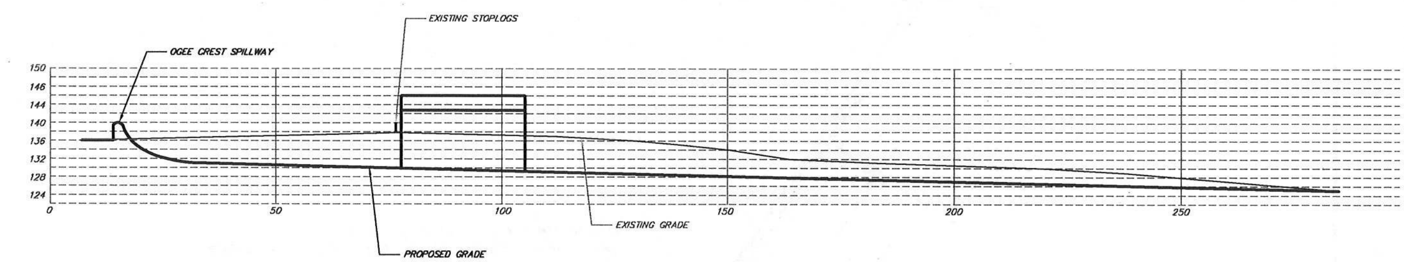
UCS: | MS VIEW: | LMAN: | CTB:

SCALE: HORZ.: 1" = 40' VERT.: DATUM: HORZ.: VERT.: NAVD 88 0 20 40 GRAPHIC SCALE	FUSS & O'NEILL *Disciplines to Deliver* WWW.FandO.COM 78 INTERSTATE DR WEST SPRINGFIELD, MA 01089 413.452.0445	NEPONSET RIVER LAND HOLDING ASSOCIATION ALTERNATIVE 3 PROPOSED SPILLWAY MODIFICATIONS WILLETT POND DAM PHASE II INVESTIGATION NORWOOD/WALPOLE MASSACHUSETTS	PROJ. No.: 20051323.A20 DATE: NOVEMBER 2009 5A



SECTION A – A

No.	DATE	DESCRIPTION	BY	
1				
REVISIONS				

PROJ MANAGER:
CHIEF DESIGNER:
REVIEWED BY: DATE

SCALE:
HORZ.: 1" = 20'
VERT.: 1" = 20'
DATUM:
HORZ.:
VERT.: NAVD 88
0 10 20
GRAPHIC SCALE

FUSS & O'NEILL
Disciplines to Deliver
WWW.FandO.COM
78 INTERSTATE DR WEST SPRINGFIELD, MA 01089 413.452.0445

NEPONSET RIVER LAND HOLDING ASSOCIATION
ALTERNATIVE 3 - SPILLWAY PROFILE
SPILLWAY ALTERNATIVE ASSESSMENT
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

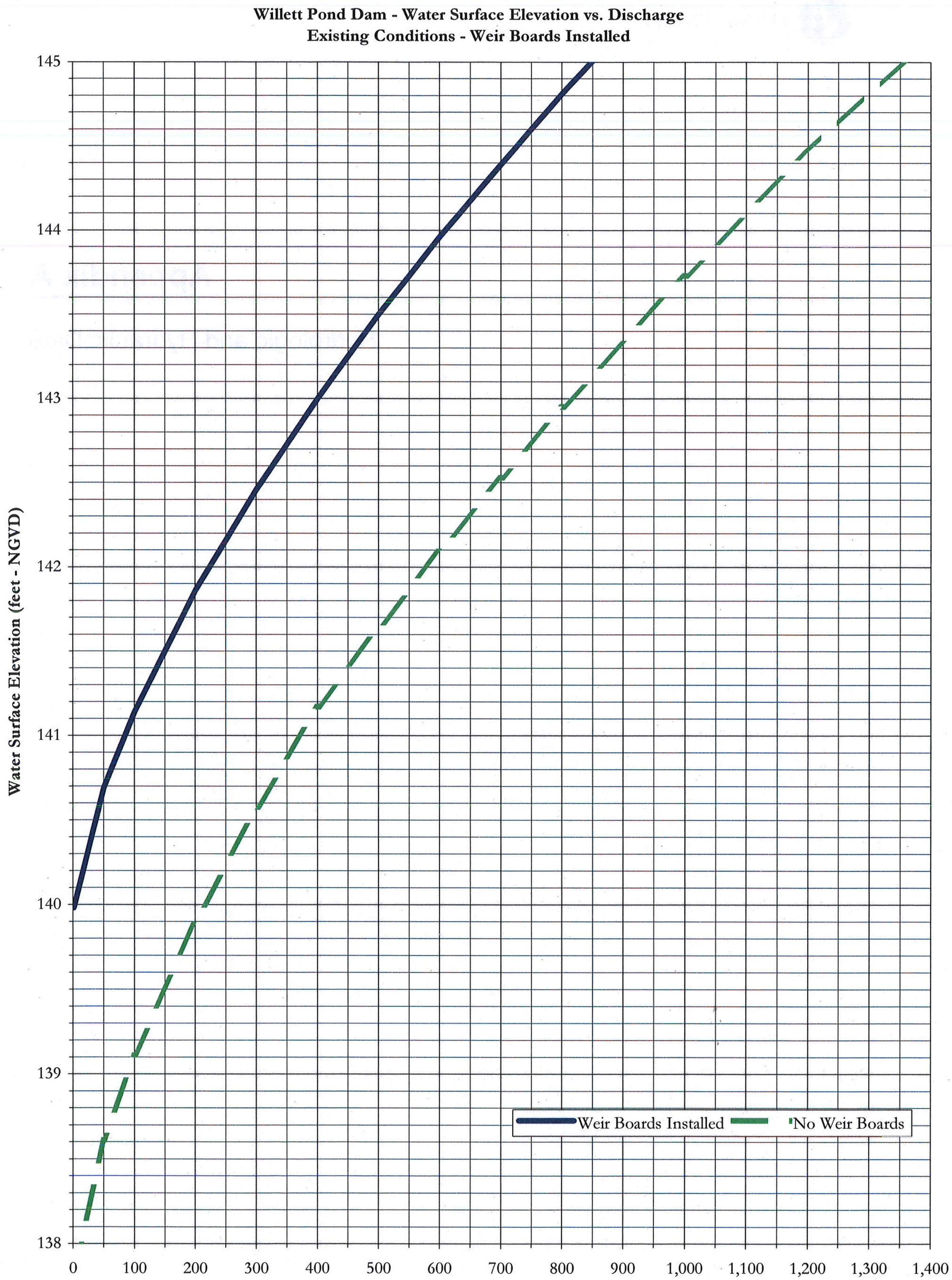
PROJ No.: 20051323.A20
DATE: NOVEMBER 2009

5B

FUSS & O'NEILL

Appendix A

Hydrologic and Hydraulic Data



Willett Pond Dam - Water Surface Elevation vs. Discharge
Existing Conditions - Weir Boards Installed
Water Surface Elevation (feet - NGVD)
145
144
143
142
141
140
139
138
0
100
200
300
400
500
600
700
800
900
1,000
1,100
1,200
1,300
1,400
Discharge (cfs)
Weir Boards Installed
No Weir Boards

HEC-RAS Version 4.0.0 March 2008
U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California

```
X     X  XXXXXX    XXXX          XXXX       XX      XXXX
X     X  X        X    X         X   X    X  X     X
X     X  X        X              X   X   X    X    X
XXXXXXX  XXXX     X        XXX   XXXX    XXXXXX     XXXX
X     X  X        X              X  X    X    X         X
X     X  X        X    X         X   X   X    X         X
X     X  XXXXXX    XXXX          X    X  X    X    XXXXX
```

****

PROJECT DATA
Project Title: Willett Pond Dam - Existing Spillway
Project File : Willett-ExistSpi.prj
Run Date and Time: 3/2/2010 8:49:16 AM

Project in English units

****

PLAN DATA

Plan Title: No Weir Boards
Plan File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.p01

Geometry Title: Existing Spillway - No Weir Boards
Geometry File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.g01

Flow Title : Steady Flow
Flow File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.f01

Plan Summary Information:

Number of:				
Cross Sections	= 14		Multiple Openings	= 0
Culverts	= 0		Inline Structures	= 0
Bridges	= 0		Lateral Structures	= 0

Computational Information

Water surface calculation tolerance	= 0.01
Critical depth calculation tolerance	= 0.01
Maximum number of iterations	= 20
Maximum difference tolerance	= 0.3
Flow tolerance factor	= 0.001

Computation Options
Critical depth computed at all cross sections
Conveyance Calculation Method: At breaks in n values only
Friction Slope Method: Program Selects Appropriate method
Computational Flow Regime: Subcritical Flow

****

FLOW DATA

Flow Title: Steady Flow
Flow File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.f01

Flow Data (cfs)

River	Reach	RS	PF 1	PF 2	PF 3	PF 4	PF 5	PF 6	PF 7	PF 8
Willett	10	1100	1	50	100	200	300	400	500	600

River	Reach	RS	PF 9	PF 10	PF 11	PF 12	PF 13	PF 14	PF 15	PF 16
Willett	10	1100	700	800	1000	1200	1400	1500	1600	1800

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
Willett	10	PF 1		Normal S = 0.033
Willett	10	PF 2		Normal S = 0.033
Willett	10	PF 3		Normal S = 0.033
Willett	10	PF 4		Normal S = 0.033
Willett	10	PF 5		Normal S = 0.033
Willett	10	PF 6		Normal S = 0.033
Willett	10	PF 7		Normal S = 0.033
Willett	10	PF 8		Normal S = 0.033
Willett	10	PF 9		Normal S = 0.033
Willett	10	PF 10		Normal S = 0.033

****

GEOMETRY DATA

Geometry Title: Existing Spillway - No Weir Boards
Geometry File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.g01

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 1100

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
1000	150	1000	137.25	1500	137.25	1500	150

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
1000	.035	1000	.035	1500	.035

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1500	10	10	10	.3	.5

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 1080.*

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
1000	150	1000	137.35	1500	137.35	1500	150

Manning's n Values num= 2

Sta	n Val	Sta	n Val
1000	.035	1500	.035

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1500	10	10	10	.3	.5

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 1060.*

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
1000	150	1000	137.45	1500	137.45	1500	150

Manning's n Values num= 2

Sta	n Val	Sta	n Val
1000	.035	1500	.035

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1500	10	10	10	.3	.5

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 1040.*

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
1000	150	1000	137.55	1500	137.55	1500	150

Manning's n Values num= 2

Sta	n Val	Sta	n Val
1000	.035	1500	.035

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1500	10	10	10	.3	.5

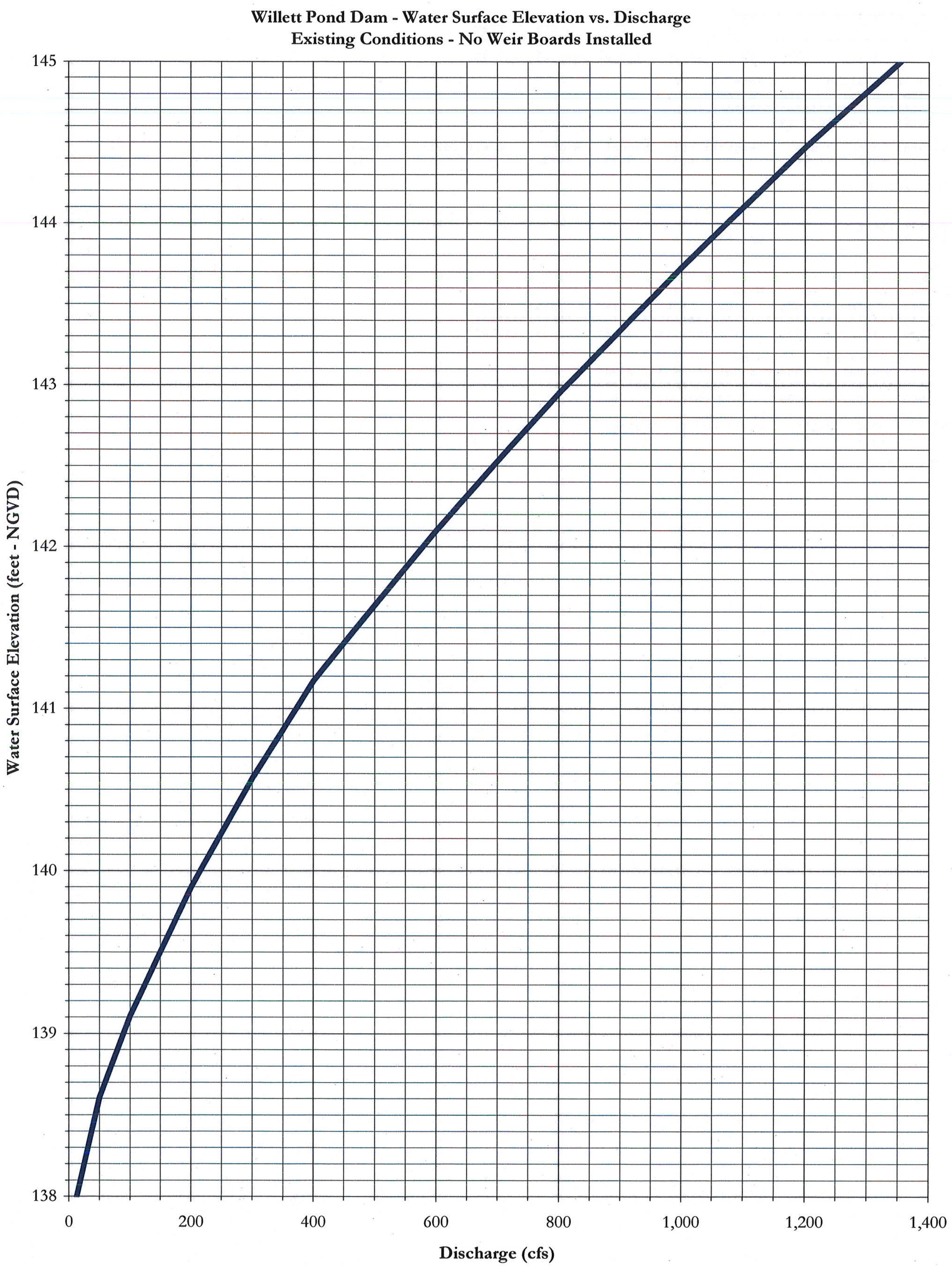
CROSS SECTION

RIVER: Willett
REACH: 10 RS: 1020.*

```
INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.65    1500  137.65    1500     150

Manning's n Values        num=       2
     Sta   n Val     Sta   n Val
********************************
    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500               10       10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1000

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.75    1500  137.75    1500     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .035    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500                2        2       2              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 990

INPUT
Description:
Station Elevation Data    num=      12
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
********************************************************************************
    1000     150    1000  137.75 1007.88  137.75 1007.88  139.91 1009.88  139.91
 1009.88  137.75 1017.76  137.75 1017.76  139.91 1019.76  139.91 1019.76  137.75
 1027.64  137.75 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .012    1000    .012 1027.64    .012

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64             9.33     9.33    9.33              .1       .3
Cross Section Lid
    num=       2
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
************************************************
    1000     150  142.75 1027.64     150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 960.*

INPUT
Description:
Station Elevation Data    num=      12
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
********************************************************************************
    1000     150    1000  137.56 1007.88  137.56 1007.88  139.86 1009.88  139.86
 1009.88  137.56 1017.76  137.56 1017.76  139.86 1019.76  139.86 1019.76  137.56
 1027.64  137.56 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .012    1000    .012 1027.64

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64             9.33     9.33    9.33              .1       .3
Cross Section Lid
    num=       2
     Sta Hi Cord Lo Cord     Sta Hi Cord Lo Cord
************************************************
    1000     150  142.75 1027.64     150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 930.*
```

```
INPUT
Description:
Station Elevation Data    num=      12
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    1000      150     1000   137.36  1007.88   137.36  1007.88   139.82  1009.88   139.82
 1009.88   137.36  1017.76   137.36  1017.76   139.82  1019.76   139.82  1019.76   137.36
 1027.64   137.36  1027.64      150

Manning's n Values        num=       3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012  1027.64

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64              9.33    9.33    9.33                .1       .3
Cross Section Lid
    num=       2
     Sta Hi Cord Lo Cord      Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75  1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 900

INPUT
Description:
Station Elevation Data    num=      12
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    1000      150     1000   137.17  1007.88   137.17  1007.88   139.77  1009.88   139.77
 1009.88   137.17  1017.76   137.17  1017.76   139.77  1019.76   139.77  1019.76   137.17
 1027.64   137.17  1027.64      150

Manning's n Values        num=       3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012  1027.64     .012

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64                 3       3       3                .1       .3
Cross Section Lid
    num=       2
     Sta Hi Cord Lo Cord      Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75  1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 890

INPUT
Description:
Station Elevation Data    num=       4
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
****************************************************************
    1000    149.9     1000   137.07  1027.64   137.07  1027.64    149.9

Manning's n Values        num=       3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012  1027.64     .012

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64              8.33    8.33    8.33                .1       .3

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 863.333*

INPUT
Description:
Station Elevation Data    num=       4
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
****************************************************************
    1000   149.62     1000    136.8  1027.64    136.8  1027.64   149.62

Manning's n Values        num=       2
     Sta    n Val      Sta    n Val
********************************
    1000     .012  1027.64     .012

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64              8.33    8.33    8.33                .1       .3

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 836.666*
```

```
INPUT
Description:
Station Elevation Data    num=        4
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
****************************************************************
    1000  149.35     1000  136.52 1027.64  136.52 1027.64  149.35

Manning's n Values        num=        2
     Sta    n Val      Sta    n Val
********************************
    1000     .012 1027.64     .012

Bank Sta: Left   Right    Lengths: Left Channel    Right      Coeff Contr.    Expan.
          1000 1027.64             8.33     8.33    8.33                .1        .3

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 810

INPUT
Description:
Station Elevation Data    num=        4
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
****************************************************************
    1000  149.07     1000  136.25 1027.64  136.25 1027.64  149.07

Manning's n Values        num=        3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012 1027.64     .012

Bank Sta: Left   Right    Coeff Contr.    Expan.
          1000 1027.64             .1        .3
```


Willett Pond Dam - Water Surface Elevation vs. Discharge
Existing Conditions - No Weir Boards Installed
145
144
143
142
141
140
139
138
Water Surface Elevation (feet - NGVD)
0
200
400
600
800
1,000
1,200
1,400
Discharge (cfs)

Existing Spillway_Weir Boards Installed

```
HEC-RAS Version 4.0.0 March 2008
 U.S. Army Corps of Engineers
Hydrologic Engineering Center
      609 Second Street
      Davis, California

X     X  XXXXXX    XXXX        XXXX      XX      XXXX
X     X  X        X    X       X   X    X  X    X
X     X  X        X            X   X   X    X   X
XXXXXXX  XXXX     X       XXX  XXXX    XXXXXX    XXXX
X     X  X        X            X  X    X    X        X
X     X  X        X    X       X   X   X    X        X
X     X  XXXXXX    XXXX        X    X  X    X   XXXXX
```

****

PROJECT DATA
Project Title: Willett Pond Dam - Existing Spillway
Project File : Willett-ExistSpi.prj
Run Date and Time: 3/2/2010 8:44:34 AM

Project in English units

****

PLAN DATA

Plan Title: Weir Boards
Plan File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.p02

Geometry Title: Existing Spillway - Weir Boards
Geometry File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.g02

Flow Title : Steady Flow
Flow File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.f01

Plan Summary Information:

Number of:				
Cross Sections	= 16	Multiple Openings	=	0
Culverts	= 0	Inline Structures	=	0
Bridges	= 0	Lateral Structures	=	0

Computational Information
Water surface calculation tolerance = 0.01
Critical depth calculation tolerance = 0.01
Maximum number of iterations = 20
Maximum difference tolerance = 0.3
Flow tolerance factor = 0.001

Computation Options
Critical depth computed at all cross sections
Conveyance Calculation Method: At breaks in n values only
Friction Slope Method: Program Selects Appropriate method
Computational Flow Regime: Subcritical Flow

****

FLOW DATA

Flow Title: Steady Flow
Flow File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.f01

Flow Data (cfs)

River	Reach	RS	PF 1	PF 2	PF 3	PF 4	PF 5	PF 6	PF 7	PF 8
Willett	10	1100	1	50	100	200	300	400	500	600

River	Reach	RS	PF 9	PF 10	PF 11	PF 12	PF 13	PF 14	PF 15	PF 16
Willett	10	1100	700	800	1000	1200	1400	1500	1600	1800

Boundary Conditions

River	Reach	Profile	Upstream	Downstream
Willett	10	PF 1		Normal S = 0.033
Willett	10	PF 2		Normal S = 0.033
Willett	10	PF 3		Normal S = 0.033
Willett	10	PF 4		Normal S = 0.033
Willett	10	PF 5		Normal S = 0.033
Willett	10	PF 6		Normal S = 0.033
Willett	10	PF 7		Normal S = 0.033
Willett	10	PF 8		Normal S = 0.033
Willett	10	PF 9		Normal S = 0.033
Willett	10	PF 10		Normal S = 0.033

```
*********************************************************************************

GEOMETRY DATA

Geometry Title: Existing Spillway - Weir Boards
Geometry File : g:\P2005\1323\A20\Hydraulics\Q Rating Curves\HEC-RAS\E Spillway\Willett-ExistSpi.g02

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1100

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.25    1500  137.25    1500     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .035    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500              10      10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1080.*

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.35    1500  137.35    1500     150

Manning's n Values        num=       2
     Sta   n Val     Sta   n Val
********************************
    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500              10      10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1060.*

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.45    1500  137.45    1500     150

Manning's n Values        num=       2
     Sta   n Val     Sta   n Val
********************************
    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500              10      10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1040.*

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.55    1500  137.55    1500     150

Manning's n Values        num=       2
     Sta   n Val     Sta   n Val
********************************
    1000    .035    1500    .035

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500              10      10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1020.*
```

```
INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.65    1500  137.65    1500     150

Manning's n Values        num=       2
     Sta   n Val     Sta   n Val
********************************
    1000    .035    1500    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000    1500                 10      10      10              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1002

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  137.75 1027.64  137.75 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .035    1000    .035 1027.64    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64                  1       1       1              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1001

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  139.91 1027.64  139.91 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .035    1000    .035 1027.64    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64                  1       1       1              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 1000

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
****************************************************************
    1000     150    1000  139.91 1027.64  139.91 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .035    1000    .035 1027.64    .035

Bank Sta: Left   Right     Lengths: Left Channel   Right     Coeff Contr.   Expan.
          1000 1027.64                  2       2       2              .3       .5

CROSS SECTION


RIVER: Willett
REACH: 10                 RS: 990

INPUT
Description:
Station Elevation Data    num=      12
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
********************************************************************************
    1000     150    1000  137.75 1007.88  137.75 1007.88  139.91 1009.88  139.91
 1009.88  137.75 1017.76  137.75 1017.76  139.91 1019.76  139.91 1019.76  137.75
 1027.64  137.75 1027.64     150

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
************************************************
    1000    .012    1000    .012 1027.64    .012
```

```
Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
          1000 1027.64                9.33     9.33     9.33                .3        .5
Cross Section Lid
    num=        2
     Sta Hi Cord Lo Cord       Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75 1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                    RS: 960.*

INPUT
Description:
Station Elevation Data    num=       12
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    1000      150     1000   137.56 1007.88   137.56 1007.88   139.86 1009.88   139.86
 1009.88   137.56 1017.76   137.56 1017.76   139.86 1019.76   139.86 1019.76   137.56
 1027.64   137.56 1027.64      150

Manning's n Values        num=        3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012 1027.64

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
          1000 1027.64                9.33     9.33     9.33                .1        .3
Cross Section Lid
    num=        2
     Sta Hi Cord Lo Cord       Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75 1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                    RS: 930.*

INPUT
Description:
Station Elevation Data    num=       12
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    1000      150     1000   137.36 1007.88   137.36 1007.88   139.82 1009.88   139.82
 1009.88   137.36 1017.76   137.36 1017.76   139.82 1019.76   139.82 1019.76   137.36
 1027.64   137.36 1027.64      150

Manning's n Values        num=        3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012 1027.64

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
          1000 1027.64                9.33     9.33     9.33                .1        .3
Cross Section Lid
    num=        2
     Sta Hi Cord Lo Cord       Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75 1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                    RS: 900

INPUT
Description:
Station Elevation Data    num=       12
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    1000      150     1000   137.17 1007.88   137.17 1007.88   139.77 1009.88   139.77
 1009.88   137.17 1017.76   137.17 1017.76   139.77 1019.76   139.77 1019.76   137.17
 1027.64   137.17 1027.64      150

Manning's n Values        num=        3
     Sta    n Val      Sta    n Val      Sta    n Val
************************************************
    1000     .012     1000     .012 1027.64     .012

Bank Sta: Left   Right     Lengths: Left Channel   Right      Coeff Contr.   Expan.
          1000 1027.64                   3        3        3                .1        .3
Cross Section Lid
    num=        2
     Sta Hi Cord Lo Cord       Sta Hi Cord Lo Cord
************************************************
    1000      150  142.75 1027.64      150  142.75

CROSS SECTION


RIVER: Willett
REACH: 10                    RS: 890
```

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev
1000	149.9	1000	137.07	1027.64	137.07
1027.64	149.9				

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
1000	.012	1000	.012	1027.64	.012

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1027.64	8.33	8.33	8.33	.1	.3

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 863.333*

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev
1000	149.62	1000	136.8	1027.64	136.8
1027.64	149.62				

Manning's n Values num= 2

Sta	n Val	Sta	n Val
1000	.012	1027.64	.012

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1027.64	8.33	8.33	8.33	.1	.3

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 836.666*

INPUT
Description:
Station Elevation Data num= 4

Sta	Elev	Sta	Elev	Sta	Elev
1000	149.35	1000	136.52	1027.64	136.52
1027.64	149.35				

Manning's n Values num= 2

Sta	n Val	Sta	n Val
1000	.012	1027.64	.012

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
1000	1027.64	8.33	8.33	8.33	.1	.3

CROSS SECTION

RIVER: Willett
REACH: 10 RS: 810

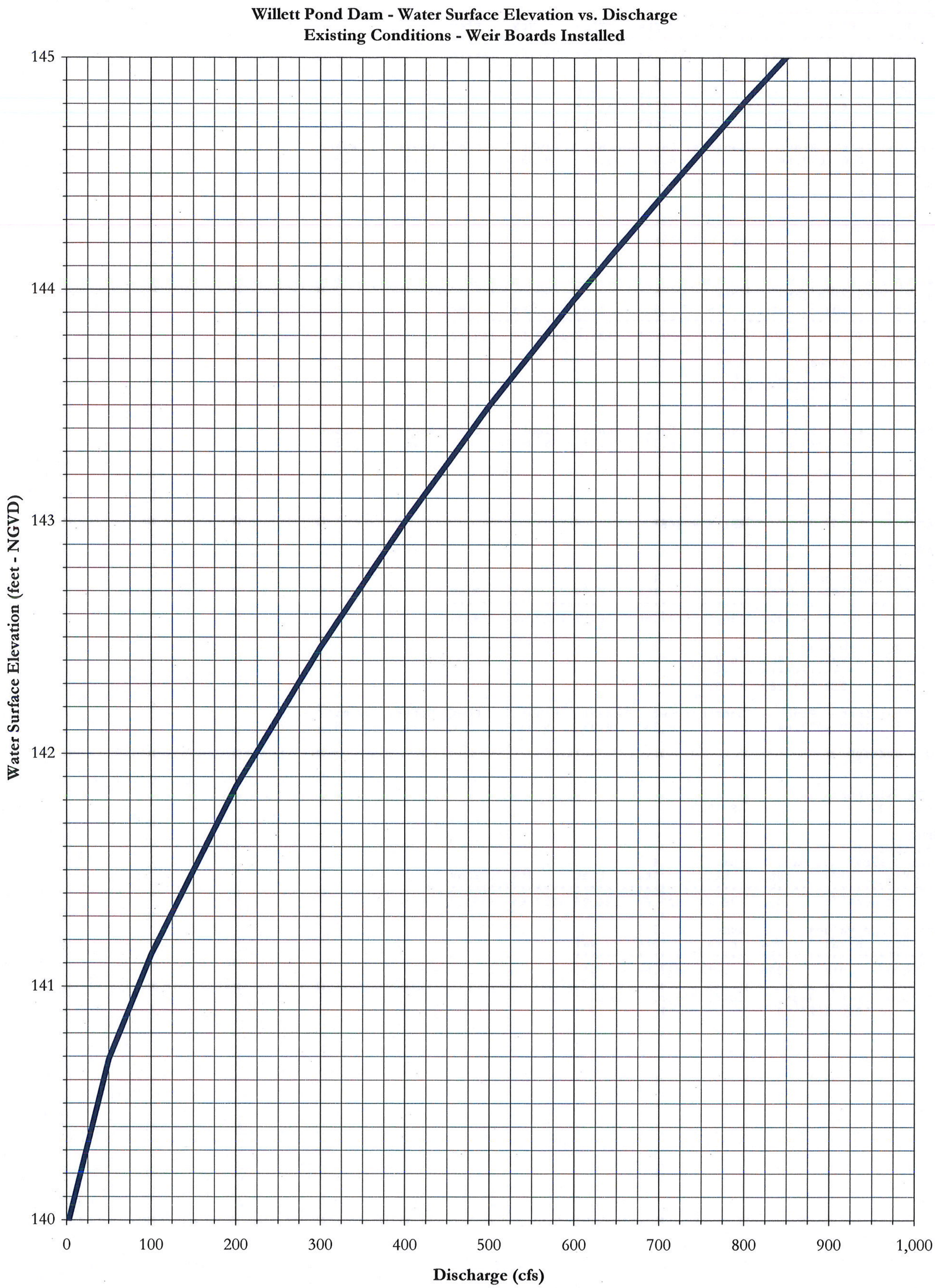
INPUT
Description:
Station Elevation Data num= 4

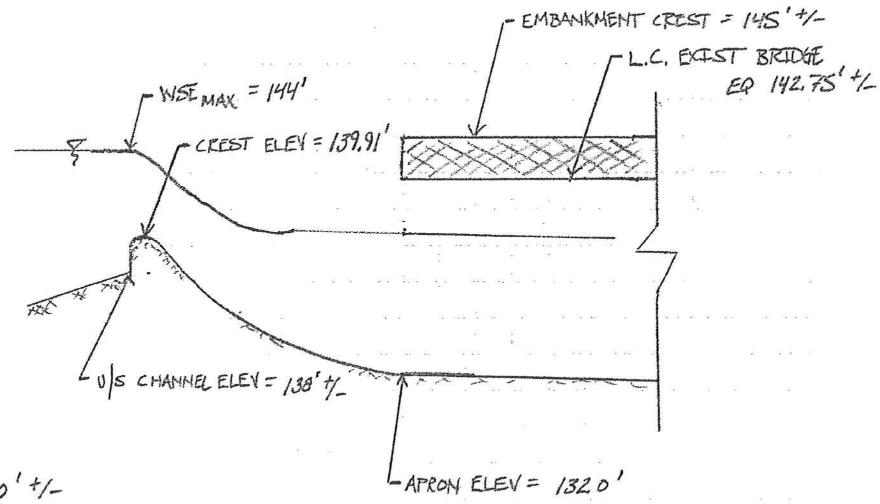
Sta	Elev	Sta	Elev	Sta	Elev
1000	149.07	1000	136.25	1027.64	136.25
1027.64	149.07				

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
1000	.012	1000	.012	1027.64	.012

Bank Sta: Left	Right	Coeff Contr.	Expan.
1000	1027.64	.1	.3


Willett Pond Dam - Water Surface Elevation vs. Discharge
Existing Conditions - Weir Boards Installed
Water Surface Elevation (feet - NGVD)
145
144
143
142
141
140
0
100
200
300
400
500
600
700
800
900
1,000
Discharge (cfs)



$Q_{MAX} = 3{,}500$ CFS

$WSE_{MAX} = 144'$ (1.0' FREEBOARD)

SPILLWAY CREST = 139.91

SPILLWAY DESIGN PARAMETERS:

$H_0 = 2.55'$ (ASSUMED)

$P = 139.91 - 138.0 \Rightarrow P = 2.0'$ +/-

$\therefore \frac{P}{H_0} = \frac{2.0}{2.55} = 0.78$

FROM FIG 9-23 (ATTACHED), $C_0 = 3.86$

WHERE, C_0 = DESIGN WEIR COEFFICIENT
H_0 = SPILLWAY DESIGN HEAD
P = U/S CHANNEL DEPTH

NOTES: - SPILLWAY HYDRAULIC DESIGN IN ACCORDENCE W/ USBR "DESIGN OF SMALL DAMS"

- EXAMPLE HAND CALC'S PROVIDED BELOW. FULL CALC'S PREFORMED IN THE ATTACHED SPREADSHEETS

CALCULATE SPILLWAY DISCHARGE FOR MAX WSE = 144':

1. EFFECTIVE HEAD OVER SPILLWAY (H_E) -

$H_E = 144.0' - 139.91' \Rightarrow \underline{H_E = 4.09'}$

2. EFFECTIVE SPILLWAY LENGTH (L_E) -

$L_E = L - 2(K_a)H_E$

WHERE, L_E = EFFECTIVE LENGTH
K_a = CONTRACTION COEF.
H_E = EFFECTIVE HEAD
L = TOTAL SPILLWAY LENGTH

FOR $L = 105$ FT
$K_a = 0.2$
$H_E = 4.09$ FT

$L_E = 105' - 2(0.2)4.09' \Rightarrow \underline{L_E = 103.4'}$

3. EFFECTIVE WEIR COEFFICIENT (FROM FIG 9-24 – ATTACHED) –

$$\left.\begin{array}{l} H_E = 4.09' \\ H_0 = 2.55 \end{array}\right] \quad H_E/H_0 = 1.60$$

FROM FIGURE 9-24, $C/C_0 = 1.07$

$$\therefore \quad C_E = C_0(1.07) \Rightarrow C_E = 3.86(1.07) \Rightarrow \underline{\underline{C_E = 4.13}}$$

4. UNSUBMERGED SPILLWAY DISCHARGE CAPACITY –

$$Q_{UNSUB} = C_E L_E H_E^{1.5} \qquad \text{FOR} \quad C_E = 4.13, \; L_E = 103.4', \; H_E = 4.09'$$

$$Q_{UNSUB} = 4.13(103.4)(4.09^{1.5}) \Rightarrow \underline{\underline{Q_{UNSUB} = 3{,}532 \text{ CFS}}}$$

5. CALCULATE TAILWATER IN D/S CHANNEL –

- ASSUME CRITICAL FLOW IN D/S CHANNEL (IE DEBRIS BLOCKAGE)
(W/OUT BLOCKAGE, D/S FLOW WILL BE SUPERCRITICAL, $S = 0.03$ FT/FT +/-)

FOR RECTANGULAR CHANNEL $Y_c = \left(q^2/g\right)^{1/3}$ WHERE, Y_c = CRITICAL DEPTH
q = UNIT DISCHARGE
g = 32.2 FT/SEC²

$$q = \frac{Q}{L_{CH}} = \frac{3532}{27.5} \Rightarrow q = 128.4 \text{ CFS/FT}$$

(NOTE: L_{CH} = D/S CHANNEL WIDTH = 27.5')

$$Y_c = \left(\frac{128.4^2}{32.2}\right)^{1/3} \Rightarrow Y_c = 8.0' \quad \therefore \quad \underline{\underline{\text{TW ELEV} = 8.0' + 132 = 140'}}$$

6. SUBMERGED SPILLWAY DISCHARGE:

$$Q_{SUB} = Q_{UNSUB}\left[1 - \left(\frac{H_{D/S}}{H_{U/S}}\right)^{1.5}\right]^{0.385}$$

WHERE, $H_{D/S}$ = D/S HEAD OVER SPILLWAY
$H_{U/S}$ = U/S HEAD OVER SPILLWAY

$$\left.\begin{array}{l} H_{D/S} = 140.0 - 139.91 = 0.09' \\ H_{U/S} = 4.09' \end{array}\right\} \quad Q_{SUB} = 3532\left[1 - \left(\frac{0.09}{4.09}\right)^{1.5}\right]^{0.385} \Rightarrow \boxed{Q_{SUB} = 3527 \text{ CFS}}$$

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A WATER RESOURCES TECHNICAL PUBLICATION

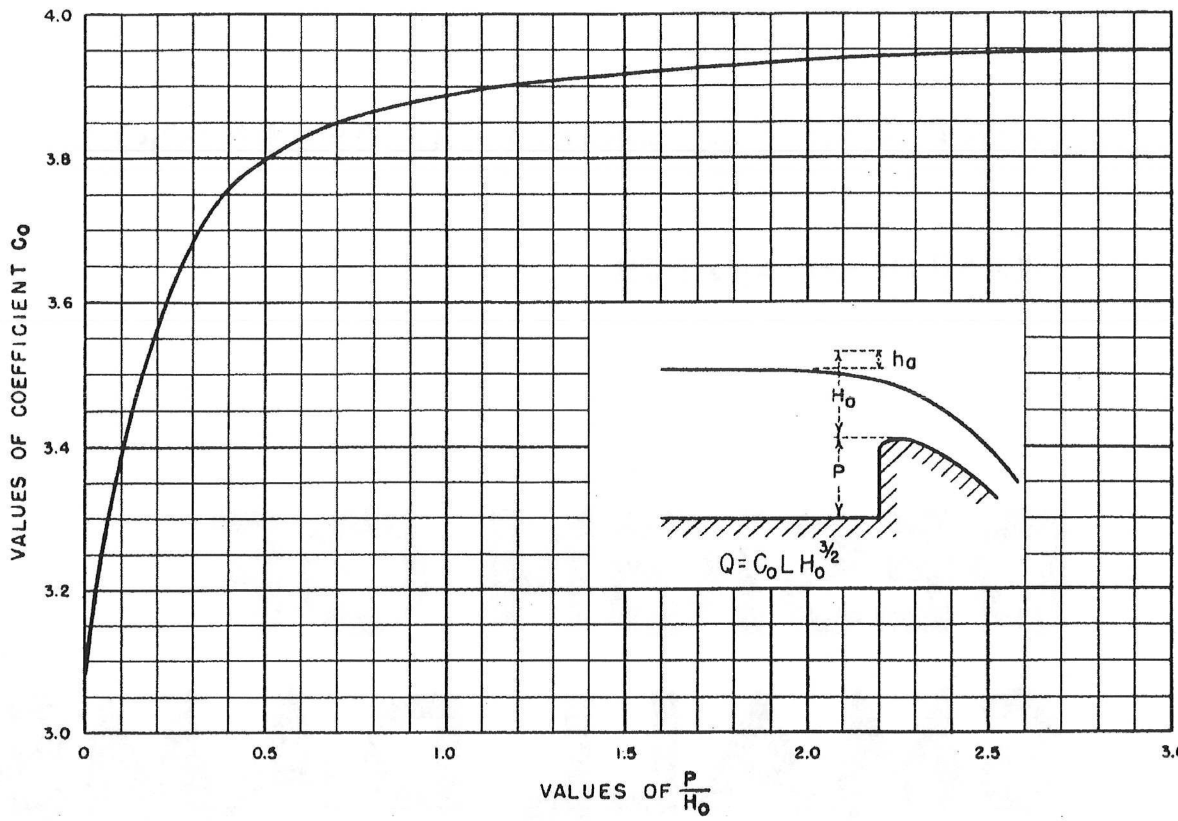


Figure 9-23.—Discharge coefficients for vertical-faced ogee crest. 288-D-2409.

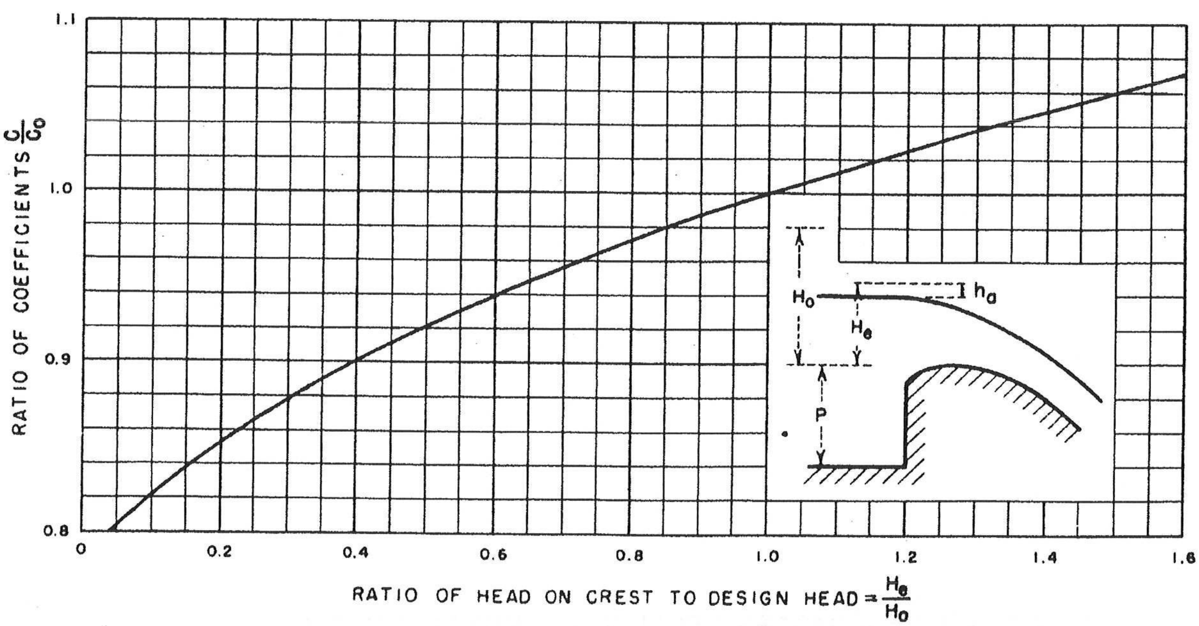


Figure 9-24.—Discharge coefficients for other than the design head. 288-D-2410.

FUSS & O'NEILL

WILLETT POND DAM
PHASE II INSPECTION / EVALUATION REPORT
SPILLWAY ALTERNATIVES ASSESSMENT

Spillway Parameters :*		
Design Head (Ho):	2.6	*feet*
Design Q Coef (Co):	3.86	
US Depth (P):	2	*feet*
Crest Elevation:	137.75	*feet*
Crest Length:	140	*feet*
No. Abutment:	2	
Abutment Coef:	0.2	
Downstream Channel Parameters :		
DS Apron Invert:	129	*feet*
DS Chanel Width:	28.3	*feet*

Spillway Discharge Capacity - No Tailwater Submergence							Tailwater Elevation**			Spillway Capcaity - Submerged Conditions	
1	2	3	4	5	6	7	8	9	10	11	
US WSE feet	HW (He) feet	Le feet	He/Ho	Ce/Co	Ce	Q(un-sub) cfs	q(un-sub) cfs/ft	TW Depth feet	TW Elev feet	Sub Depth cfs	Q(sub) cfs
140.00	2.25	139.1	0.865	0.98	3.80	1,783	62.99	4.98	133.98	0.00	1783
140.25	2.50	139.0	0.962	1.00	3.85	2,113	74.68	5.57	134.57	0.00	2113
140.50	2.75	138.9	1.058	1.01	3.89	2,465	87.12	6.18	135.18	0.00	2465
140.75	3.00	138.8	1.154	1.02	3.94	2,838	100.30	6.79	135.79	0.00	2838
141.00	3.25	138.7	1.250	1.03	3.98	3,232	114.22	7.40	136.40	0.00	3232
141.25	3.50	138.6	1.346	1.04	4.02	3,648	128.89	8.02	137.02	0.00	3648
141.50	3.75	138.5	1.442	1.05	4.06	4,085	144.35	8.65	137.65	0.00	4085
141.75	4.00	138.4	1.538	1.06	4.11	4,546	160.63	9.29	138.29	0.54	4458
142.00	***4.25***	***138.3***	***1.635***	***1.08***	***4.15***	***5,032***	***177.80***	***9.94***	***138.94***	***1.19***	***4731***
142.25	4.50	138.2	1.731	1.09	4.20	5,545	195.94	10.60	139.60	1.85	4927
142.50	4.75	138.1	1.827	1.10	4.26	6,088	215.14	11.29	140.29	2.54	5033
142.75	5.00	138.0	1.923	1.12	4.32	6,665	235.52	11.99	140.99	3.24	5020
143.00	5.25	137.9	2.019	1.14	4.39	7,279	257.21	12.71	141.71	3.96	4828
143.25	5.50	137.8	2.115	1.16	4.46	7,934	280.37	13.46	142.46	4.71	4321
143.50	5.75	137.7	2.212	1.18	4.55	8,636	305.17	14.25	143.25	5.50	3017

* Spillway Design Parameters: Methodology from USBR "Design of Small Dams" for hydraulic design of Ogee crest overflow spillways.

**Tailwater elevation caclulated assuming critical depth in downstream channel. Normal depth in downstream channel will be supercritical (i.e. less than calculated above). Critical depth was assumed to account for potential debris blockage.

1. WSE in impoundment
2. Head water measured above spillway crest
3. Effective length of spillway crest - adjusted for end contractions
4. Ratio of effective head to design head
5. Ratio of effective spillway discharge coefficient to design spillway discharge coefficient
6. Effective spillway discharge coefficient calculated from Figure 9-24 (USBR, Design of Small Dams)
7. Spillway discharge assuming no tailwater submergence
8. Unsubmerged unit discharge
9. Tailwater depth calculated assuming critical depth
10. Tailwater elevation based upon critical depth
11. Submerged depth measured from the TW elevation to the spillway crest
12. Spillway discharge capacity adjusted for TW submergence

FUSS & O'NEILL

WILLETT POND DAM
PHASE II INSPECTION / EVALUATION REPORT
SPILLWAY ALTERNATIVES ASSESSMENT

Spillway Parameters:*

Design Head (Ho):	2.55 *feet*
Design Q Coef (Co):	3.86
US Depth (P):	2 *feet*
Crest Elevation:	139.91 *feet*
Crest Length:	140 *feet*
No. Abutment:	2
Abutment Coef:	0.2

Downstream Channel Parameters:

DS Apron Invert:	131 *feet*
DS Chanel Width:	28.3 *feet*

Spillway Discharge Capacity - No Tailwater Submergence							Tailwater Elevation**			Spillway Capcaity - Submerged Conditions	
1	2	3	4	5	6	7	8	9	10	11	12
US WSE feet	HW (He) feet	Le feet	He/Ho	Ce/Co	Ce	Q(un-sub) cfs	q(un-sub) cfs/ft	TW Depth feet	TW Elev feet	Sub Depth cfs	Q(sub) cfs
140.00	0.09	140.0	0.035	0.80	3.08	12	0.41	0.17	131.17	0.00	12
140.25	0.34	139.9	0.133	0.83	3.20	89	3.14	0.67	131.67	0.00	89
140.50	0.59	139.8	0.231	0.86	3.31	210	7.42	1.20	132.20	0.00	210
140.75	0.84	139.7	0.329	0.88	3.41	367	12.97	1.73	132.73	0.00	367
141.00	1.09	139.6	0.427	0.91	3.50	556	19.65	2.29	133.29	0.00	556
141.25	1.34	139.5	0.525	0.93	3.58	775	27.37	2.85	133.85	0.00	775
141.50	1.59	139.4	0.624	0.95	3.65	1,020	36.05	3.43	134.43	0.00	1020
141.75	1.84	139.3	0.722	0.96	3.72	1,291	45.63	4.01	135.01	0.00	1291
142.00	2.09	139.2	0.820	0.98	3.77	1,586	56.05	4.60	135.60	0.00	1586
142.25	2.34	139.1	0.918	0.99	3.82	1,904	67.27	5.20	136.20	0.00	1904
142.50	2.59	139.0	1.016	1.00	3.87	2,243	79.26	5.80	136.80	0.00	2243
142.75	2.84	138.9	1.114	1.01	3.92	2,604	92.00	6.41	137.41	0.00	2604
143.00	3.09	138.8	1.212	1.03	3.96	2,985	105.49	7.02	138.02	0.00	2985
143.25	3.34	138.7	1.310	1.04	4.00	3,389	119.74	7.64	138.64	0.00	3389
143.50	3.59	138.6	1.408	1.05	4.05	3,814	134.76	8.26	139.26	0.00	3814
143.75	3.84	138.5	1.506	1.06	4.09	4,262	150.60	8.90	139.90	0.00	4262
144.00	***4.09***	***138.4***	***1.604***	***1.07***	***4.14***	***4,735***	***167.32***	***9.54***	***140.54***	***0.63***	***4622***
144.25	4.34	138.3	1.702	1.08	4.19	5,235	184.98	10.20	141.20	1.29	4888
144.50	4.59	138.2	1.800	1.10	4.24	5,764	203.68	10.88	141.88	1.97	5075
144.75	4.84	138.1	1.898	1.11	4.30	6,326	223.55	11.58	142.58	2.67	5166
145.00	5.09	138.0	1.996	1.13	4.37	6,925	244.70	12.30	143.30	3.39	5123

* Spillway Design Parameters: Methodology from USBR "Design of Small Dams" for hydraulic design of Ogee crest overflow spillways.

**Tailwater elevation calculated assuming critical depth in downstream channel. Normal depth in downstream channel will be supercritical (i.e. less than calculated above). Critical depth was assumed to account for potential debris blockage.

1. WSE in impoundment
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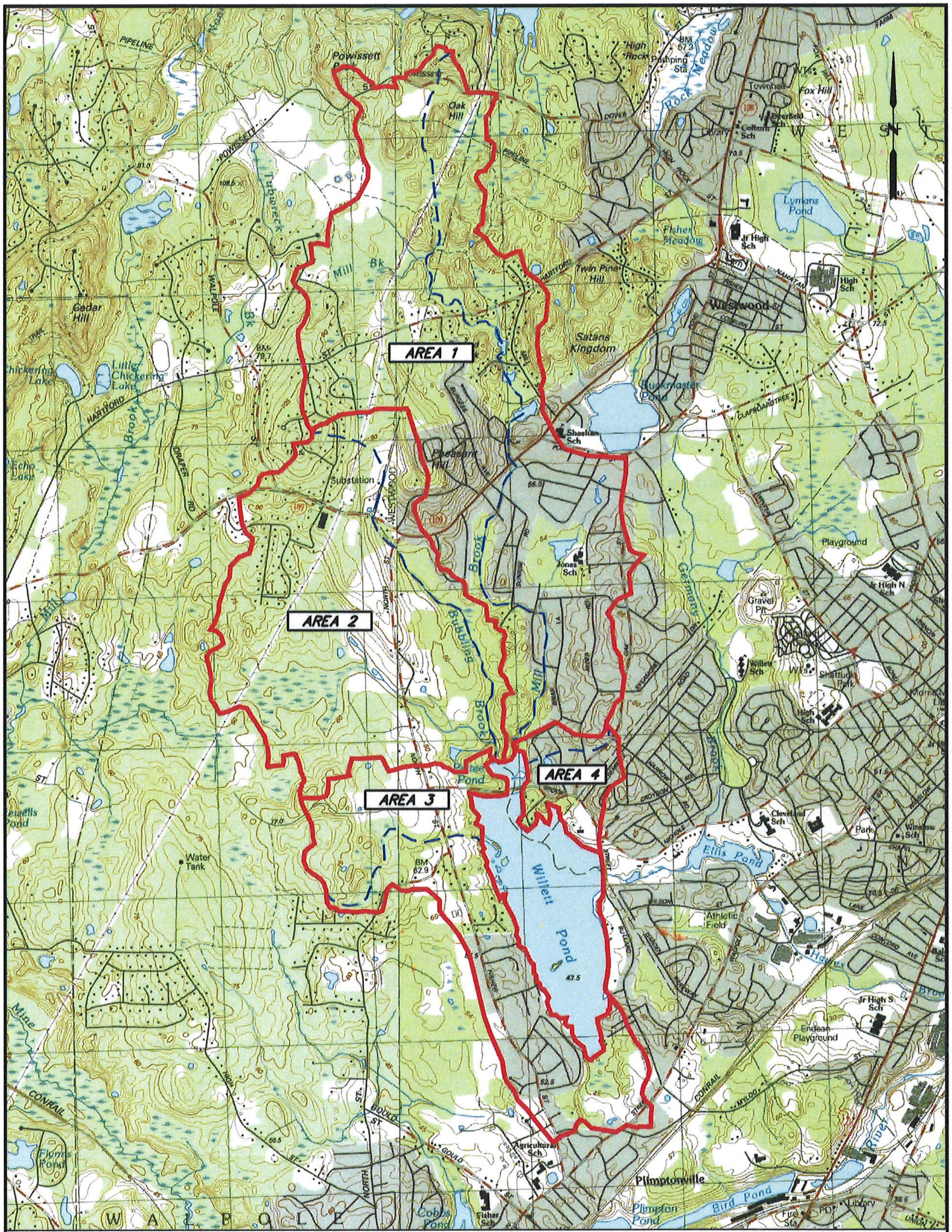
7. Spillway discharge assuming no tailwater submergence
8. Unsubmerged unit discharge
9. Tailwater depth calculated assuming critical depth
10. Tailwater elevation based upon critical depth
11. Submerged depth measured from the TW elevation to the spillway crest
12. Spillway discharge capacity adjusted for TW submergence



LEGEND

DRIANAGE AREA BOUNDARY

TIME OF CONCENTRATION

MAP REFERENCE:

BASE MAP PREPARED FROM USGS TOPOGRAPHIC MAPPING PROVIDED BY MassGIS*

* OFFICE OF GEOGRAPHIC & ENVIRONMENTAL INFORMATION, COMMONWEALTH OF MASS. EXECUTIVE OFFICE OF ENVIRONMENTAL AFFAIRS

MASSACHUSETTS

LOCATION

SCALE:	
HORZ.: 1" = 2000'	
VERT.: NA	
DATUM:	
HORZ.: NA	
VERT.: AN	
0 1000 2000	
GRAPHIC SCALE	

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NEPONSET RIVER LAND HOLDING ASSOCIATION

DRAINAGE AREA MAP

WILLETT POND DAM

NORWOOD MASSACHUSETTS

PROJ. No.: 20061323.A20
DATE: NOVEMBER 2009

File Path: J:\DWG\P2006\1323\A20\Plan\061323A10DRA001.dwg, Layout: ATT. A Tue, Mar 02, 2010 - 1:22 PM User: kmikolinski

UCS: | LMAX: DRAINAGE AREA | CTB: FO_COLOR.ctb | MS VIEW: MVIEW_DA

Hydrologic Modeling System - Basin Input Parameters

Project: Willett Pond Dam
Project No: 20051323.A20

Scenario: Spillway Design Flood

```
Basin: 0.1-LAG72
     Last Modified Date: 7 December 2009
     Last Modified Time: 21:00:45
     Version: 3.3
     Unit System: English
     Missing Flow To Zero: No
     Enable Flow Ratio: No
     Allow Blending: No
     Compute Local Flow At Junctions: No

     Sediment Grade Scale: NONE
     Enable Sediment Routing: No

     Enable Quality Routing: No
End:

Subbasin: Area 1
     Canvas X: -4046.2427745664745
     Canvas Y: 3800.578034682081
     Label X: -69.0
     Label Y: -4.0
     Area: 1.907
     Downstream: Willett Pond

     Canopy: None

     Surface: None

     LossRate: Initial+Constant
     Percent Impervious Area: 0.0
     Initial Loss: 0.1
     Constant Loss Rate: 0.1

     Transform: SCS
     Lag: 281
     Unitgraph Type: STANDARD

     Baseflow: None

     Water Quality: ZERO
     End Water Quality:

     Erosion: None
End:

Subbasin: Area 2
     Canvas X: -5173.410404624278
     Canvas Y: 1965.3179190751443
     Label X: -69.0
     Label Y: -2.0
     Area: 1.310
     Downstream: Willett Pond

     Canopy: None

     Surface: None

     LossRate: Initial+Constant
     Percent Impervious Area: 0.0
     Initial Loss: 0.1
     Constant Loss Rate: 0.1

     Transform: SCS
```

```
     Lag: 206
     Unitgraph Type: STANDARD

     Baseflow: None

     Water Quality: ZERO
     End Water Quality:

     Erosion: None
End:

Subbasin: Area 3
     Canvas X: -5130.0578034682085
     Canvas Y: -419.07514450867075
     Label X: -69.0
     Label Y: 2.0
     Area: 0.767
     Downstream: Willett Pond

     Canopy: None

     Surface: None

     LossRate: Initial+Constant
     Percent Impervious Area: 0.0
     Initial Loss: 0.1
     Constant Loss Rate: 0.1

     Transform: SCS
     Lag: 189
     Unitgraph Type: STANDARD

     Baseflow: None

     Water Quality: ZERO
     End Water Quality:

     Erosion: None
End:

Subbasin: Water Surface
     Canvas X: -3988.439306358382
     Canvas Y: -1560.6936416184972
     Label X: -48.0
     Label Y: -26.0
     Area: 0.325
     Downstream: Willett Pond

     Canopy: None

     Surface: None

     LossRate: Initial+Constant
     Percent Impervious Area: 0.0
     Initial Loss: 0.05
     Constant Loss Rate: 0.05

     Transform: SCS
     Lag: 5
     Unitgraph Type: STANDARD

     Baseflow: None

     Water Quality: ZERO
     End Water Quality:

     Erosion: None
End:

Subbasin: Area 4
     Canvas X: -1791.2552891396326
```

```
     Canvas Y: 1897.03808180536
     Area: 0.1497
     Downstream: Willett Pond

     Canopy: None

     Surface: None

     LossRate: Initial+Constant
     Percent Impervious Area: 0.0
     Initial Loss: 0.1
     Constant Loss Rate: 0.1

     Transform: SCS
     Lag: 129
     Unitgraph Type: STANDARD

     Baseflow: None

     Water Quality: ZERO
     End Water Quality:

     Erosion: None
End:

Junction: Willett Pond
     Canvas X: -3060.6488011283495
     Canvas Y: 980.2538787023982
     Label X: -102.0
     Label Y: -1.0
     Downstream: Inflow
End:

Sink: Inflow
     Canvas X: -1856.115107913669
     Canvas Y: -338.1294964028775
End:

Basin Schematic Properties:
     Last View N: 5000.0
     Last View S: -5000.0
     Last View W: -5000.0
     Last View E: 5000.0
     Maximum View N: 5000.0
     Maximum View S: -5000.0
     Maximum View W: -5000.0
     Maximum View E: 5000.0
     Extent Method: Elements
     Buffer: 0
     Draw Icons: Yes
     Draw Icon Labels: Yes
     Draw Map Objects: No
     Draw Gridlines: Yes
     Draw Flow Direction: No
     Fix Element Locations: No
End:
```

Hydrologic Modeling System - Meteorlogic Input Parameters

Project: Willett Pond Dam
Project No: 20051323.A20

Scenario: Spillway Design Flood

```
Meteorology: PMP - 1 hr
     Last Modified Date: 8 December 2009
     Last Modified Time: 14:29:22
     Version: 3.4
     Unit System: English
     Precipitation Method: Specified Average
     Radiation Method: None
     Snowmelt Method: None
     Evapotranspiration Method: No Evapotranspiration
     Use Basin Model: 0.1
     Use Basin Model: 0.1-LAG72
     Use Basin Model: CN
End:

Precip Method Parameters: Specified Average
     Allow Depth Override: No
     Set Missing Data to Zero: Yes
End:

Subbasin: Area 1
     Gage: PMP - 1 hr
End:

Subbasin: Area 2
     Gage: PMP - 1 hr
End:

Subbasin: Area 3
     Gage: PMP - 1 hr
End:

Subbasin: Water Surface
     Gage: PMP - 1 hr
End:

Subbasin: Area 4
     Gage: PMP - 1 hr
End:
```

Hydrologic Modeling System – Precipitation Input Parameters

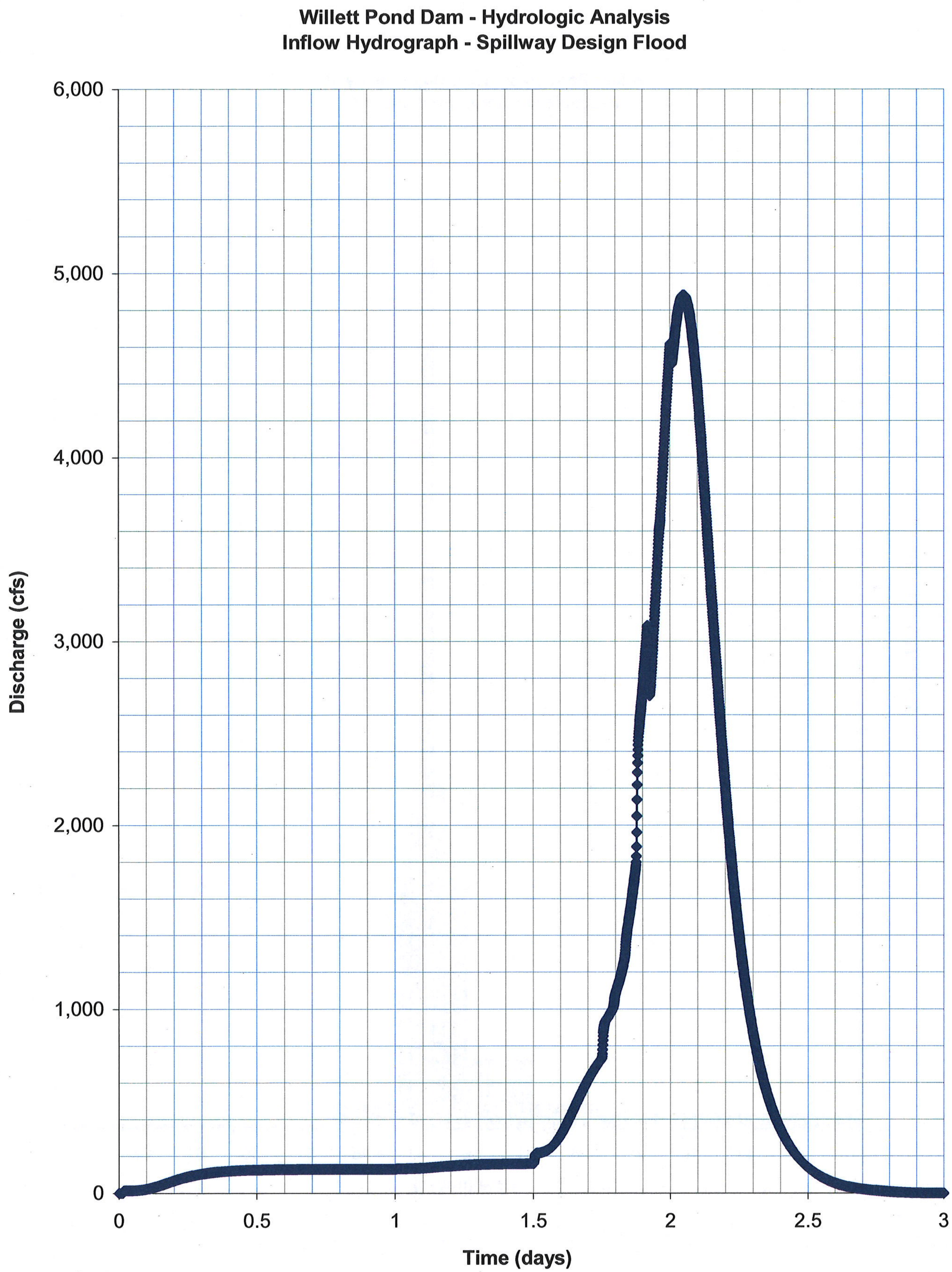
Project: Willett Pond Dam
Project No: 20051323.A20

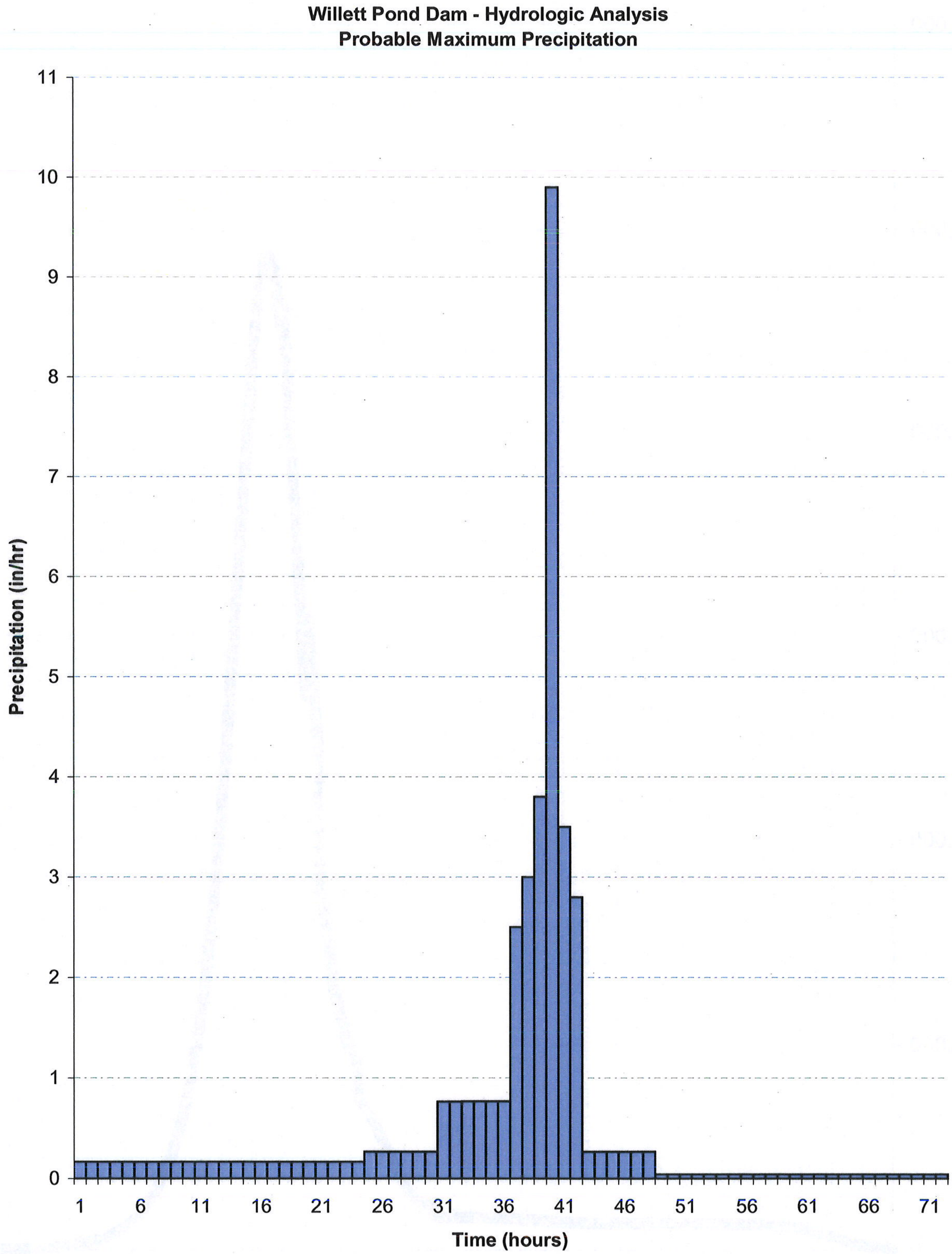
Scenario: Spillway Design Flood

```
Gage Manager: HMS
     Version: 3.3
End:

Gage: PMP - 6 hr
     Last Modified Date: 7 December 2009
     Last Modified Time: 20:20:38
     Units System: English
     Gage Type: Precipitation
     Precipitation Type: Incremental
     Units: IN
     Data Type: PER-CUM
     Local to Project: YES
     Start Time: 1 January 2009, 00:00
     End Time: 4 January 2009, 01:00
     DSS File: HMS.dss
     Pathname: //PMP - 6 HR/PRECIP-INC//1HOUR/GAGE/
End:

Gage: PMP - 1 hr
     Last Modified Date: 8 December 2009
     Last Modified Time: 14:25:50
     Units System: English
     Gage Type: Precipitation
     Precipitation Type: Incremental
     Units: IN
     Data Type: PER-CUM
     Local to Project: YES
     Start Time: 1 January 2009, 00:00
     End Time: 4 January 2009, 00:00
     Start Time: 1 January 2009, 00:00
     End Time: 4 January 2009, 01:00
     DSS File: HMS.dss
     Pathname: //PMP - 1 HR/PRECIP-INC//1HOUR/GAGE/
End:
```


Willett Pond Dam - Hydrologic Analysis
Inflow Hydrograph - Spillway Design Flood
6,000
5,000
4,000
3,000
2,000
1,000
0
Discharge (cfs)
0
0.5
1
1.5
2
2.5
3
Time (days)


Willett Pond Dam - Hydrologic Analysis
Probable Maximum Precipitation
Precipitation (in/hr)
0
1
2
3
4
5
6
7
8
9
10
11
1
6
11
16
21
26
31
36
41
46
51
56
61
66
71
Time (hours)

FUSS & O'NEILL

Appendix B

Geotechnical Data

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-1 SHEET: 1 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME
5/11	surface	5'	0 hr

Time and Date of Completion: 5/11/09

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0						Processed Aggregate, Asphalt, crushed stone base.		
1								
2								1
3								
4								
5								
6	S-1	5-7	0/24	3-3-2-1	No recovery (stone lodged in spoon tip)			
7						Sand		
8	S-2	7-9		1-1-0-1	Very loose, light brown, fine to medium SAND.		SW	
9								
10	S-3	9-11	0/24	7-1-0-3	No recovery (Asphalt chunk in spoon tip)			
11								
12	S-4	11-13		1-0-1-1	Very loose, light brown, fine to medium SAND, trace silt, trace gravel.		SW	
13								
14	S-5	13-15		4-3-1-1	Loose, light brown, fine to medium SAND.		SW	

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 1. Auguring through asphalt and aggregate base course.

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-1 SHEET: 2 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION:145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion: ________________

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
15								
16	S-6	15-17		WOH-1-0-1	Very loose, grey, fine SAND,.		SP	
17								
18	S-7	17-19		2-2-2-2	Loose, grey, fine SAND.		SP	
19						Sand		
20	S-8	19-21		2-1-0-1	Very loose, grey, fine SAND, little silt, trace gravel.		SP	
21								
22	S-9	21-23		2-1-3-3	Loose, grey fine SAND, little silt. (Very thin lense of organic material at the base of the hole)		SP	
23								
24	S-10	23 -23.5		50/5"	No recovery (rock in spoon tip).			2
25								
26						Rock		
27	S-11	27		50/1"	(Spoon refusal at 27'1")			
28								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 2. Auger through weathered rock from 23.5 to 27 feet.

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-2 SHEET: 1 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion:

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								
1								
2								
3								
4								
5								
6	S-1	5-7		3-4-4-3	Loose, light brown, fine SAND.		SP	
7						Fine Sand		
8								
9								
10								
11	S-2	10-12		2/3/3/4	Loose, light brown, fine SAND.		SP	
12								
13								
14								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: .

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-2 SHEET: 2 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion: ______

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
15								
16	S-3	15-17	0/24	5-7-3-2	(Asphalt chunk stuck in spoon tip)			
17								
18								
19								
20						Fine Sand		
21	S-3	20-22		4-6-6-7	Medium dense, light brown, fine SAND, trace Silt		SP	
22								
23								
24								
25								
26	S-4	25-27	16/24	27-50/4"	Very dense, gravel, sand, brocken ROCK.	Rock, gravel, sand		
27					Auger refusal at 27.5 feet			
28								
29								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: .

G:\P2005\1323\A20\Phase II\Boring B-2.doc

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-3 SHEET: 1 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME
5/11	Surface	20'	0 hr
6/11	Surface	20.6'	30 days

Time and Date of Completion: 5/11/09

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								
1								
2								
3								
4								
5								
6	S-1	5-7		3-5-3-3	Loose, light brown, fine SAND, trace silt.		SP	
7						Sand		
8								
9								
10								
11	S-2	10-12		2-2-2-2	Loose, light brown, fine to medium SAND.		SW	
12								
13								
14								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: .

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-3 SHEET: 2 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion:

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
15								
16	S-3	15-17		2-3-3-5	Loose, light brown, fine to medium SAND, trace gravel.		SW	
17								
18								
19								
20								
21	S-3	20-22	0/24	WOH/2'	No recovery	Sand		
22								
23								
24								
25								
26	S-4	25-26.33		24-27-50/4"	Very dense, fine to coarse Sand, little silt, little gravel.			1
27					Auger refusal at 26' 4"	Rock		
28								
29								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 1. Rock fragments in tip of spoon.

FUSS & O'NEILL, INC. **CONSULTING ENGINEERS** **MANCHESTER, CT 06040**	**BORING LOG** **PROJECT: Willett Pond Dam** **LOCATION: Walpole/Norwood, MA**	**BORING ID:** B-4 **SHEET:** 1 of 2 **PROJECT NO:** 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion: ______________

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								1
1								
2								
3								
4								
5						Sand		2
6	S-1	5-7		1-1-1-2	Very loose, light brown, fine SAND, trace silt. (some processed aggregate)		SP	
7								
8								
9								
10								
11	S-2	10-12		2-1-2-1	Very loose, light brown, fine to medium SAND, some gravel.		SW	
12						Sand and gravel		
13								
14								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 1. Auger returns of asphalt, processed aggregate, gravel and sand through 5 feet. 2. Very moist starting at 5 feet (presumed water level).

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dam LOCATION: Walpole/Norwood, MA	BORING ID: B-4 SHEET: 2 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/11/09 DATE FINISHED: 5/11/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion:

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
15								
16	S-3	15-17		1-1-0-1	Very loose, olive, fine to medium SAND, trace gravel.		SW	3
17								
18								
19								
20								
21								
22								
23								
24								
25								
26								
27								
28								
29								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 3. Hole filled with water to 5 feet.

FUSS & O'NEILL, INC.
CONSULTING ENGINEERS
MANCHESTER, CT 06040

BORING LOG

PROJECT: Willett Pond Dike

LOCATION: Walpole/Norwood, MA

BORING ID: B-5
SHEET: 1 of 2
PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 143 (approx.)
DATE STARTED: 5/12/09 DATE FINISHED: 5/12/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion:

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								
1	S-1	0-2		5-7-12-13	Organics, topsoil, Rock fragment lodged in tip.			
2								
3	S-2	2-4		8-5-11-11	Medium dense , light brown, fine SAND, trace silt.			
4								
5	S-3	4-6		12-9-8-7	Medium dense, light brown, fine SAND. (some organic material/roots)			
6								
7	S-4	6-8		2-2-1-2	Very loose, olive, fine SAND, trace silt.	Sand		
8								
9	S-5	8-10		6-4-4-12	Loose, light brown, fine SAND, trace silt.			
10								
11	S-6	10-12		3-5-5-3	Loose, light brown, fine SAND, trace silt.			
12								
13	S-7	12-14		6-9-3-3	Medium dense, light brown, fine SAND, trace silt.			
14								1

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 1. Water encountered at 13.5 feet.

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dike LOCATION: Walpole/Norwood, MA	BORING ID: B-5 SHEET: 2 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 145 (approx.)
DATE STARTED: 5/12/09 DATE FINISHED: 5/12/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME

Time and Date of Completion:

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
15	S-8	14-16		1-0-1-0	Very loose, light brown, fine SAND, trace silt.			
16						Sand		
17	S-9	16-18		9-12-16-19	Medium Dense, light brown, fine SAND, trace silt, little gravel.(Rock fragments in tip of spoon)			
18								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: Core holes were pushed upstream of core wall and water was encountered at 5 feet.

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dike LOCATION: Walpole/Norwood, MA	BORING ID: B-6 SHEET: 1 of 1 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 143 (approx.)
DATE STARTED: 5/12/09 DATE FINISHED: 5/12/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME
5/12	surface	5'	0 hrs

Time and Date of Completion: 5/12/09

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								
1	S-1	0-2		3-5-7-9	Medium dense, light brown, fine SAND, trace silt. (organics in surface layer)		SP	
2						Sand		
3	S-2	2-4		5-7-6-6	Medium dense, light brown, fine SAND, trace silt.		SP	
4								
5	S-3	4-6		5-5-7-3	Medium dense, GRAVEL, some silt.	Gravel	GP	
6								
7	S-4	6-8		1-1-1-1	Very loose, light brown, silt..		ML	
8						Silt		
9	S-5	8-		50/4"	Spoon refusal at 8'4".			
10								
11								
12								
13								
14								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: 1. Auger returns of asphalt, processed aggregate, gravel and sand through 5 feet. 2. Very moist starting at 5 feet (presumed water level).

FUSS & O'NEILL, INC. CONSULTING ENGINEERS MANCHESTER, CT 06040	**BORING LOG** PROJECT: Willett Pond Dike LOCATION: Walpole/Norwood, MA	BORING ID: B-7 SHEET: 1 of 2 PROJECT NO: 20051323.A20

CONTRACTOR: Geosearch
OPERATOR: Jason
F&O REPRESENTATIVE: C. Barnwell
DRILLING METHOD: HSA (3 ¾" ID)
SAMPLING METHOD: Split Spoon (2" OD)
HAMMER WT: 140 lbs HAMMER FALL (IN): 30"
BORING LOCATION: (see plan)
GROUND ELEVATION: 143 (approx.)
DATE STARTED: 5/12/09 DATE FINISHED: 5/12/09

WATER LEVEL MEASUREMENTS

DATE	MS. PT.	WATER AT	TIME
5/12	surface	6'	0 hr
6/11	surface	4.3'	29 days

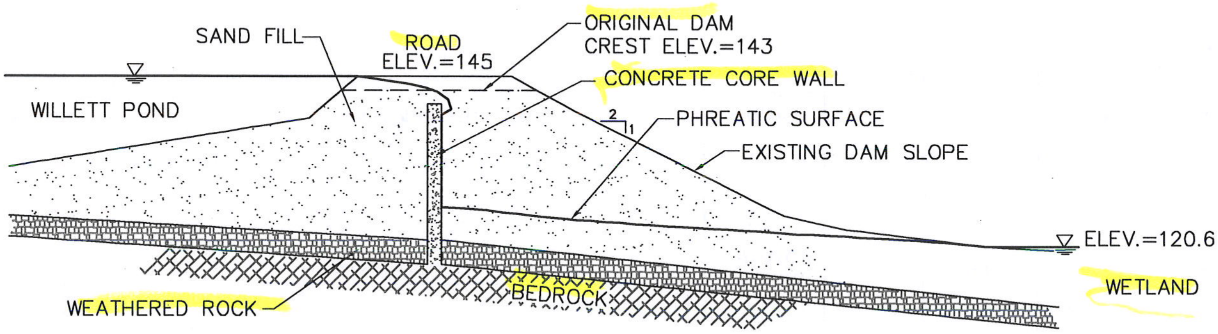
Time and Date of Completion: 5/12/09

DEPTH (FT)	SAMPLE NO.	SAMPLE DEPTH (FT)	REC/ PEN (IN/IN)	BLOWS/ 6"	SAMPLE DESCRIPTION	STRATA CHANGE	USCS Class.	NOTES
0								
1	S-1	0-2		1/1/3/5	Loose, SILT, little sand, (organics in top layer)		ML	
2								
3	S-2	2-4		5/5/2/1	Loose, SILT, little sand.		OL	
4								
5	S-3	4-6		WOH-WOH-1-0	Organics (2" recovery in spoon)			
6								
7	S-4	6-8		2-1-1-2	Very Loose, light brown silt and organics		OL	
8								
9	S-5	8-10		8-29-34-36	Very Dense, olive fine SAND, some silt.		SM	
10								
11	S-6	10-12		6-24-26-20	Dense, olive SAND, trace silt, some gravel.			
12								
13	S-7	12-14		10-16-22-24	Dense, olive SAND, trace silt, some gravel.			
14								

MINOR CONSTITUENT PROPORTIONS USED:

Trace	0 to 10%	Some	20 to 35%
Little	10 to 20%	And	35 to 50%

REMARKS: Water = 3.5 feet on pond side of wall



NOTES

HORIZONTAL PERMEABILITY OF DAM EMBANKMENT SAND FILL WAS MEASURED TO BE 17.9 FEET PER DAY BASED ON BOREHOLE PERMEABILITY TESTING PERFORMED ON JUNE 11, 2009 BY FUSS & O'NEILL.

SCALE:		
HORZ.: 1" = 20'		
VERT.: 1" = 20'		
DATUM:		
HORZ.:		
VERT.:		
0 10 20 GRAPHIC SCALE		

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NEPONSET RIVER LAND HOLDING ASSOCIATION

SEEPAGE ANALYSIS - FULL POND

WILLETT POND DAM PHASE II INVESTIGATION

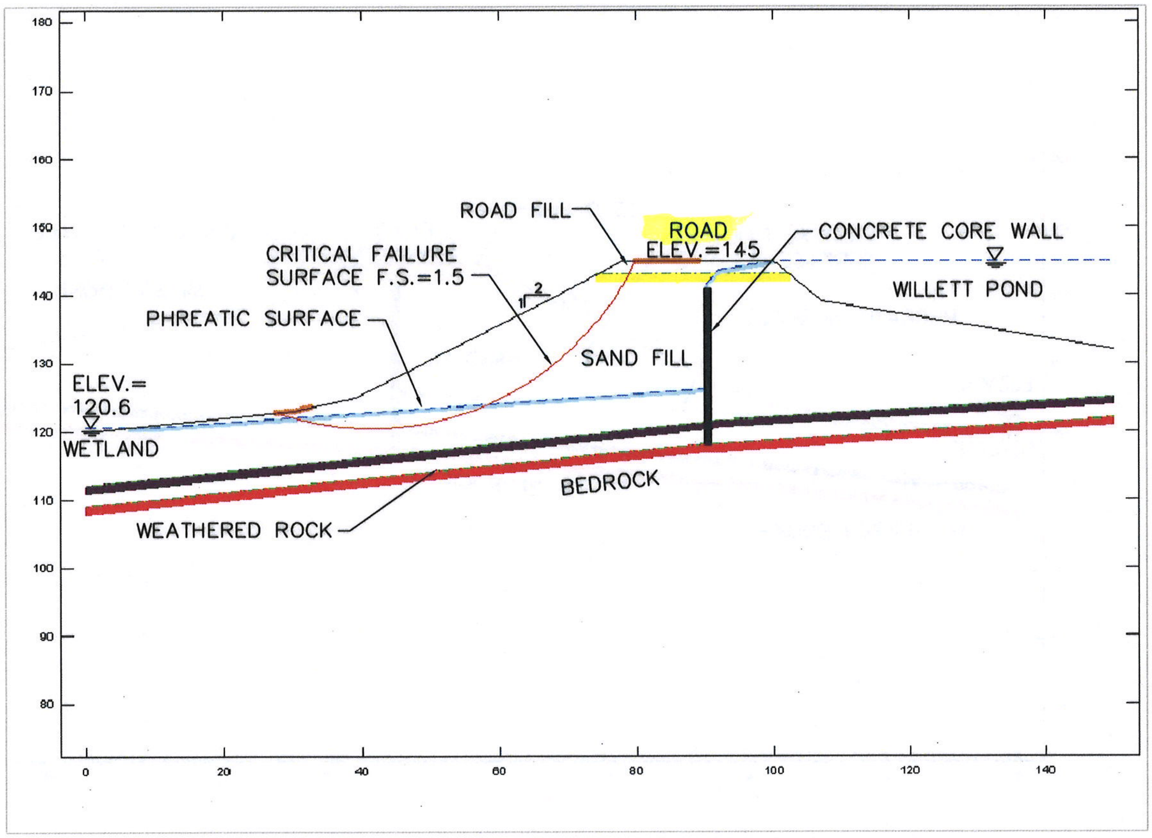
NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323.A20
DATE: OCTOBER 2009

B-1

File Path: G:\P2005\1323\A20\Geotech\Willett Pond Full.dwg, Layout: Seepage-Full Pond
Date: Fri, Dec 11, 2009 - 7:52 PM User: ccullen

UCS: MS VIEW: LMAN: CTB:



File Path: G:\P2005\1323\A20\Geotech\Willett Pond Full.dwg, Layout: STABILITY FULL POND
Date: Fri, Dec 11, 2009 - 7:52 PM User: ccullen

UCS: | MS VIEW: | LMAN: | CTB:

SCALE:
HORZ.: 1" = 20'
VERT.: 1" = 20'
DATUM:
HORZ.:
VERT.:
0 10 20
GRAPHIC SCALE

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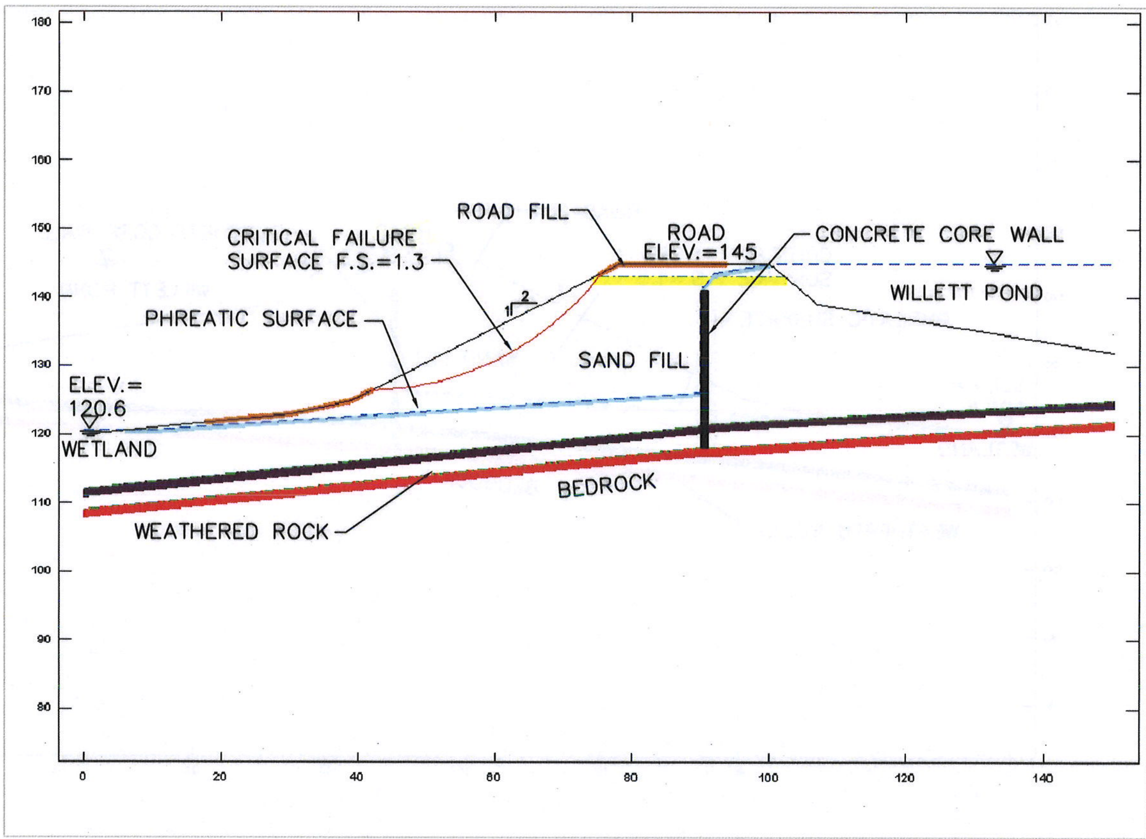
NEPONSET RIVER LAND HOLDING ASSOCIATION
DAM SLOPE STABILITY
FULL POND - TOE FAILURE
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323 A20
DATE: OCT. 2009

B-2



SCALE:
HORZ.: 1" = 20'
VERT.: 1" = 20'
DATUM:
HORZ.:
VERT.:
0 10 20
GRAPHIC SCALE

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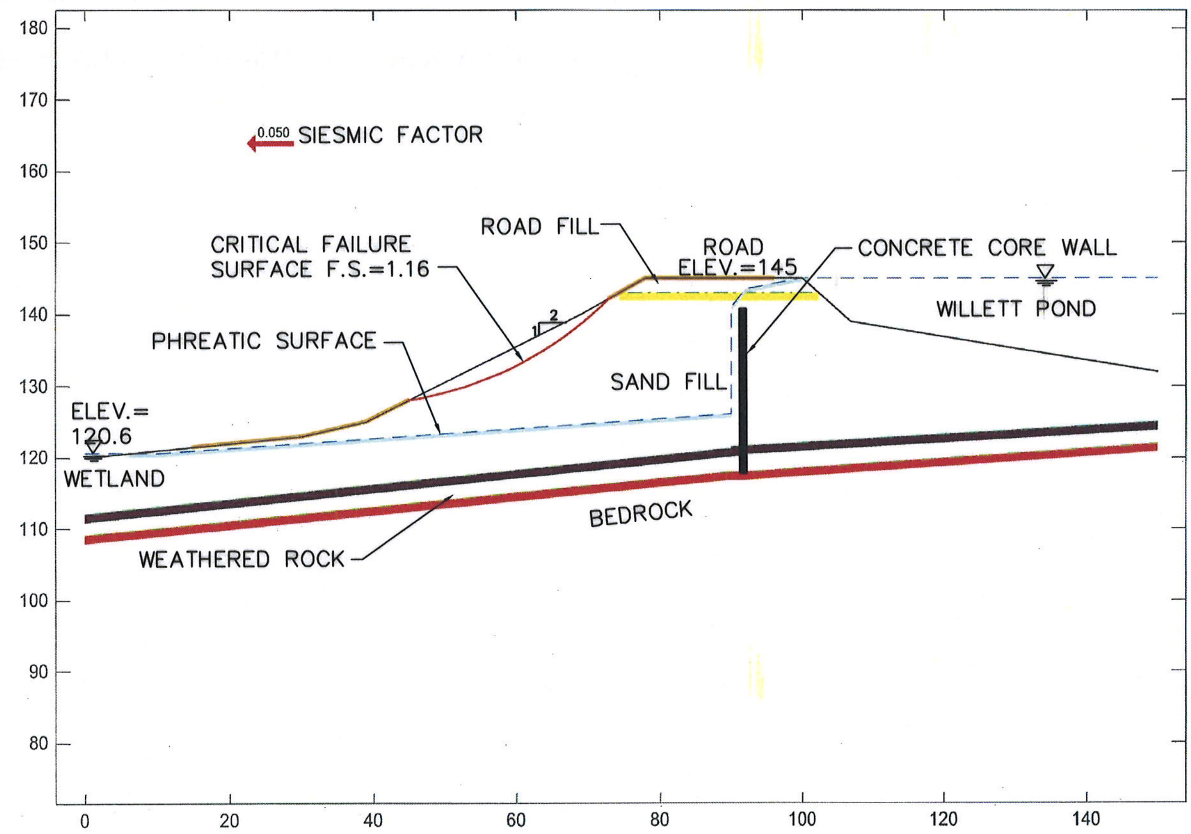
NEPONSET RIVER LAND HOLDING ASSOCIATION
DAM STABILITY
FULL POND - SHALLOW SLOUGH FAILURE
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323.A20
DATE: OCT. 2009

B-3

File Path: G:\P2005\1323\A20\Geotech\Willett Pond Full.dwg, Layout: STABILITY FULL POND SHALLOW FAILURE
Date: Fri, Dec 11, 2009 - 7:52 PM User: ccullen

UCS: MS VIEW: LMAN: CTB:



File Path: G:\P2005\1323\A20\Geotech\Willett Pond Full.dwg, Layout: Stability Full Pond Seismic
Date: Fri, Dec 11, 2009 - 7:53 PM User: ccullen

SCALE:
HORZ.: 1" = 20'
VERT.: 1" = 20'
DATUM:
HORZ.:
VERT.:
0 10 20
GRAPHIC SCALE

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NEPONSET RIVER LAND HOLDING ASSOCIATION
DAM STABILITY
FULL POND - SEISMIC CONDITION
WILLETT POND DAM PHASE II INVESTIGATION
NORWOOD/WALPOLE MASSACHUSETTS

PROJ. No.: 20051323.A20
DATE: OCT. 2009

B-4

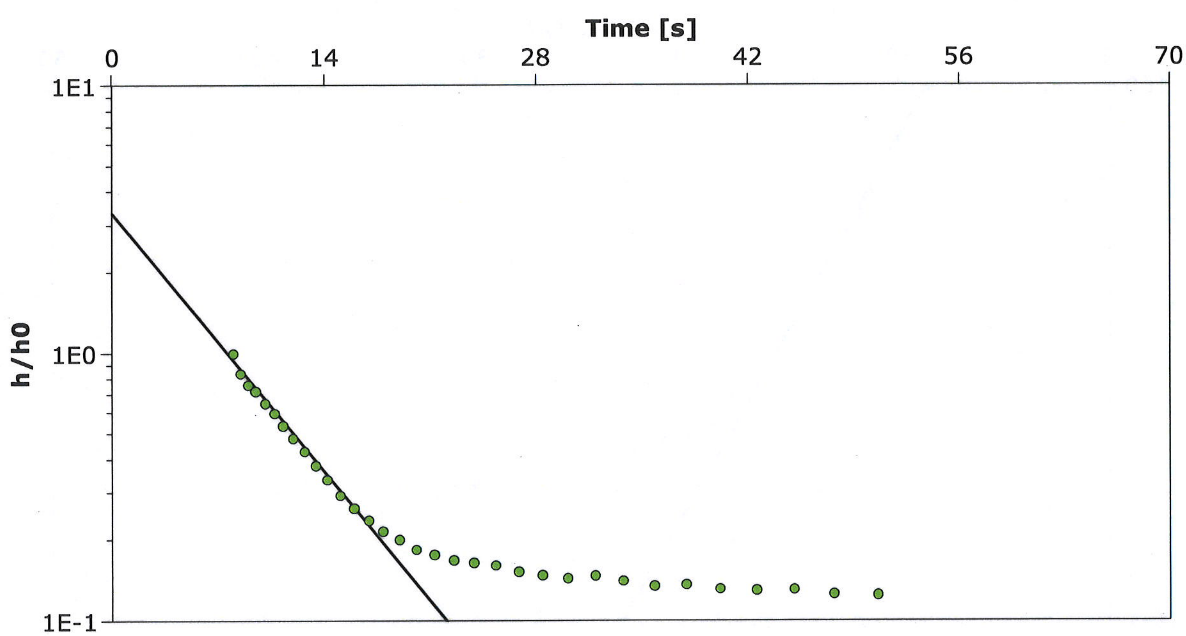
FUSS & O'NEILL

Appendix C

Field Hydraulic Conductivity Test Results

Fuss & O'Neill 146 Hartford Road Manchester, CT 06040	Slug Test Analysis Report Project: Willett Pond Dam Number: 20051323A20 Client:

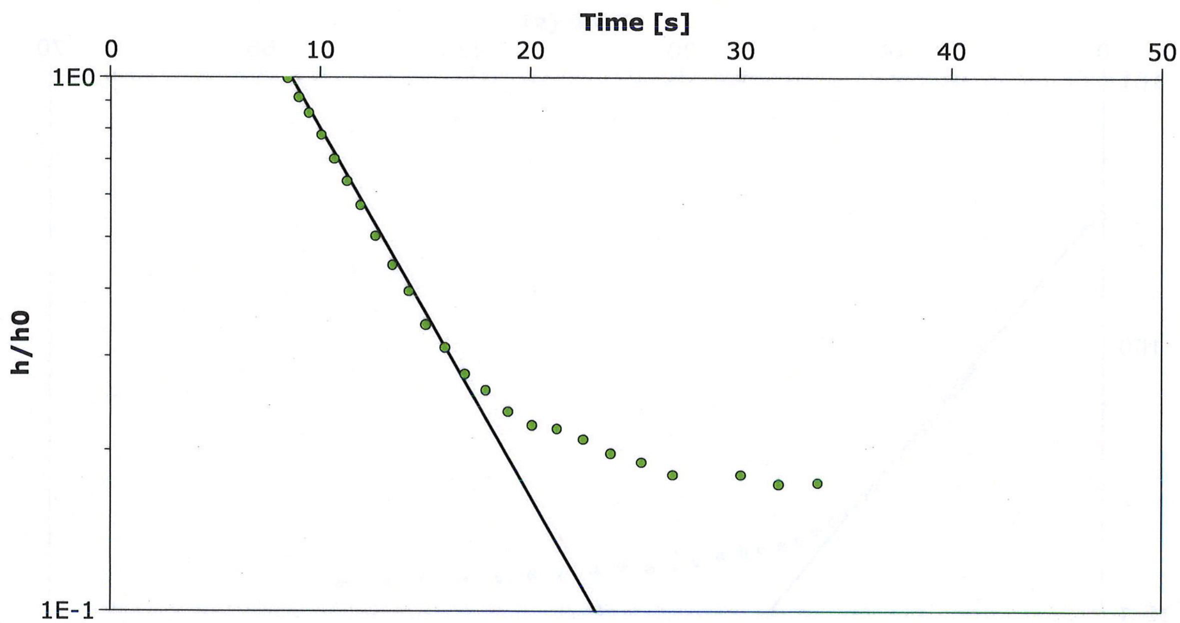
Location: Norwood, MA	Slug Test: MW-C R1	Test Well: MW-C
Test conducted by: KAK		Test date: 6/11/2009
Analysis performed by: KAK	MW-CR1	Date: 6/11/2009
Aquifer Thickness: 30.00 ft		



Calculation after Bouwer && Rice

Observation well	K [ft/d]	
MW-C	1.79×10^1	

Fuss & O'Neill 146 Hartford Road Manchester, CT 06040	Slug Test Analysis Report Project: Willett Pond Dam Number: 20051323A20 Client:	
Location: Norwood, MA	Slug Test: MW-C R2	Test Well: MW-C
Test conducted by: KAK		Test date: 6/11/2009
Analysis performed by: KAK	MW-C R2	Date: 6/11/2009
Aquifer Thickness: 30.00 ft		

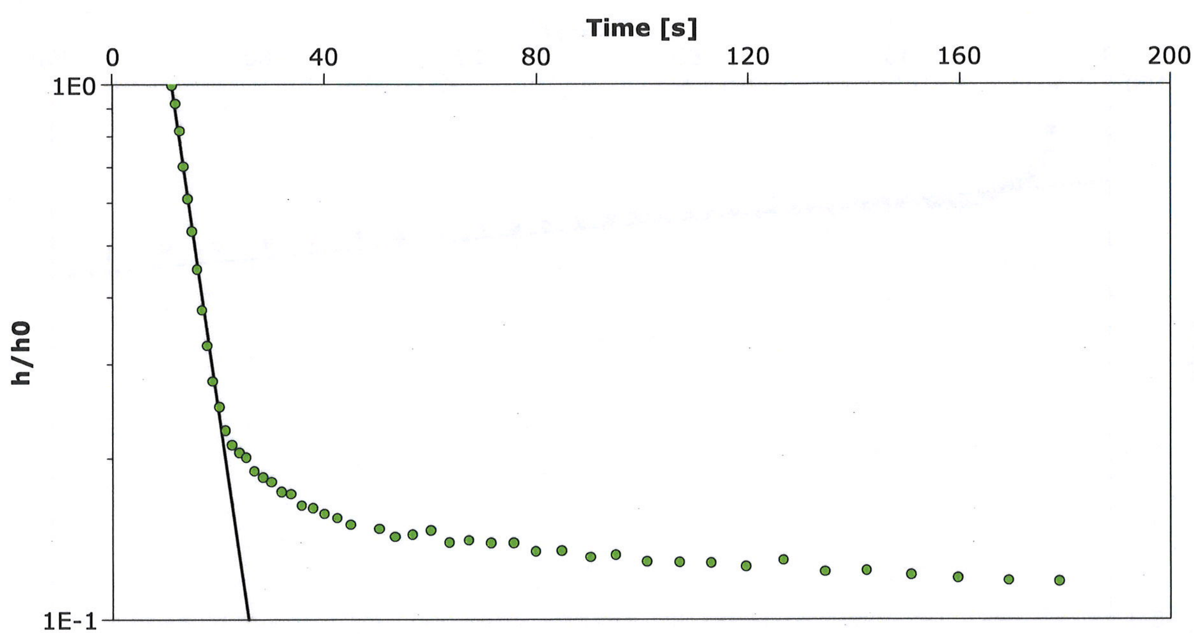


Calculation after Bouwer && Rice

Observation well	K [ft/d]	
MW-C	1.79×10^{1}	

Fuss & O'Neill **146 Hartford Road** **Manchester, CT 06040**	**Slug Test Analysis Report** Project: Willett Pond Dam Number: 20051323A20 Client:

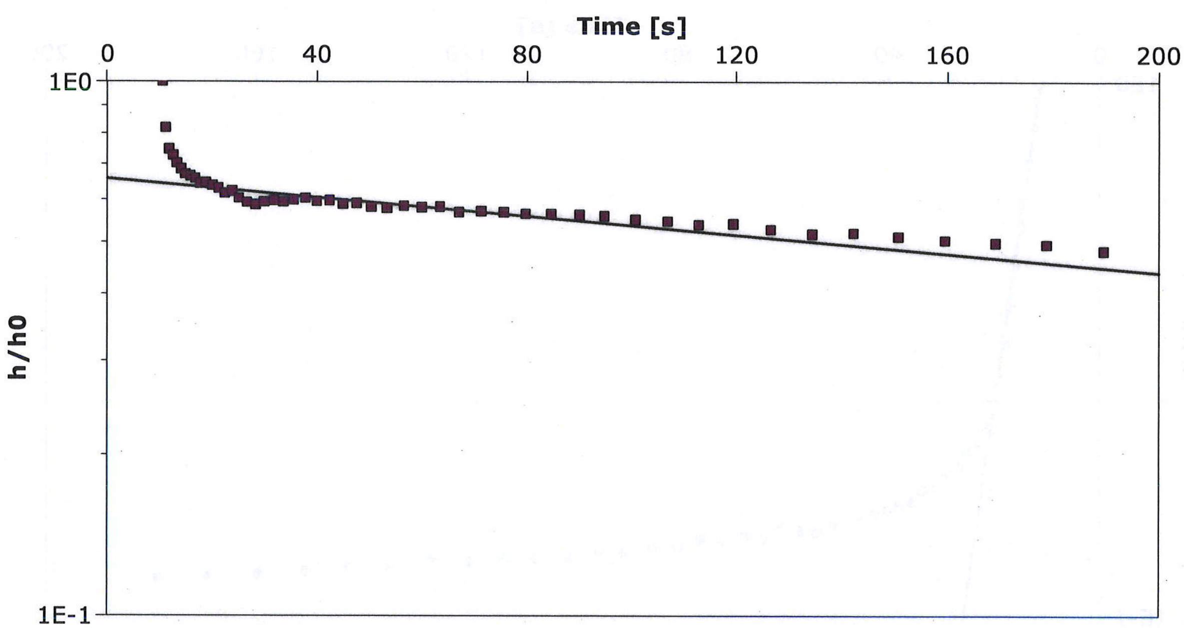
Location: Norwood, MA	Slug Test: MW-C R3	Test Well: MW-C
Test conducted by: KAK		Test date: 6/11/2009
Analysis performed by: KAK	MW-C R3	Date: 6/11/2009
Aquifer Thickness: 30.00 ft		



Calculation after Bouwer && Rice

Observation well	K [ft/d]	
MW-C	1.79×10^{1}	

Fuss & O'Neill 146 Hartford Road Manchester, CT 06040	Slug Test Analysis Report Project: Willett Pond Dam Number: 20051323A20 Client:	
Location: Norwood, MA	Slug Test: MW-E R1	Test Well: MW-E
Test conducted by: KAK		Test date: 6/11/2009
Analysis performed by: KAK	MW-E R1	Date: 6/11/2009
Aquifer Thickness: 20.00 ft		

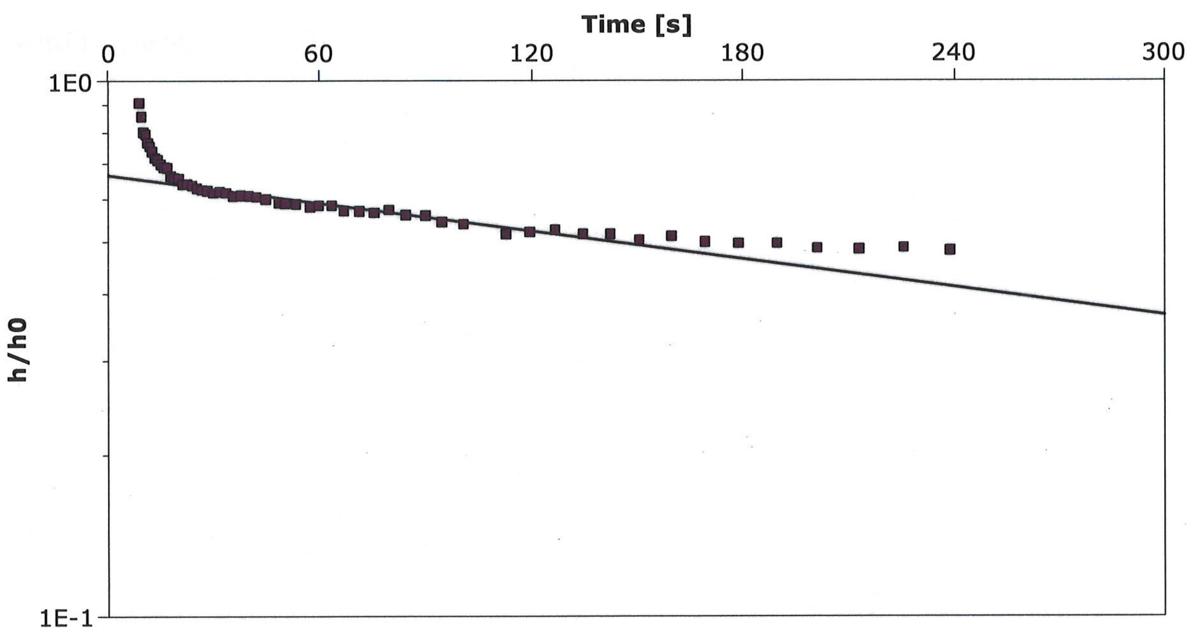


Calculation after Bouwer && Rice

Observation well	K [ft/d]	
MW-E	1.92×10^{-1}	

Fuss & O'Neill 146 Hartford Road Manchester, CT 06040	Slug Test Analysis Report Project: Willett Pond Dam Number: 20051323A20 Client:

Location: Norwood, MA	Slug Test: MW-E R2	Test Well: MW-E
Test conducted by: KAK		Test date: 6/11/2009
Analysis performed by: KAK	MW-E R2	Date: 6/11/2009
Aquifer Thickness: 20.00 ft		



Calculation after Bouwer && Rice

Observation well	K [ft/d]	
MW-E	1.90×10^{-1}	

FUSS & O'NEILL

Appendix D

Cost Estimate Data

FUSS & O'NEILL, INC.

146 HARTFORD ROAD
MANCHESTER, CONNECTICUT

OPINION OF CONSTRUCTION COST		DATE PREPARED :	24-Mar-11
Type:	**Order of Magnitude**		
PROJECT :	Willet Pond Dam	BASIS : Experience, RSMeans, NEI	
LOCATION :	Norwood /Walpole		
DESCRIPTION:	ALT 1-Maintain Current Water elev., Raise Dike, Short Weir		
DRAWING NO.	20051323A20STP002P_ALT1	ESTIMATOR : CJC	CHECKED BY PWM

OPINION OF CONSTRUCTION COST - ORDER OF MAGNITUDE: An opinion of cost made without detailed engineering data. Costs may be estimated by comparison with similar projects. It is normally expected that an estimate of this type would be accurate within plus 50% or minus 30%. Since Fuss & O'Neill has no control over the cost of labor, materials, equipment or services furnished by others, or over the Contractor('s) methods of determining prices, or over competitive bidding or market conditions, Fuss & O'Neill's opinion of probable Total Project Costs and Construction Costs are made on the basis of Fuss & O'Neill's experience and qualifications and represent Fuss & O'Neill's best judgment as an experienced and qualified professional engineer, familiar with the construction industry; but Fuss & O'Neill cannot and does not guarantee that proposals, bids or actual Total Project or Construction Costs will not vary from opinions of probable cost prepared by Fuss & O'Neill. If prior to the bidding or negotiating Phase the Owner wishes greater assurance as to Total Project or Construction Costs, the Owner should employ an independent cost estimator.

ITEM NO.	ITEM	UNIT MEAS.	NO. UNITS	PER UNIT	TOTAL COST
1	**SITE ACCESS WORK**				
	Restoration	L.S.	1	$25,000	$25,000
	Site Access	L.S.	1	$45,000	$45,000
2	**SOIL AND EROSION CONTROL**				
	Sediment and Erosion Control	L.S.	1	$7,500	$7,500
3	**WATER CONTROL**				
	Control of Water	L.S.	1	$100,000	$100,000
4	**BRIDGE ENLARGEMENT EARTHWORK (Downstream)**				
	Excavation	C.Y.	800	$35	$28,000
	Slab Demolition	L.S.	1	$60,000	$60,000
	Excavation under Bridge	C.Y.	160	$90	$14,400
	Rock removal	C.Y.	100	$100	$10,000
	Spillway Concrete	C.Y.	100	$700	$70,000
	Bridge Underpinning	L.S.	1	$250,000	$250,000
	Concrete Walls & Channel	C.Y.	360	$700	$252,000
5	**WEIR CONSTRUCTION (Upstream)**				
	OGEE Spillway and spillway Apron	C.Y.	700	$700	$490,000
	Sediment Excavation	C.Y.	1000	$35	$35,000
	Rock Removal	C.Y.	100	$100	$10,000
	Sediment Disposal (1,000 CY, if contaminated)	Ton	0	$100	$0
6	**DIKE EARTHWORK**				
	Dike Raising/Compaction/Seeding	C.Y.	2800	$35	$98,000
7	**GENERAL**				
	Mobilization & Demobilization	L.S.	1	$15,000	$15,000
	Survey, Construction Stakeout	L.S.	1	$10,000	$10,000
	Traffic Control	L.S.	1	$15,000	$15,000
	Overhead, Bonds	L.S.	1	$75,000	$75,000
	General Contractor (Office, rewstrooms, etc.)	L.S.	1	$25,000	$25,000
8	**ENGINEERING, PERMITTING, CONSTR. ADMIN.**	L.S.	1	$220,000	$220,000
	TOTAL				$1,854,900

Range (-30% to +50%) $1,299,000 to $2,783,000

FUSS & O'NEILL, INC.
146 HARTFORD ROAD
MANCHESTER, CONNECTICUT

OPINION OF CONSTRUCTION COST		DATE PREPARED :	24-Mar-11
Type:	**Order of Magnitude**		
PROJECT :	Willet Pond Dam	BASIS : Experience, RSMeans, NEI	
LOCATION :	Norwood /Walpole		
DESCRIPTION:	ALT 2 - Lower Water Elevation, Short Ogee Weir		
DRAWING NO. :	20051323A20STP002P_ALT2	ESTIMATOR : CJC	CHECKED BY : PWM

OPINION OF CONSTRUCTION COST - ORDER OF MAGNITUDE: An opinion of cost made without detailed engineering data. Costs may be estimated by comparison with similar projects. It is normally expected that an estimate of this type would be accurate within plus 50% or minus 30%. Since Fuss & O'Neill has no control over the cost of labor, materials, equipment or services furnished by others, or over the Contractor('s) methods of determining prices, or over competitive bidding or market conditions, Fuss & O'Neill's opinion of probable Total Project Costs and Construction Costs are made on the basis of Fuss & O'Neill's experience and qualifications and represent Fuss & O'Neill's best judgment as an experienced and qualified professional engineer, familiar with the construction industry; but Fuss & O'Neill cannot and does not guarantee that proposals, bids or actual Total Project or Construction Costs will not vary from opinions of probable cost prepared by Fuss & O'Neill. If prior to the bidding or negotiating Phase the Owner wishes greater assurance as to Total Project or Construction Costs, the Owner should employ an independent cost estimator.

ITEM NO.	ITEM	UNIT MEAS.	NO. UNITS	PER UNIT	TOTAL COST
1	**SITE ACCESS WORK**				
	Site Access	L.S.	1	$35,000	$35,000
	Restoration	L.S.	1	$25,000	$25,000
2	**SOIL AND EROSION CONTROL**				
	Sediment and Erosion Control	L.S.	1	$7,500	$7,500
3	**WATER CONTROL**				
	Control of Water	L.S.	1	$100,000	$100,000
4	**BRIDGE ENLARGEMENT EARTHWORK (Downstream)**				
	Excavation	C.Y.	800	$35	$28,000
	Slab Demolition	L.S.	1	$60,000	$60,000
	Excavation under Bridge	C.Y.	160	$90	$14,400
	Rock Removal	C.Y.	100	$100	$10,000
	Spillway Concrete	C.Y.	100	$700	$70,000
	Bridge Underpinning	L.S.	1	$250,000	$250,000
	Concrete Walls & Channel	C.Y.	360	$700	$252,000
5	**WEIR CONSTRUCTION (Upstream)**				
	Rock removal	C.Y.	100	$100	$10,000
	Sediment excavation	C.Y.	1000	$35	$35,000
	OGEE Spillway and Spillway Apron	C.Y.	700	$700	$490,000
	Sediment Disposal (1,000 CY, if contaminated)	Ton	0	$100	$0
6	**GENERAL**				
	Mobilization & Demobilization	L.S.	1	$15,000	$15,000
	Survey, Construction Stakeout	L.S.	1	$10,000	$10,000
	Traffic Control	L.S.	1	$15,000	$15,000
	Overhead, Bonds	L.S.	1	$75,000	$75,000
	General Contractor (Office, rewstrooms, etc.)	L.S.	1	$25,000	$25,000
7	**ENGINEERING, PERMITTING, CONSTR. ADMIN.**	L.S.	1	$200,000	$200,000
	TOTAL				$1,726,900

Range (-30% to +50%)	$1,209,000	to	$2,591,000

FUSS & O'NEILL, INC.
146 HARTFORD ROAD
MANCHESTER, CONNECTICUT

DRAFT OPINION OF CONSTRUCTION COST		DATE PREPARED :	13-Apr-11
Type: Order of Magnitude			
PROJECT : Willett Pond Dam	BASIS : Experience, RSMeans , NEI		
LOCATION : Norwood /Walpole			
DESCRIPTION: ALT 3 - Long Ogee Weir, Maintain Lake Level, Dike Elevation			
DRAWING NO. 20051323A20STP002P_ALT3 Revised	ESTIMATOR : CJC	CHECKED BY : PWM	

OPINION OF CONSTRUCTION COST - ORDER OF MAGNITUDE: An opinion of cost made without detailed engineering data. Costs may be estimated by comparison with similar projects. It is normally expected that an estimate of this type would be accurate within plus 50% or minus 30%. Since Fuss & O'Neill has no control over the cost of labor, materials, equipment or services furnished by others, or over the Contractor('s) methods of determining prices, or over competitive bidding or market conditions, Fuss & O'Neill's opinion of probable Total Project Costs and Construction Costs are made on the basis of Fuss & O'Neill's experience and qualifiations and represent Fuss & O'Neill's best judgment as an experienced and qualified professional engineer, familiar with the construction industry;but Fuss & O'Neill cannot and does not guarantee that proposals, bids or actual Total Project or Construction Costs will not vary from opinions of probable cost prepared by Fuss & O'Neill. If prior to the bidding or negotiating Phase the Owner wishes greater assurance as to Total Project or Construction Costs, the Owner should employ an independent cost estimator.

ITEM NO.	ITEM	UNIT MEAS.	NO. UNITS	PER UNIT	TOTAL COST
1	**SITE ACCESS WORK**				
	Access	L.S.	1	$35,000	$35,000
	Restoration	L.S.	1	$25,000	$25,000
2	**SOIL AND EROSION CONTROL**				
	Sediment and Erosion Control	L.S.	1	$7,500	$7,500
3	**WATER CONTROL**				
	Control of Water	L.S.	1	$275,000	$275,000
4	**BRIDGE ENLARGEMENT EARTHWORK (Downstream)**				
	Excavation	C.Y.	800	$35	$28,000
	Slab Demolition	L.S.	1	$60,000	$60,000
	Excavation under Bridge	C.Y.	160	$90	$14,400
	Rock Removal	C.Y.	100	$100	$10,000
	Spillway Concrete	C.Y.	100	$700	$70,000
	Bridge Underpinning	L.S.	1	$250,000	$250,000
	Concrete Walls & Channel	C.Y.	360	$700	$252,000
5	**WEIR CONSTRUCTION (Upstream)**				
	Rock Removal	C.Y.	100	$100	$10,000
	Sediment Excavation for Footings and Apron	C.Y.	3,600	$35	$126,000
	OGEE Spillway	C.Y.	2,200	$700	$1,540,000
	Sediment Disposal	Ton	0	$100	$0
6	**GENERAL**				
	Mobilization & Demobilization	L.S.	1	$15,000	$15,000
	Survey, Construction Stakeout	L.S.	1	$10,000	$10,000
	Traffic Control	L.S.	1	$15,000	$15,000
	Overhead, Bonds	L.S.	1	$75,000	$75,000
	General Contractor (Office, rewstrooms, etc.)	L.S.	1	$25,000	$25,000
7	**ENGINEERING, PERMITTING, CONSTR. ADMIN.**				
		L.S.	1	$430,000	$430,000
	Total				$3,272,900

Range (-30% To +50%) $ 2,292,000 to $ 4,910,000

OPTIONS	COST OF ALTERNATIVE	COST DIFFERENCE	ADJUSTED TOTAL COST
1. Minimal Up Stream Rock Removal (50 CY)	5,000	-$5,000	$3,267,900
2. Six-Foot Drawdown	140,000	-$135,000	$3,137,900
3. Contaminated Sediment (3,600 CY, 5,200 tons)	520,000	$520,000	$3,792,900
1and 2	145,000	-$140,000	$3,132,900
2 and 3	660,000	$385,000	$3,657,900
1and 3	525,000	$515,000	$3,787,900
All 3	665,000	$380,000	$3,652,900

FUSS & O'NEILL, INC.
146 HARTFORD ROAD
MANCHESTER, CONNECTICUT

OPINION OF CONSTRUCTION COST		DATE PREPARED :	13-Apr-11
Type:	**Order of Magnitude**		
PROJECT :	Willet Pond Dam	BASIS : Experience, RSMeans, NEI	
LOCATION :	Norwood /Walpole		
DESCRIPTION:	Flood Gate Alternatives		
DRAWING NO. :		ESTIMATOR : CJC	CHECKED BY : PWM

OPINION OF CONSTRUCTION COST - ORDER OF MAGNITUDE: An opinion of cost made without detailed engineering data. Costs may be estimated by comparison with similar projects. It is normally expected that an estimate of this type would be accurate within plus 50% or minus 30%. Since Fuss & O'Neill has no control over the cost of labor, materials, equipment or services furnished by others, or over the Contractor('s) methods of determining prices, or over competitive bidding or market conditions, Fuss & O'Neill's opinion of probable Total Project Costs and Construction Costs are made on the basis of Fuss & O'Neill's experience and qualifications and represent Fuss & O'Neill's best judgment as an experienced and qualified professional engineer, familiar with the construction industry; but Fuss & O'Neill cannot and does not guarantee that proposals, bids or actual Total Project or Construction Costs will not vary from opinions of probable cost prepared by Fuss & O'Neill. If prior to the bidding or negotiating Phase the Owner wishes greater assurance as to Total Project or Construction Costs, the Owner should employ an independent cost estimator.

ITEM NO.	ITEM	UNIT MEAS.	NO. UNITS	PER UNIT	TOTAL COST
1	**SITE ACCESS WORK**				
	Site Access	L.S.	1	$35,000	$35,000
	Restoration	L.S.	1	$25,000	$25,000
2	**SOIL AND EROSION CONTROL**				
	Sediment and Erosion Control	L.S.	1	$7,500	$7,500
3	**WATER CONTROL**				
	Control of Water	L.S.	1	$75,000	$75,000
4	**BRIDGE ENLARGEMENT EARTHWORK (Downstream)**				
	Excavation	C.Y.	800	$20	$16,000
	Slab Demolition	L.S.	1	$60,000	$60,000
	Excavation Under Bridge	C.Y.	160	$90	$14,400
	Rock Removal	C.Y.	100	$100	$10,000
	Slab Concrete	C.Y.	100	$700	$70,000
	Bridge Underpinning	L.S.	1	$250,000	$250,000
	Concrete Walls & Channel	C.Y.	360	$700	$252,000
5	**GATE CONSTRUCTION (Upstream)**				
	Additional Concrete Support	C.Y.	50	$700	$35,000
	Hinged Bottom Gate	L.S.	1	$325,000	$325,000
	Gate Automation Equipment	L.S.	1	$50,000	$50,000
	Back-up Generator	L.S.	1	$6,000	$6,000
	Rock Removal	C.Y.	100	$100	$10,000
5	**GENERAL**				
	Mobilization & Demobilization	L.S.	1	$15,000	$15,000
	Survey, Construction Stakeout	L.S.	1	$10,000	$10,000
	Traffic Control	L.S.	1	$15,000	$15,000
	Overhead, Bonds	L.S.	1	$75,000	$75,000
	General Contractor (Office, restrooms, etc.)	L.S.	1	$25,000	$25,000
6	**ENGINEERING, PERMITTING. CONSTR. ADMIN.**	L.S.	1	$250,000	$250,000
	TOTAL				$1,630,900

esc to 2017 (+10%) = $1.8m

	Range (-30% to +50%)		$1,142,000	to	$2,447,000

SUBSTITUTE FOR HINGED BOTTOM GATE					
	Tainter/Vertical Lift Gate	L.S.	1	$175,000.00	$175,000
	TOTAL				$1,480,900

	Range (-30% to +50%)		$1,037,000	to	$2,222,000

plus 50% $2.7m

plus 20% $2.16m

incl escal thru 2016

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Flood Gate Option

Cost Estimate

Draft Final Rpt
4/25/11

Mustonen, John

From:	Ian Cooke <cooke@neponset.org>
Sent:	Wednesday, June 01, 2011 11:02 AM
To:	Leo Cesareo; Mustonen, John; John Glossa; 'Tom Palmer (home)'
Cc:	Brendan McLaughlin; 'Taber Keally'; da_brookfield@brookfieldengineering.com; David Brookfield
Subject:	Final Meeting with Fuss and O'Neil June 17
Attachments:	Report_04-22-11.pdf

Hello Leo, John, et. al,

Attached for your review and comment is the DRAFT final report on the Willett Pond Dam. This version combines their initial work with the additional work on floodgates and other options. I have not yet read it through myself. Please look it over and forward any comments or suggestions you may have to me by next Wednesday June 8. I will then combine all the feedback into a single set of comments that I will forward to Fuss and O'Neil by June 10. If there are other people you would like to have review it, feel free to forward it, HOWEVER given that it is a DRAFT please be judicious in who you share it with.

The extension of our contract with Fuss and O'Neil provides for a final debrief meeting with them. We have scheduled this for 1 pm Friday June 17, at their office in CT (about a two hour drive each way). We do not currently have plans to have them make another presentation to the full pond community, so this will be our final opportunity to ask questions of F and O and seek their advice on strategy going forward (unless we want to pay them more).

Tom and I are planning to go to the meeting, and if any of you would like to tag along in person, or join in by speaker phone, we would be happy to have you. Again, if there are others you think should participate, let me know, as we want to make sure that the group of participants is reasonably small in order to keep it productive.

Thanks!

Ian Cooke
Executive Director
Neponset River Watershed Association
2173 Washington Street
Canton, MA 02021
781-575-0354 x 305