Dewberry

ARCHITECTS ENGINEERS CONSULTANTS

Colburn Street Dam

Town of Dedham, Massachusetts

Phase II Investigation Report

March 31, 2016

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PREFACE

The assessment of the general condition of the dam reported herein was based upon available data and visual inspections, subsurface investigations, detailed hydrologic and hydraulic and stability analyses.

In reviewing this report, it should be realized that the reported condition of the dam was based on observations of field conditions at the time of inspection, along with data available to the inspection team and other information collected as part of the evaluation. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under normal operating environment of the structure.

It is critical to note that the condition of the dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the reported condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

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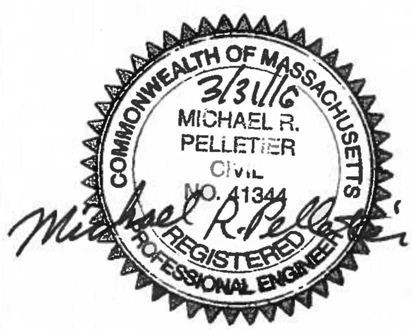

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1.0 PROJECT INFORMATION

1.1 Introduction

Dewberry Engineers Inc (Dewberry), has been retained by the Town of Dedham to perform a Phase II assessment including geotechnical, land survey, hydraulic/hydrologic analysis and conceptual design services for the Colburn Street dam on Mother Brook in Dedham, Massachusetts. See Figure 1-1 for locus map showing Colburn Street Dam location.

A Phase 1 Inspection was performed in May 2006 by Weston and Sampson. A follow-up inspection was performed in July 2013 by GZA GeoEnvironmental, Inc. and is included as Attachment A to this report.

We understand that based on the July 2013 inspection, the condition of the dam was downgraded to “fair” and the hazard classification of the dam was upgraded to “Significant” from “Low” based upon a visual assessment. The downgraded condition of the dam was based on several deficiencies including downstream scour, seepage through the masonry face of the dam, large voids between masonry stones and leakage through the stop logs in the sluiceway. The upgraded hazard classification was based upon a visual assessment without the benefit of a hydraulic or breach analysis.

1.2 Project Description

The purpose of this project is to provide land survey, subsurface investigations and geotechnical services in order to analyze the existing conditions, develop potential alternatives for repair or replacement of the dam, recommend a preferred alternative and prepare conceptual design plans and cost estimate for the recommended alternative. Additionally, this project included performing scour, stability, and dam break analyses.

1.3 Dam Data

Dam Name: Colburn Street Dam
Dam Owner: Town of Dedham, Massachusetts
Nat. ID Number: MA 02571
Hazard Potential: Significant
Size Classification: Small
Location of Dam (town): Dedham, MA
Coordinate location (lat, long): 42.2490°N, -71.1598°W
Type of Dam: Recreation

1.4 Dam Description

Colburn Street Dam is a stone masonry structure located on the Mother Brook in Dedham. The dam is approximately 100 feet in length with a slightly arched (bowed upstream) configuration. The height is a minimum of 9 feet high with a maximum height of 13 feet. Mother Brook is a stream which conveys water from the Charles River to the Neponset River. The dam is located adjacent to Condon Park, which has a baseball field and small playground, and the major use of the reservoir appears to be recreational. During periods of lower flow, water passes through a sluiceway notch, which is about 2 feet deep and 4 feet wide with the stop logs installed. During periods of higher flow, the dam is inundated.

Colburn Street Dam (Looking South, Downstream)

Colburn Street Dam (Looking South, Upstream)

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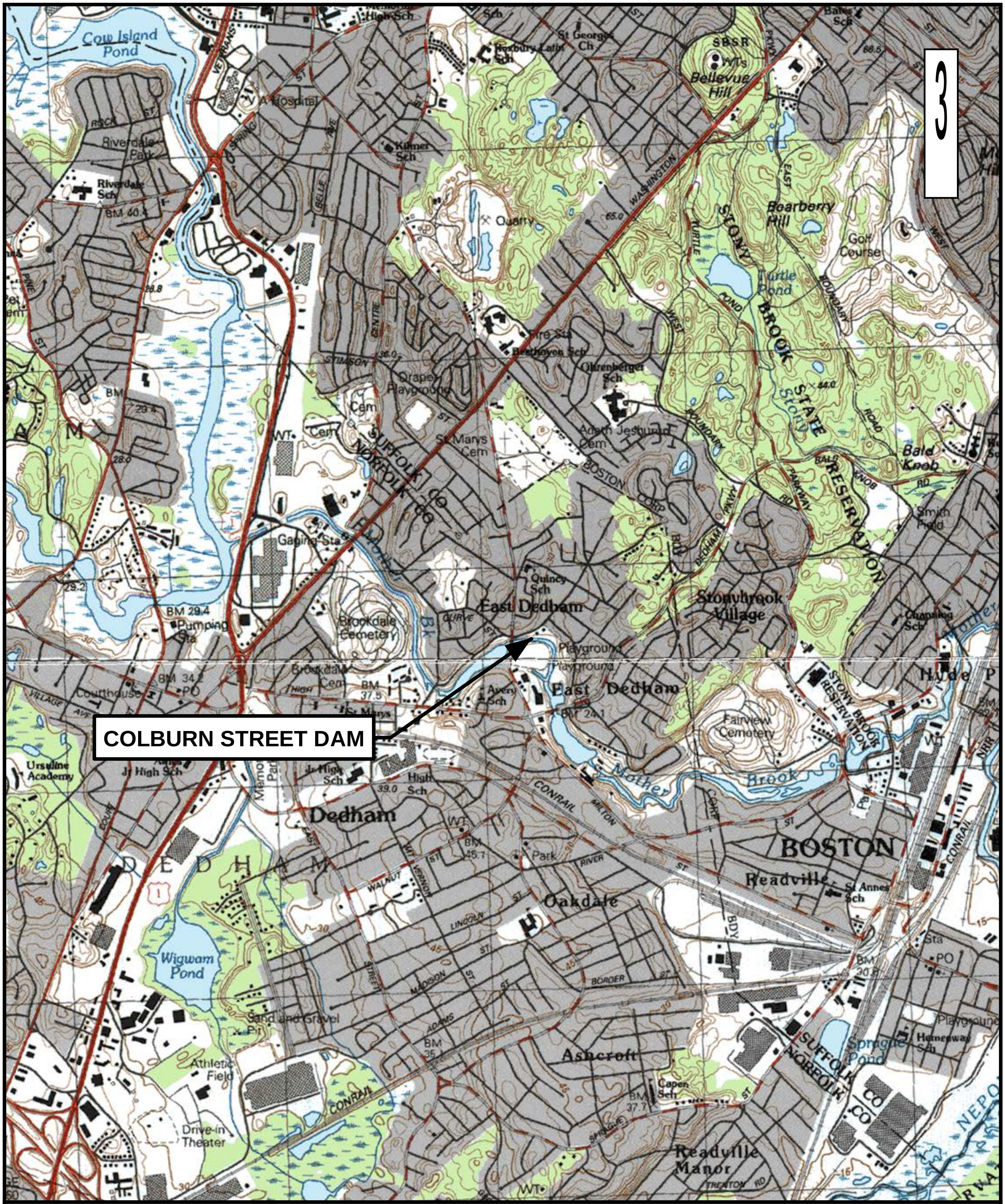
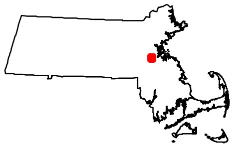


DAM LOCATION



MASSACHUSETTS

FIGURE 1
DEDHAM, MA
COLBURN STREET DAM - MA02571
USGS QUAD: NEWTON

LOCUS MAP

SCALE 1:25000
SOURCE: USGS

2.0 HYDROLOGIC AND HYDRAULIC ASSESSMENT

2.1 Software Selection

The dam breach was performed using HEC-RAS version 4.1 which is the current fully released version of the US Army Corps of Engineers' software to perform dam breach analysis with an unsteady flow simulation. HEC-RAS is currently one of the most widely used models for dam breach analysis. The governing equations used in this unsteady flow analysis are the conservation of mass (continuity) and the momentum equations derived from the full equations of motion (St. Venant equations). For the dam breach analysis, the reservoir outflow was dynamically routed downstream. For this analysis we fully developed the model, including cross-sections spaced as shown in the Figure 2-1 below using a combination of Dedham Topographic plans, new field survey and Massachusetts LiDAR data.

2.2 Dam Breach Analysis

The Colburn Street Dam is located on Mother Brook approximately 1.25 miles downstream of the Mother Brook Diversion at Charles River. The model extends from approximately 675 feet upstream of Maverick Street to approximately 565 feet downstream of Centennial Dam. The location of the model can be seen below in Figure 2-1 which is shown in the HEC-RAS Geometry Editor.

Per the Massachusetts Office of Dam Safety, both the Usual (Sunny Day) and Unusual (100-yr) loading conditions were modeled. The upstream extent was set at a location that would be a sufficient distance upstream of the flood pool for both events and the downstream extent was set such that flooding extents were contained within the FEMA Effective Zone AE extents. Figure 2-2 below shows the HEC-RAS flood profile for both the Sunny Day and 100-yr non-breach scenarios and Table 2-1 shows the pool elevations.

Table 2-1: Model Loading Conditions

Loading Condition	Scenario	U/S Pool Elevation (ft)	
		Geotechnical Report	Model
Usual	Sunny Day	78.2	78.23
Unusual	100-yr	81.2	81.10

A constant inflow of 30 cubic feet per second (cfs) was used to represent the Usual (Sunny Day) loading condition as this produced pool elevations consistent with the Geotechnical Report. A peak value of 1,509 cfs was used for the Unusual (100-yr) loading condition and it was taken from the Norfolk County, Massachusetts FEMA Effective FIS Report dated July 16, 2015 (FIS Study #25021CV001C). A simplified triangular hydrograph was used to transform the peak flow into an unsteady inflow as seen in Figure 2-3:

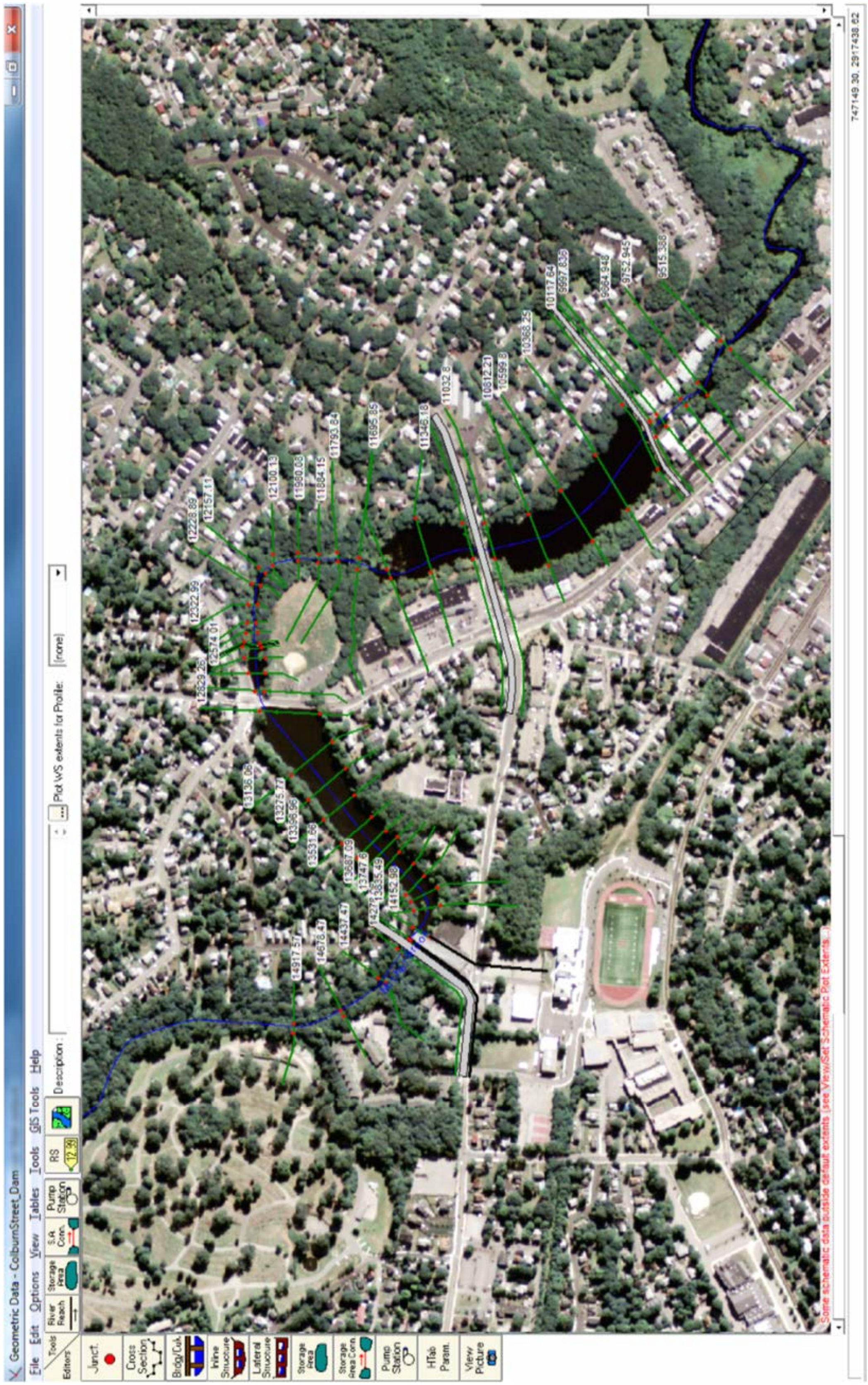


Figure 2-1: Location Map in HEC-RAS Geometry Editor

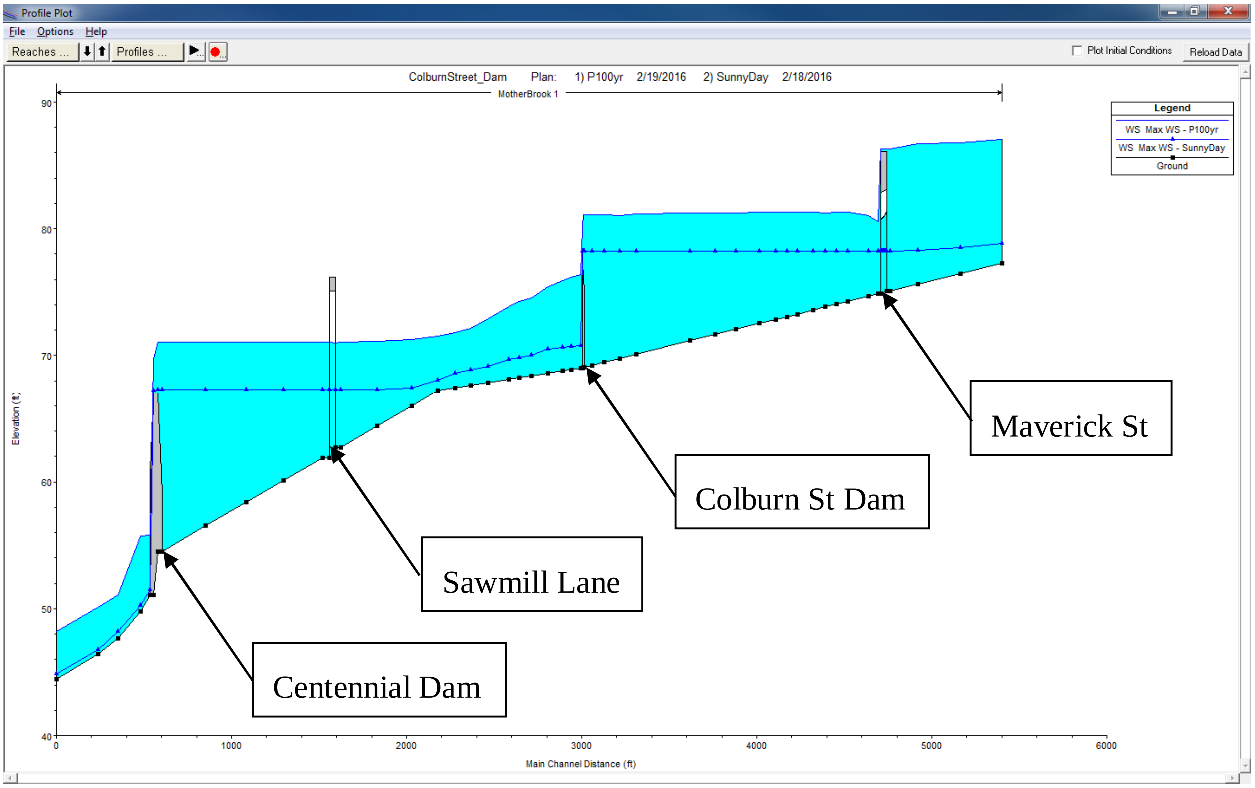


Figure 2-2: HEC-RAS Non-Breach Flood Profiles

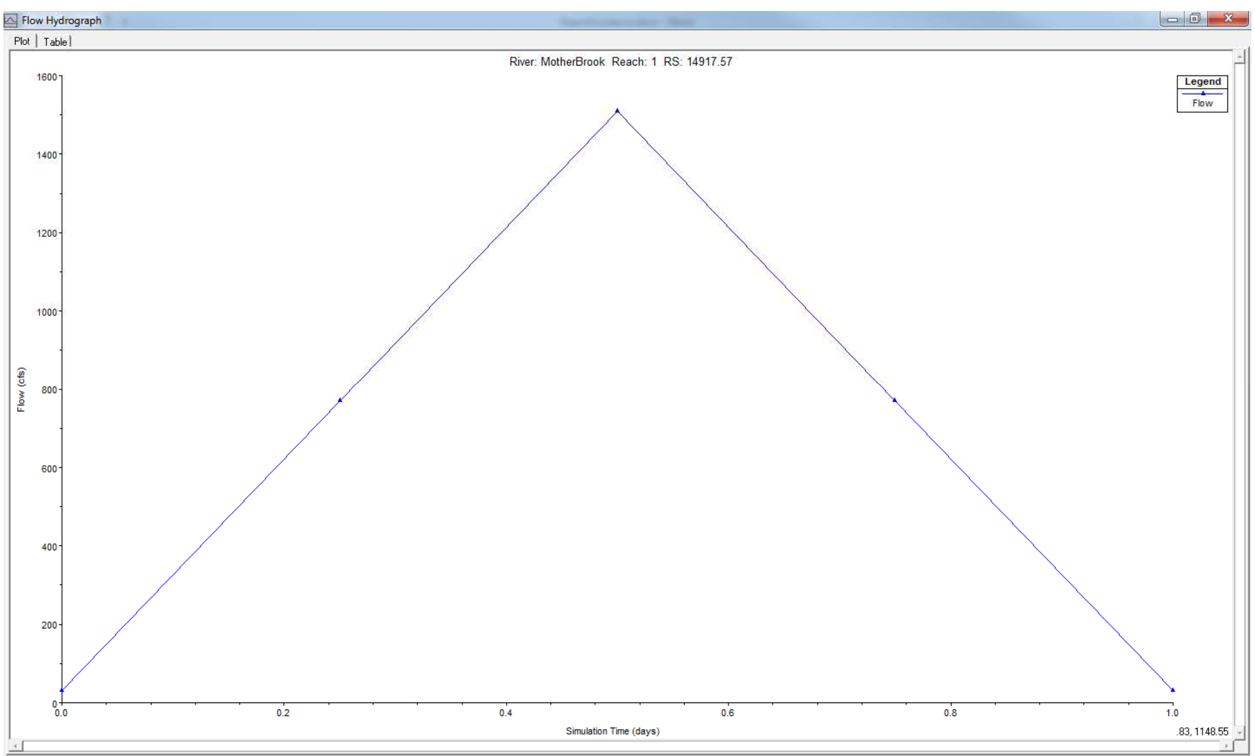


Figure 2-3: 100-yr Inflow

The HEC-RAS model was created in the MA State Plane Mainland projection. The topographic information was taken from data gathered as part of the 2013-2014 data collection effort and downloaded from www.mass.gov. The data summary for the LiDAR can be seen below in Figure 2-4. This information was supplemented by ground survey taken in the area immediately surrounding the dam. The Existing Conditions Plan created from the topographic on-the-ground survey is included in Attachment B - Conceptual Design Drawings to this report.

MassGIS LiDAR Terrain Data Project Area Summary

Project Name	Acquisition Dates	No. of Returns	Nominal Pulse Spacing	Vertical Accuracy	Raster DEM Resolution	Points Delivered	Contours Delivered	Intensity	Projection Information
2013-14 Sandy	Fall-Spring 2013-14	3	0.7 m *E	NVA = 18.13 cm	1 m	Ground-classified LAS 1.2	NA	All returns in LAS file and 1m raster	UTM Zones 18N & 19N NAD83 (2011)/NAVD88 Meters

Figure 2-4: LiDAR Summary

According to the Phase 1 Dam Inspection and Evaluation performed on May 23, 2006, the normal storage capacity of the Colburn Street Dam pool is 25-30 acre-feet. Although the channel shapes and inverts were unknown, the HEC-RAS model is characterized by 28.61 acre-feet for the Colburn Street Dam normal pool, which is an accurate representation of storage. Dam dimensions were taken from the Geotechnical Report and surveyed CAD drawing. The Colburn Street Dam was modeled as an inline structure in HEC-RAS and can be seen in Figure 2-5:

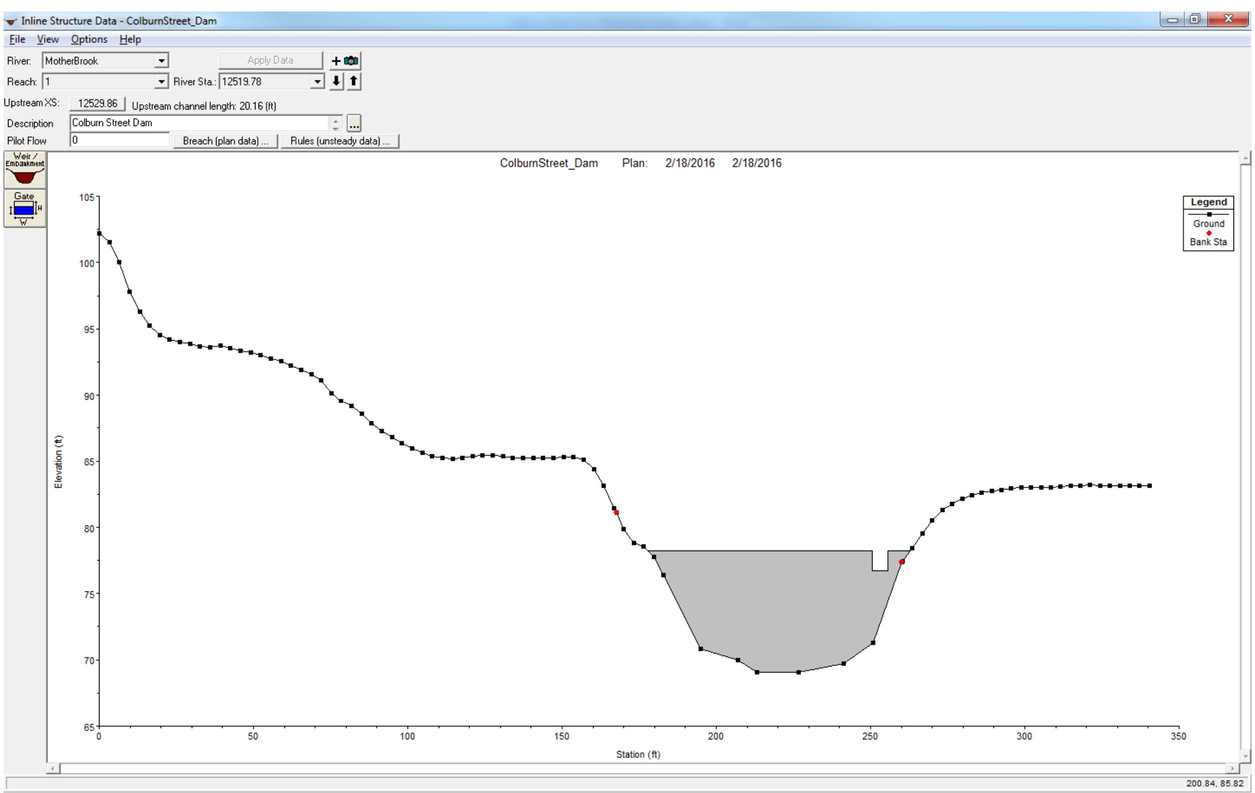


Figure 2-5: Colburn Street Dam as Inline Structure (Looking downstream)

The HEC-RAS model also contains two additional road crossings and an additional dam. The roadway structures were estimated based on the best available aerial photography and topography and these included Maverick Street (u/s) and Sawmill Lane (d/s). Bussey Street, which is immediately upstream of the Colburn Street Dam, was not included because it was deemed hydraulically insignificant due to the size of the structure, hydraulic opening and the nature of the flows being modeled. However, contraction/expansion losses consistent with a bridge were modeled at this location. The dimensions of Centennial Dam, which is approximately 0.46 miles downstream of Colburn Street Dam, was also estimated based on the best available aerial photography and topography.

The dam breach parameters were estimated using the US Army Corps of Engineers' Technical Document 39 (TD-39), which includes a compilation of breach parameter ranges from multiple federal agencies including the Federal Energy Regulatory Commission (FERC) and the National Weather Service (NWS). An overtopping breach was assumed with a weir coefficient of 2.6 and the final breach width was 49.25 feet, or approximately half of the dam's overall length. The breach side slopes were assumed to be vertical and the most conservative recommended failure time of 6 minutes was used. The HEC-RAS breach parameter inputs and final breach shape for the 100-yr breach can be seen below in Figure 2-6:

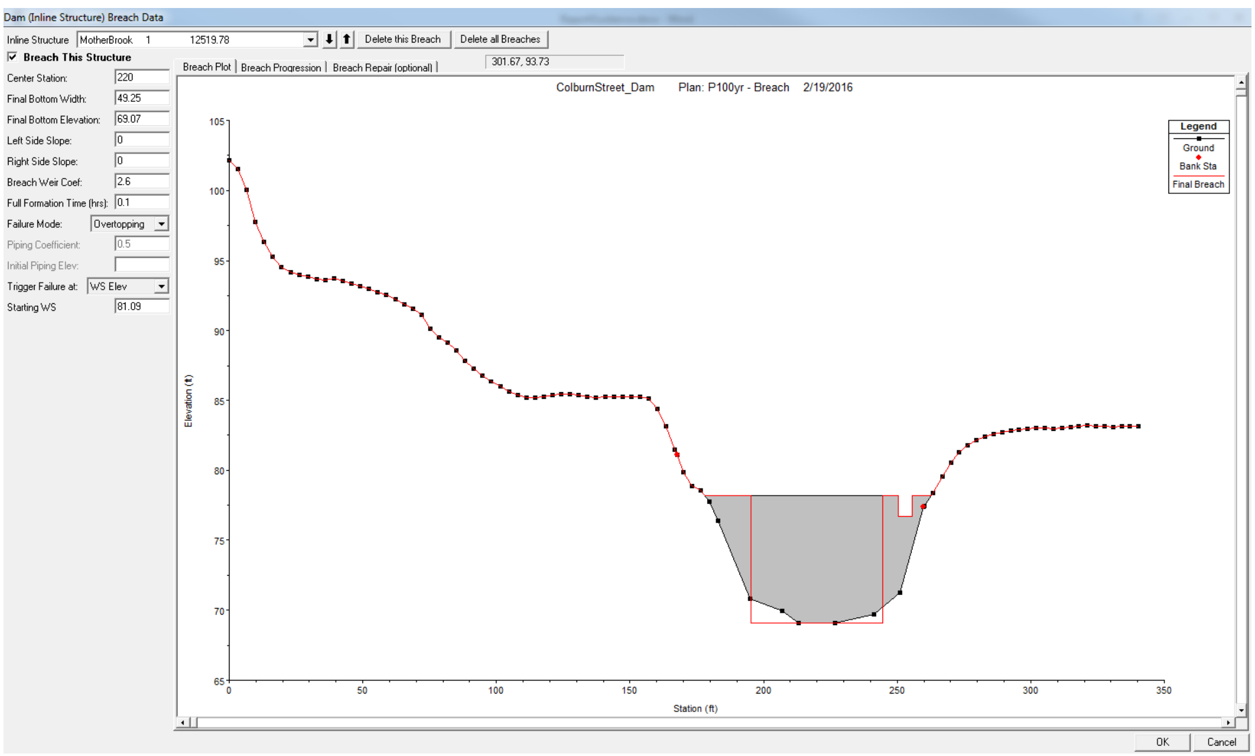


Figure 2-6: HEC-RAS 100-yr Breach Parameter Input

2.3 Dam Breach Analysis Results

Based on the results of the 100-year dam breach, Colburn Street Dam should continue to be listed as a Significant Hazard Potential dam per the Massachusetts Office of Dam Safety. This hazard classification refers to dams located where failure may cause loss of life and damage to home(s), industrial or commercial facilities, secondary highway(s) or railroad(s), or cause interruption of use or to service relatively important facilities. It is important to note that there is no immediate threat that the dam will breach based on the parameters modeled.

In this study area, no structures with a Finished Floor Elevation below the Sunny Day breach are located within the inundation area. Two structures, 186-188 Colburn St and 17 Emmett Ave, with a Finished Floor Elevation below the 100-yr breach are located within the inundation area and are indicated in Figure 2-7. A summary of water surface elevations, listed by cross section station, is shown in Table 2-2. Cross Section stations and locations are shown on Figures 2-1 and 2-7.

It should be noted that, barring any extreme storm events, the dam is not expected to fail due to breaching within the near future. However, the ongoing seepage through the dam face should be considered a safety issue. Ongoing monitoring of the seepage flows with respect to their location, character or volume is recommended. Any noticeable changes to any of these could indicate deteriorating conditions with the dam structure.

The full HEC-RAS Results for Sunny Day and 100-yr flood are included as Attachment C to this report.

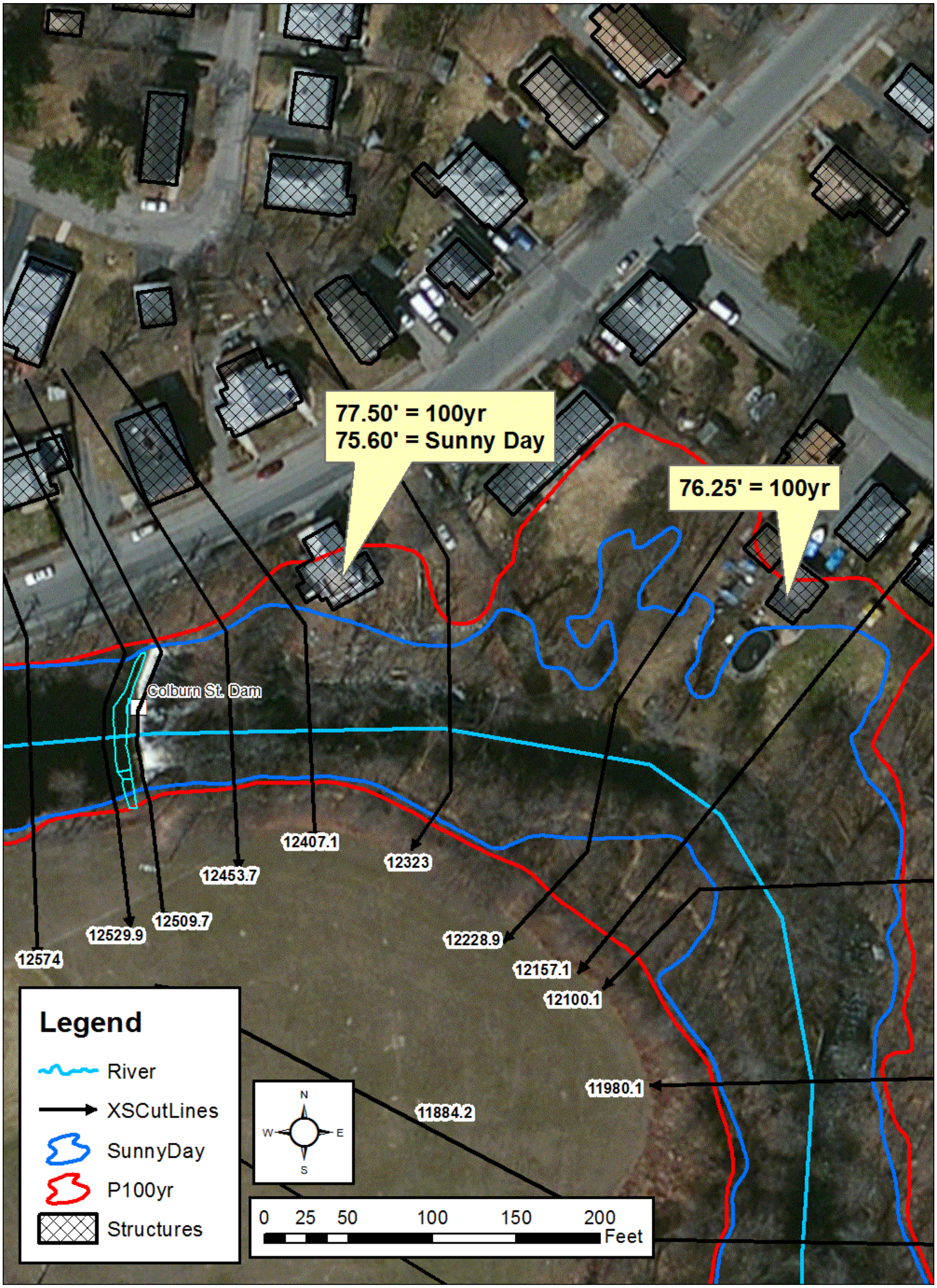


Figure 2-7: Dam Breach Inundation Areas

Table 2-1: Summary of Dam Breach Results (From U/S to D/S)

River Cross-Section (Station, feet from mouth)	Channel Invert (ft)	Sunny Day	Sunny Day Breach	100-yr	100-yr Breach
		Water Surface Elevation (ft)			
14917.57	77.31	78.88	78.88	87.05	87.05
14678.47	76.48	78.50	78.50	86.80	86.80
14437.47	75.65	78.28	78.28	86.69	86.69
14278.06	75.10	78.24	78.24	86.32	86.32
14239.99	**Maverick Street**				
14209.94	74.87	78.24	78.24	80.56	80.54
14152.98	74.65	78.24	78.24	81.06	81.06
14034.54	74.26	78.23	78.23	81.30	81.29
13968.54	74.03	78.23	78.23	81.32	81.31
13908.55	73.83	78.23	78.23	81.28	81.27
13835.49	73.58	78.23	78.23	81.28	81.28
13747.60	73.27	78.23	78.23	81.29	81.28
13687.09	73.06	78.23	78.23	81.29	81.29
13625.62	72.85	78.23	78.23	81.29	81.28
13531.66	72.53	78.23	78.23	81.28	81.28
13396.96	72.06	78.23	78.23	81.27	81.27
13275.77	71.64	78.23	78.23	81.28	81.27
13136.06	71.16	78.23	78.23	81.28	81.27
12829.26	70.10	78.23	78.23	81.16	81.16
12730.73	69.76	78.23	78.23	81.02	81.02
12645.91	69.47	78.23	78.23	81.11	81.11
12574.01	69.22	78.23	78.23	81.10	81.10
12529.86	69.07	78.23	78.23	81.10	81.09
12519.78	**Colburn Street Dam**				
12509.70	69.00	70.80	76.20	76.41	78.51
12453.66	68.88	70.73	76.01	76.21	78.13
12407.11	68.78	70.64	75.74	75.93	77.81

River Cross-Section (Station, feet from mouth)	Channel Invert (ft)	Sunny Day	Sunny Day Breach	100-yr	100-yr Breach
		Water Surface Elevation (ft)			
Structure Location					
12322.99	68.60	70.48	75.31	75.43	76.92
12228.89	68.40	70.03	74.03	74.52	76.36
Structure Location					
12157.11	68.24	69.82	73.75	74.24	76.14
12100.13	68.12	69.69	73.38	73.85	75.64
11980.08	67.86	69.16	72.40	72.90	74.37
11884.15	67.65	68.85	71.55	72.13	73.14
11793.84	67.46	68.60	71.03	71.82	72.78
11695.85	67.25	68.04	70.53	71.52	72.43
11544.00	66.02	67.41	69.13	71.25	72.09
11346.18	64.43	67.30	68.65	71.11	71.93
11136.80	62.73	67.30	68.65	71.05	71.82
11083.69	**Sawmill Lane**				
11032.80	61.89	67.30	68.65	71.06	71.82
10812.21	60.11	67.30	68.65	71.07	71.85
10599.80	58.40	67.30	68.65	71.07	71.85
10368.25	56.52	67.30	68.65	71.06	71.85
10117.64	54.50	67.30	68.65	71.06	71.85
10083.22	**Centennial Dam**				
10051.66	51.09	51.46	52.56	55.80	56.90
9997.84	49.80	50.27	52.21	55.74	57.04
9864.95	47.65	48.18	49.43	51.05	51.70
9752.95	46.45	46.77	48.04	50.10	51.07
9515.39	44.45	44.88	46.33	48.23	49.11

3.0 GEOTECHNICAL ASSESSMENT

3.1 General

The geotechnical assessment was performed during the fall of 2015 and includes the following:

- A subsurface exploration program:
    - Two test borings to observe soil and bedrock conditions.
    - Hand-held probes on the upstream side of the dam to evaluate sediment depths and upstream geometry of the dam.
- A stability analysis of the dam for the normal pool and design flood cases.

The full Geotechnical Report is attached to this report as Attachment D.

3.2 Subsurface Explorations

Two borings (B101 and B102) were completed on the banks of the brook and are shown approximately in Appendix A of the Geotechnical Report and boring logs can be found in Appendix B. The borings were advanced using a track-mounted CME-45 drill rig and drive and wash methods.

We also performed 25 hand auger probes on the upstream side of the dam to evaluate the depth of sediment and approximate geometry of the dam. Profiles developed from the auger probes are included in Appendix C of the Geotechnical Report.

3.3 Subsurface Conditions

The soil encountered in the borings consisted of 5.5 to 7.5 feet of silty sand with gravel, overlying highly fractured Granite bedrock.

The silty sand with gravel was generally described as well-graded brown sand with about 30 percent of the constituents being fine to coarse gravel, and 20 percent being non-plastic fines. N-values ranged from 4 to 17 blows per foot (bpf) which is indicative of a very loose to medium dense compactness.

Bedrock was encountered at El. 74.6 in B101 and El. 66.5 in B102. Bedrock was cored at the depths of 7 to 26.5 in B101 and 10.5 to 35.5 in B102. The bedrock was generally described as moderately hard to hard, weathered, and severely jointed granite. The Rock Quality Designation (RQD) ranged from 0 to 47 percent which is indicative of poor to very poor rock mass quality. Core recovery ranged from 40 percent to 100 percent.

3.4 Stability Analysis and Results

We conducted a stability analysis for the pool levels at normal pool and design flood (100-yr flood elevations). The stability analysis was performed using Slope/W Program.

Based on our stability evaluation for the dam using the assumptions above, we conclude the following:

- The factor of safety against sliding for the 100-year flood is greater than the factor of safety of 2.0 as required in the USACE guidance document EM 1110-2-2200, *Gravity Dam Design*.
- The spillway structure is not likely to overturn during the 100-year flood event.
- The continuing leakage through the face of the dam creates some risk of the dam blocks shifting and becoming unstable over time.

Some preliminary recommendations for mitigating seepage and scour are discussed below.

3.5 Preliminary Geotechnical Recommendations

The following options should be considered to mitigate the seepage through the face of the dam based on the geotechnical assessment and stability analysis performed:

- Perform grouting of the stone blocks of the dam to reduce seepage.
- Install a geomembrane on the back face of the dam to reduce seepage.
- Place large riprap to fill scour holes at toe of dam.

These preliminary recommendations are being used as the basis to identify and evaluate the repair alternatives described in the following section.

4.0 EVAULATION OF REPAIR ALTERNATIVES

Several alternatives were considered for repair of this spillway structure. These alternatives were developed based on the results of the analyses performed, review of earlier inspection reports and field visits to the dam to view the current conditions.

Alternatives considered include:

1. No Build

2. Repairs consisting of applying shotcrete to the upstream face of the dam, replacement of the existing wood stop logs with an aluminum stop log system, grouting and repointing the stones on the downstream face and placement of riprap downstream of the dam.

3. Repairs similar to those in Alternative 2 except that an epoxy waterproofing membrane would be installed instead of shotcrete and the limits and depth of the riprap would be less than in Alternative 2.

4. Complete replacement.

5. Complete removal of the dam

Alternatives 2, 3 and 4 will also address removal of woody vegetation and replacement with grass on the abutments and providing a fence to protect the dam from the adjacent Condon Park.

4.1 ALTERNATIVE 1 – No Build

This alternative was determined not to be viable based on the recommendations of the geotechnical report, descriptions of conditions in the older reports and field observation of the current condition of the dam. It was clear that some repair is required as there is visible seepage occurring at the face of the dam, leaks at the existing stop logs and scour of varying depths all along the downstream face of the dam.

This alternative has no estimated construction cost. This alternative will add no years to the useful life of the dam.

4.2 ALTERNATIVE 2 – Repairs – Shotcrete upstream face, Grout downstream face, Install new stop logs, Riprap placement downstream

This alternative will provide substantial repairs and improvements to the dam.

Application of shotcrete to the upstream face of the dam will serve multiple purposes. It will provide structural reinforcement to that face and will be able to fill any voids in the upstream face that may be uncovered when the accumulated sediment is removed. It will also provide a water proofing layer on that face which will substantially reduce seepage through the dam structure.

Grout packing and pointing the stones on the downstream face will eliminate the voids between the stones and reinforce their structural integrity.

Replacing the stop logs will provide an operable stop log system while eliminating the current leakage that occurs there.

Placement of riprap to the extents noted (approximately 30 feet downstream from the face) will protect the downstream area behind the dam from scour during high flow situations. In addition, the riprap would extend far enough to dissipate the energy from all but the highest flows.

This alternative has an estimated construction cost of $600,000. It is estimated that this alternative would add 40 years to the useful life of the dam assuming regular maintenance is performed.

4.3 ALTERNATIVE 3 – Repairs – Epoxy membrane on upstream face, Grout downstream face, Install new stop logs, Reduced riprap coverage downstream

This alternative is similar to the recommended alternative and provides similar benefits to alternative 2 with the differences described below.

Application of an epoxy membrane to the upstream face of the dam will provide a waterproof coating to the face of the dam similar to the shotcrete but would not provide any of the associated structural reinforcement. In addition, should voids or other openings be identified in the upstream face once repairs begin, they would need to be filled or patched with concrete prior to the application of the epoxy. This would offset the price savings of the epoxy membrane and require more time to complete the work.

Reduction in the depth and extent of the riprap would be less expensive due to a reduction in the amount of material (approximately 30% less than Alternative 2). This will provide less scour protection since the riprap would not extend as far downstream and would not effectively dissipate the energy of the water during higher flows.

This alternative has an estimated construction cost of $550,000 which assumes no additional concrete is required prior to application of the epoxy membrane. It is estimated that this alternative would add 40 years to the useful life of the dam assuming regular maintenance is performed.

4.4 ALTERNATIVE 4 – Complete Replacement

This alternative includes a full replacement of the dam and would provide the longest design life. However, based on information gathered during this and previous investigations this level of effort is not necessary.

This alternative has an estimated construction cost of $1,400,000. It is estimated that this alternative would add more than 50 years to the useful life of the dam assuming regular maintenance is performed.

4.5 ALTERNATIVE 5 – Complete Removal

This alternative was included to provide to a more complete list of options and includes removing most or all of the existing structure and allowing the brook to return to a state similar to what existed before the dam was constructed. This alternative is not a viable option. See discussion in Section 4.7.

This alternative has an estimated construction cost of $400,000. An estimate of useful life of the dam is not applicable to this alternative.

4.6 Comparison of Alternatives

Table 4-1 below ranks the options against each other. Each option is ranked 1 through 10 for each item, a lower rank indicates it is anticipated to be a better alternative when compared to the other alternatives for that item.

Table 4-1 – Rating Table for Repair Alternatives

Description	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5
Structural	10	3	5	1	n/a
Seepage	10	3	4	1	n/a
Life	7	3	5	1	n/a
Cost	1	4	3	10	4
Permitting	1	4	4	8	1
Construction Impacts	1	3	3	10	5
TOTAL:	**30**	**20**	**24**	**31**	**n/a****
OVERALL RANK:	**3rd**	**1st**	**2nd**	**4rd**	**5th**

*1 = Highest Rank (best value), 10 = lowest rank (least value)

** See Section 4.7

4.7 Discussion Regarding Removal of the Dam

Removal of an existing dam is a complex process requiring the consideration of a number of factors including but not limited to hydraulics, impact on downstream flooding and structures, environmental and possibly historic permitting, disposition of the accumulated sediment, restoration and stabilization of the stream or lake bed and aesthetics.

The project proponent would need to contact the Department of Conservation and Recreation (DCR) Office of Dam Safety at the start of the process to discuss the proposed project and to request a Jurisdictional Determination. In addition, a number of additional actions would need to occur.

Removal of the dam will require the need for a number of new analyses. Colburn Street Dam is located upstream of Centennial Dam which is classified as a high hazard dam. The hydraulics before and after dam removal will need to be modeled to determine the impact on Centennial Dam.

One of the most important items will be to determine the extent of the existing accumulated sediment and its disposition once the existing dam is removed. Some sediment would be removed as part of the dam demolition but it is unreasonable to assume that it would be possible for all of the sediment to be removed. It would be necessary to also model the impact of the sediment moving downstream over time and analyzing its impact downstream.

The existing pond created by the Colburn Street dam is adjacent to both a recreation area and an historic area. If the decision were to be made to remove the dam it would be necessary to hold one or more public hearings regarding the impact to both the recreation area and the historic area primarily in terms of aesthetics. One or more designs for the area that is currently occupied by the pond will need to be developed and presented to the public for their input.

Following all of these actions and the acquisition of all of the needed permits, the resulting design plans and analyses would be submitted to the Office of Dam Safety for their review and acceptance.

More information regarding the requirements and processes for the removal of a dam in the State of Massachusetts may be found at http://www.mass.gov/eea/agencies/dfg/der/aquatic-habitat-restoration/river-restoration/dam-removal.html.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Dam Break Analysis

Based on the results of the 100-year dam breach, Colburn Street Dam should continue to be listed as a Significant Hazard Potential dam per the Massachusetts Office of Dam Safety.

In this study area, no structures with a Finished Floor Elevation below the Sunny Day breach are located within the inundation area. Two structures, 186-188 Colburn St and 17 Emmett Ave, with a Finished Floor Elevation below the 100-yr breach are located within the inundation area.

Geotechnical Assessment

The following items were included in our alternatives analysis to mitigate the seepage through the face of the dam based on the geotechnical assessment and stability analysis performed:

- Perform grouting of the stone blocks of the dam to reduce seepage.
- Install a geomembrane on the back face of the dam to reduce seepage.
- Place large riprap to fill scour holes at toe of dam.

5.2 Recommended Alternative

The recommended alternative is Alternative 2 – Repairs – Shotcrete upstream face, Grout and point downstream face, Install new stop logs, and Riprap placement downstream. This alternative is recommended because it balances the need for repairs with total cost and design life. While a new dam would last longer it would cost at least two times more than this option and the dam has not deteriorated to a state that would require full replacement. Therefore, repairing the existing dam to extend the useful life of the dam makes sense and reduces the cost the Town needs to invest into the dam at this time.

Conceptual design plans were developed for this alternative and included as Attachment B to this report.

A cost estimate breakdown for the preferred alternative is included as Attachment E to this report. This estimate includes a contingency amount of 25% typically used at the conceptual design level. It also includes a construction allowance to cover the cost of installing rock bolts and constructing a concrete grade beam along the toe of the dam. This additional work may be deemed necessary to improve stability of the dam once the accumulated sediment is removed and repairs have begun.

Attachment A: Colburn Street Dam Follow-up Inspection/Evaluation Report, July, 2013, prepared by GZA GeoEnvironmental, Inc.

COLBURN STREET DAM

FOLLOW-UP
INSPECTION / EVALUATION REPORT

Dam Name:	**Colburn Street Dam**
State Dam ID#:	**6-11-73-2**
NID ID#:	**MA02571**
Owner:	**Town of Dedham**
Owner Type:	**Commonwealth of Massachusetts**
Town:	**Dedham, MA**
Consultant:	**GZA GeoEnvironmental, Inc.**
Date of Inspection:	**July 15, 2013**

GZA
GeoEnvironmental, Inc.

Engineers and Scientists

249 Vanderbilt Avenue
Norwood
Massachusetts
02062
781-278-3700
FAX 781-278-5701
http://www.gza.com

July 29, 2013
File No. 01.18802.38

Mr. William Salomaa
Department of Conservation and Recreation
Office of Dam Safety
251 Causeway Street, Suite 600
Boston, MA 02114-2104

Re: Follow-up Inspection/Evaluation Report
Colburn Street Dam, Dedham, MA
NID# MA02571

Dear Mr. Salomaa

GZA GeoEnvironmental, Inc. (GZA) is pleased to present the Department of Conservation and Recreation (DCR) Office of Dam Safety the attached Follow-Up Inspection/Evaluation Report for the Colburn Street Dam in Dedham, Massachusetts. This report has been developed under GZA's current task order agreement with DCR from RFR No. DCR395 and the Notice to Proceed from DCR (Assignment No. 2 FY14) dated July 11, 2013. The results and recommendations contained herein are subject to the Limitation attached as **Appendix A**. This follow-up inspection report is intended to corroborate the observations made during previous inspections and to document changes since the last inspections.

The follow-up inspection was completed by GZA on July 15, 2013. Flow conditions at the Colburn Street Dam allowed better observation than during the May 2006 Phase I Inspection, when high flows hindered the ability to see the overflow portions of the dam. On the basis of more extensive observations, the condition of the dam is now considered to be **FAIR**, in GZA's opinion. This is a downgrade in the previously reported condition of the dam. The noted deficiencies at Colburn Street Dam include scour downstream of the sluiceway area as well as scour approximately three to four feet downstream of the face of the dam, along the entire length of the dam. Seepage was noticed through the unmortared masonry face of the dam approximately six feet from the top of the dam near the sluiceway area. Large voids were observed between the stones comprising the downstream face of the dam. Leakage through the installed stop logs at the sluiceway was also observed. The concrete cap is also scoured along the upstream face the length of the dam.

In addition to permitting better observation of conditions, the low-flow conditions also permitted a better assessment of the size of the dam. Based on measurements taken during the follow-up inspection, it is GZA's opinion that the Size of the dam meets the definition of a "Small" structure as per 302 CMR 10.06. In addition, observations made by GZA during flooding in 2010, combined with current downstream reconnaissance, suggest that the appropriate Hazard classification for the dam, as per 302 CMR 10.06, is "Significant," in GZA's opinion. If accepted by the Commissioner, both of these recommendations would require modifications to the current data contained in the dam safety inventory.

It is our understanding that the DCR assigned GZA to perform this inspection as a courtesy to the dam owner, the Town of Dedham, to take advantage of DCR water control efforts which were on-going in Mother Brook during the inspection. A representative of the Town of Dedham Engineering Department was present during the inspection. As per our instructions from you, GZA has provided a copy of this report directly to the Town of Dedham.

We are happy to have been able to assist you with this inspection and appreciate the opportunity to continue to provide the DCR with dam engineering consulting services. Please contact the undersigned if you have any questions or comments regarding the content of this Inspection/Evaluation Report.

Sincerely,

GZA GEOENVIRONMENTAL, INC.

Derek J. Schipper, P.E
Senior Project Manager

Peter H. Baril, P.E.
Consultant/Reviewer

Chad W. Cox. P.E
Principal-In-Charge

Cc: William A. Gode-von Aesch– DCR Flood Control Director
Jason Mammone – Town of Dedham Engineering Department

J:\17,000-18,999\18802\18802-38.DJS\Report\MA02571 Colburn Streetl Dam July 2013 Follow-up.docx

PREFACE

The assessment of the general condition of the dam is based upon available data and visual inspections. This follow-up inspection report is intended to corroborate the observations made during previous inspections and document changes since the last inspection. Detailed investigations and analyses involving topographic mapping, subsurface investigations, testing and detailed computational evaluations are beyond the scope of this report.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection, along with data available to the inspection team. In cases where an impoundment is lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions, which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is critical to note that the condition of the dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Prepared by:
GZA GeoEnvironmental, Inc.

COMMONWEALTH OF MASSACHUSETTS
CHAD W. COX
CIVIL
No. 45856
REGISTERED
PROFESSIONAL ENGINEER

7/23/13

Chad W. Cox, P.E.

Massachusetts License No.: 45856
Principal
GZA GeoEnvironmental, Inc.

COMMONWEALTH OF MASSACHUSETTS
DEREK J. SCHIPPER
CIVIL
No. 47577
REGISTERED
PROFESSIONAL ENGINEER

7/26/2013

Derek J. Schipper, P.E.

Massachusetts License No.: 47577
Project Manager
GZA GeoEnvironmental, Inc.

Commonwealth of Massachusetts
Department of Conservation and Recreation
Office of Dam Safety Poor and Unsafe Condition Dam Follow-up Inspection Form

Dam Name: Colburn Street Dam
Dam Owner: Town of Dedham, Massachusetts
Nat. ID Number: MA 02571
Hazard Potential: N/A (Current); Significant (Recommended)
Size Classification: Non-Jurisdictional (Current); Small (Recommended)
Location of Dam (town): Dedham, MA
Coordinate location (lat, long): 42.2490°N, -71.1598°W
Date of Inspection: July 15, 2013
Weather: Sunny, 85 degrees Fahrenheit
State of Impoundment: ~2 feet below top of dam (about Elev. 74 feet – NGVD-1929 Datum)

Consultant Inspector(s): GZA GeoEnvironmental, Inc. – Chad W. Cox, P.E.
Derek J. Schipper, P.E.

Others in Attendance at Field Inspection: Jason Mammone, Town of Dedham
William A. Gode-von Aesch, DCR
Ed Hughes, DCR

Attachments:

Figure 1:	Locus Map
Appendix A:	Limitations
Appendix B:	Updated Photographs
Appendix C:	Updated Site Sketch

I. Previous Inspection date/Overall Condition:

- Date of most recent formal Phase I Inspection Report: May 23, 2006 (By Weston and Sampson)
- Date of most recent formal Follow-Up Inspection Form: N/A
- List the overall condition reported in most recent Phase I Inspection Report: SATISFACTORY

II. Previous Inspection Deficiencies:

- List identified deficiencies in the most recent Phase I Inspection Report:

1. Woody vegetation on the abutments;
2. Heavy brush on the left embankment;

Note that previous Phase I stated that observations of the overflow portion of the dam were obscured by flow.

III. Overall Condition of Dam at the Time of the Current Follow-up Inspection:

- **State the current condition:** FAIR
- **Have conditions changed since the previous inspection?** Dam was inspected during low water levels in July 2013.

IV. Comparison of Current Conditions to Condition Listed in Previous Phase I Inspection Report:

- **Have any of the deficiencies listed in the previous Phase I Inspection Report worsened? If yes, list the changes.** No.
- **Are there any additional deficiencies that have been identified in the current inspection?** Yes. (The top of dam, downstream face, and stoplogs were obscured by flow during the previous inspection so it is likely that these deficiencies were present during the previous inspection but could not be observed.)
- **If yes, list the deficiencies and describe.**

    - Seepage was observed through the unmortared masonry downstream face of the dam, approximately six feet from the top of the dam.
    - Scour of up to approximately up to 5 feet was observed via probing immediately downstream of the sluiceway, as well as two to four feet downstream of the face of the dam, for the length of the dam.
    - The timber stop logs appeared to be quite old and are likely inoperable. There is no access to the stop logs under normal flow conditions.
    - Sediment was found to have accumulated to within approximately one foot of the top of the stoplogs.
    - Leakage through the installed timber stop logs was also observed.
    - Voids were found in the downstream face of the dam which suggested that large stones may have been displaced from the structure. There was not a general connection between the location of the voids and the location of seepage.
    - Any previously present mortar and most of the smaller chink stones are no longer in place along the downstream face of the structure.
    - The concrete cap on top of the overflow section of the dam was seen to exhibit shallow scour of concrete paste resulting in exposed aggregate over fundamentally the full area of the cap.

V. Dam Safety Orders:

- **List dam safety orders that have been issued to the dam owner pertaining to this dam.** None issued.

VI. Maintenance:

1. **Indicate if there exists an operation and maintenance plan for the dam.** No operation and maintenance manual exists for the dam.
2. **Indicate if it appears the dam is being maintained.** No maintenance is performed at the dam on a regularly scheduled basis, to the best of GZA's knowledge.

VII. Recommendations:

GZA recommends that the SIZE classification of the dam be amended based on measurements taken during the 2013 follow-up inspection. Height of the dam was found to be a minimum of 9 feet when measured from the crest of the overflow section to the stream invert downstream of the dam. If the height is measured from the crest of the overflow section to the deepest location immediately downstream of the stop log sluiceway, then the height is approximately 13 feet. In either case, the height of the dam is greater than 6 feet and less than 15 feet, therefore falling within the SMALL category as defined by 302 CMR 10.06 (2).

In GZA's opinion, the HAZARD classification of the dam should be amended. Based on observations of flood impacts on the residential property immediately downstream of the dam on the left bank during flooding in 2010 and observations made during the inspection of 2013, it appears that the failure of the dam has the potential to, at minimum, cause damage to that home. This meets the definition of a SIGNIFICANT Hazard structure as per 302 CMR 10.06 (3).

The 2006 Phase I Inspection Report by Weston and Sampson made the following recommendations:

1. Prepare a site topographic and bathymetric survey;
2. Perform a hydrologic / hydraulic analysis for the dam;
3. Monitor condition of the dam during low flow [Note: accomplished during this follow-up inspection];
4. Observe the condition of the dam for changes, made at least quarterly, as well as during and following rainfall events that exceed the 25-year, 24-hour storm (approximately 5 inches of rain in 24 hours);
5. Woody vegetation on the abutments should be cut to ground surface, then a healthy stand of grass should be developed on those areas and maintained in that condition;
6. The right abutment area is directly accessible from Condon Park, which is utilized by families with small children. Consideration should be given to installing and maintain fencing or other means to prevent access to the dam for purposes of public safety.

In addition, GZA recommends that consideration be given to addressing the observed leakage through the dam, missing stones on the downstream face, scour at the toe, and the condition of the stop logs.

VIII. Other Comments or Observations: According to reports by a local resident, the impoundment upstream of the dam was last dredged over 40 years ago. Bedrock was observed at both abutments.

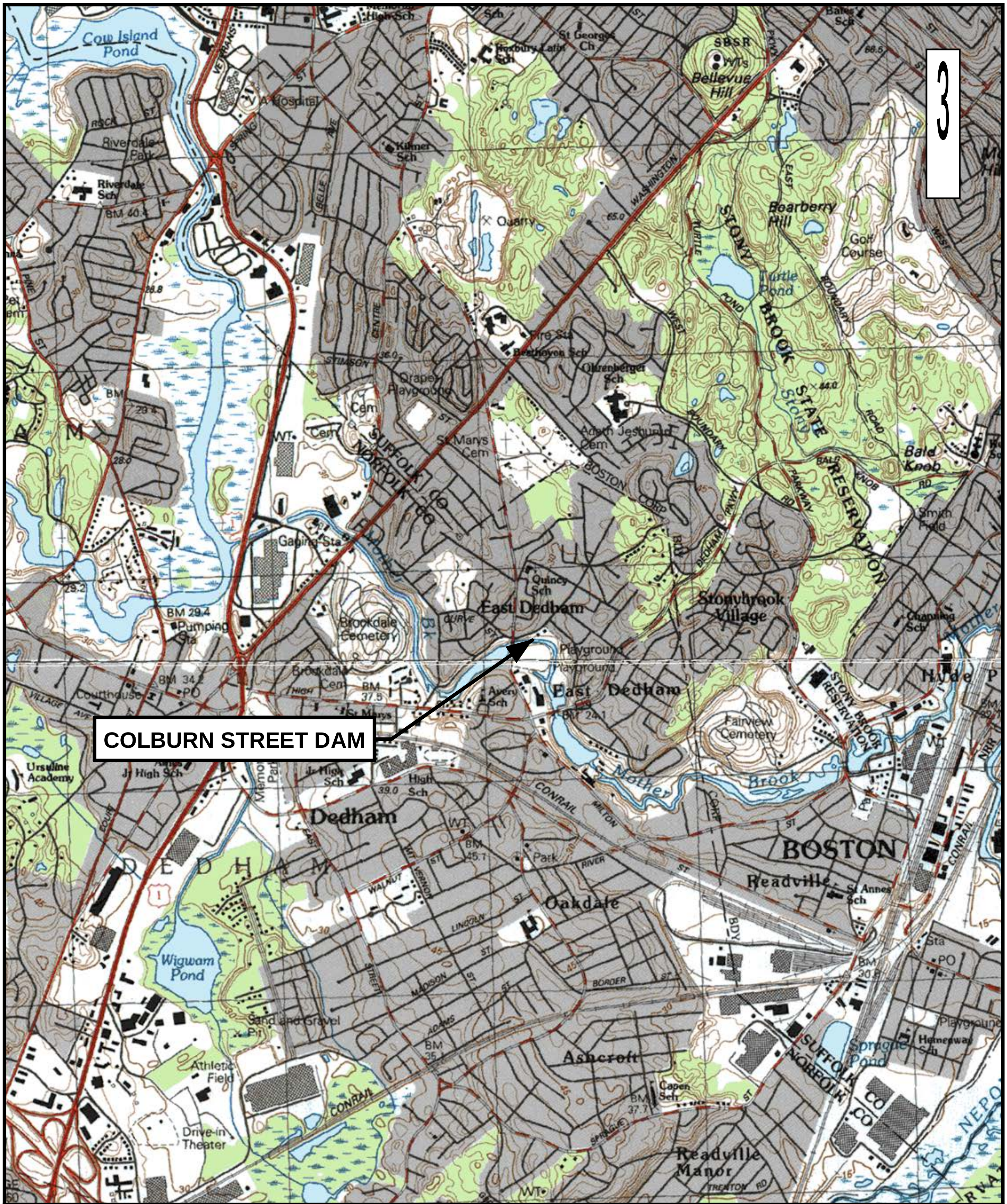
IX. Updated Site Sketch with Photo Locations: Attached

X. Updated Photos: Attached

XI. Copy of Locus Map from Phase I Report: Attached

XII. Other applicable attachment: GZA Limitations

FIGURES



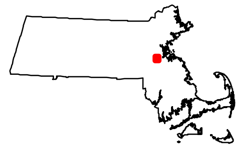


FIGURE 1
DEDHAM, MA
COLBURN STREET DAM - MA02571
USGS QUAD: NEWTON

LOCUS MAP

SCALE 1:25000
SOURCE: USGS

Weston&Sampson ENGINEERS, INC.

APPENDIX A

LIMITATIONS

DAM ENGINEERING REPORT LIMITATIONS

Use of Report

1. GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of the Commonwealth of Massachusetts Department of Conservation and Recreation (Client) for the Colburn Street Dam in Dedham and the stated purpose(s) and location(s) identified in the Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. Our findings and conclusions are based on the work conducted as part of the Scope of Services set forth in the Report and/or proposal, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. Conditions other than described in this report may be found at the subject location(s).

3. Our services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

Subsurface Conditions

4. If presented, the generalized soil profile(s) and description, along with the conclusions and recommendations provided in our Report, are based in part on widely-spaced subsurface explorations by GZA and/or others, with a limited number of soil and/or rock samples and groundwater /piezometers data and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then appear evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

5. Water level readings have been made in test holes (as described in the Report), monitoring wells and piezometers, at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the groundwater and piezometer levels, however, occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, reservoir and tailwater levels, the presence of subsurface utilities, and/or natural or artificially induced perturbations.

General

6. The observations described in this report were made under the conditions stated therein. The conclusions presented were based solely upon the services described therein, and not on scientific tasks or procedures beyond the scope of described services or the time and budgetary constraints imposed by the Client.

7. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein available to GZA at the time of the evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

8. Any GZA hydrologic analysis presented herein is for the rainfall volumes and distributions stated herein. For storm conditions other than those analyzed, the response of the site's spillway, impoundment, and drainage network has not been evaluated.

9. Observations were made of the site and of structures on the site as indicated within the report. Where access to portions of the structure or site, or to structures on the site was unavailable or limited, GZA renders no opinion as to the condition of that portion of the site or structure. In particular, it is noted that water levels in the impoundment and elsewhere and/or flow over the spillway may have limited GZA's ability to make observations of underwater portions of the structure. Excessive vegetation, when present, also inhibits observations.

10. In reviewing this Report, it should be realized that the reported condition of the dam is based on observations of field conditions during the course of this study along with data made available to GZA. It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued inspection and care can there be any chance that unsafe conditions be detected.

Compliance with Codes and Regulations

11. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

12. This scope of work does not include an assesment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

Cost Estimates

13. Unless otherwise stated, our cost estimates are for comparative, or general planning purposes. These estimates may involve approximate quantity evaluations and may not be sufficiently

accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over the labor and material costs required to plan and execute the anticipated work, our estimates were made using our experience and readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

14. It is recommended that GZA be retained to provide services during any future: site observations, explorations, evaluations, design, implementation activities, construction and/or implementation of remedial measures recommended in this Report. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.

APPENDIX B

PHOTOGRAPHS

Photo 1: View of dam from downstream.

Photo 2: Downstream discharge channel from top of dam.

Photo 3: Leakage through the stop logs at the sluiceway.

Photo 4: Woody vegetation at right abutment. Note seepage through face of dam.

Photo 5: View of crest and sluiceway from right abutment.

Photo 6: View of left abutment. Note good contact between concrete and bedrock.

Photo 7: View of right abutment. Note good contact between concrete and bedrock.

Photo 8: Seepage through face of dam and leakage through stop logs. Note scour measured downstream by approximately six foot long stick.

Photo 9: Large voids between stones on face of dam (no indication of soil movement).

Photo 10: Upstream view of dam.

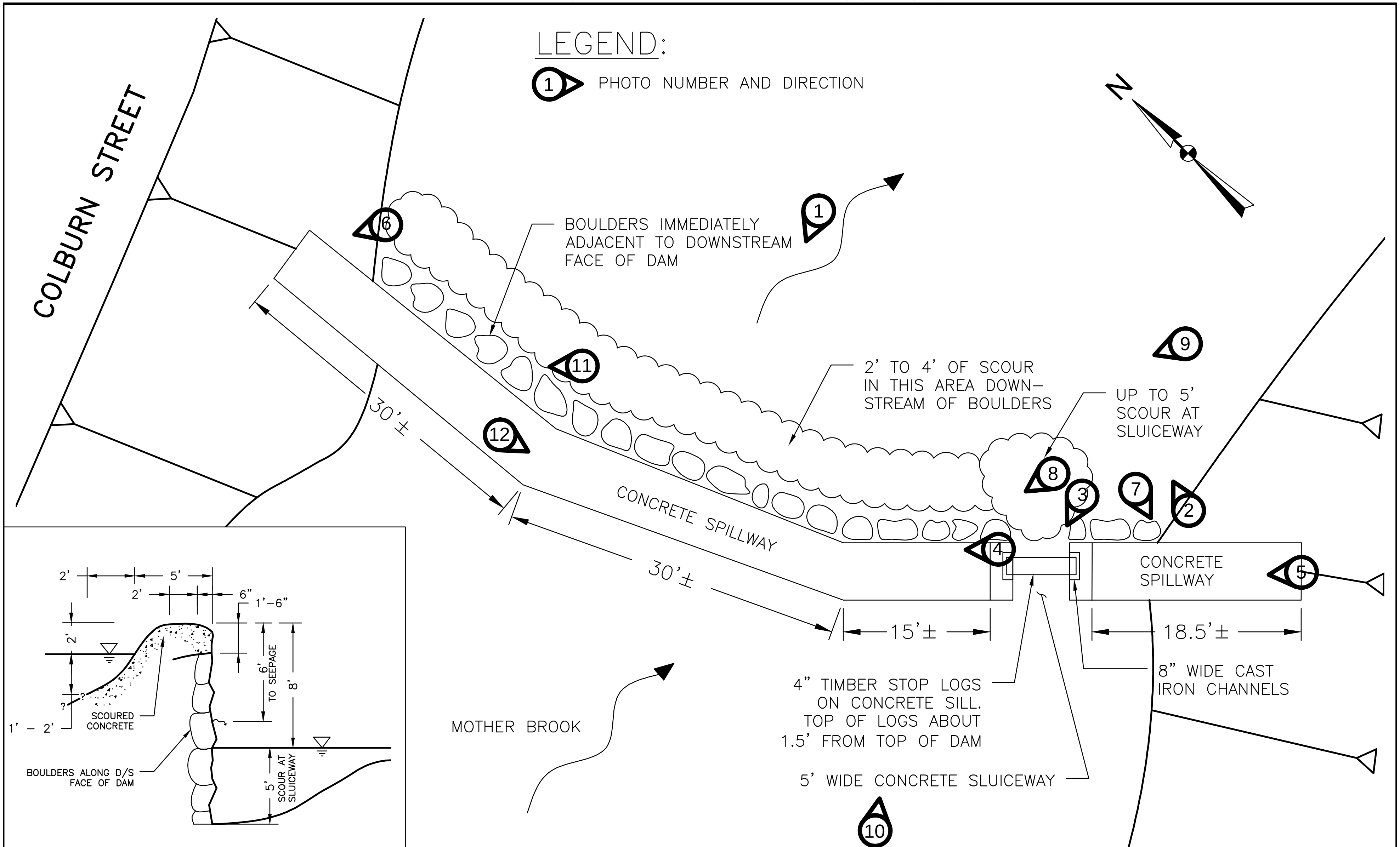
Photo 11: Large voids between stones making up downstream face of dam.

Photo 12: Scoured concrete along top of dam.

APPENDIX C

SITE SKETCH

©2013 – GZA GeoEnvironmental, Inc. GZA-J:\17,000-18,999\18802\18802-38.DJS\Figures\CADD\DAM INSPECTION SKETCH 1.dwg [Layout1] July 22, 2013 – 12:13pm connor.roman



NO.	ISSUE/DESCRIPTION	BY	DATE

COLBURN STREET DAM
DEDHAM, MA

COLBURN STREET DAM INSPECTION
DAM SKETCH

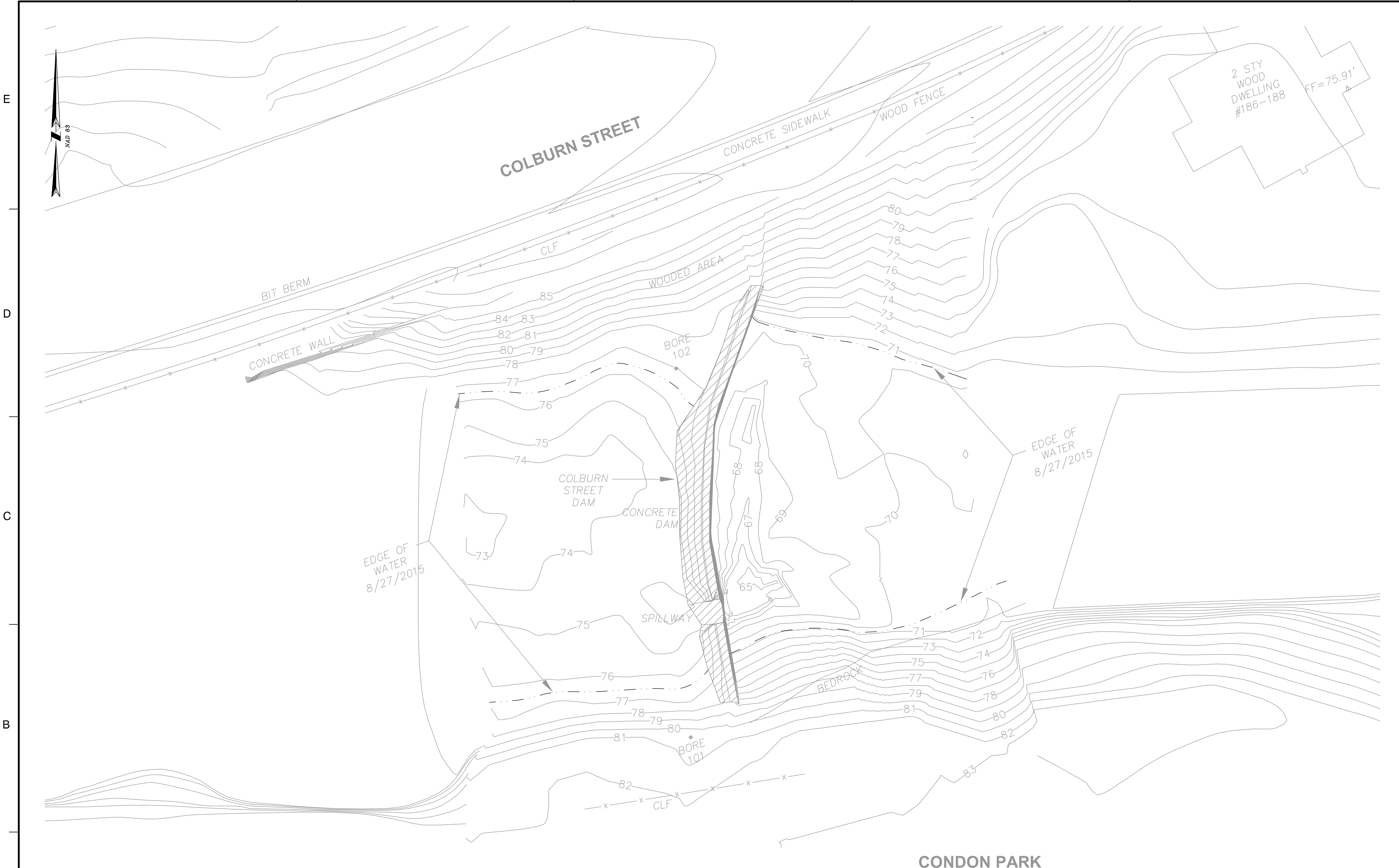
PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
MASSACHUSETTS DEPARTMENT OF CONSERVATION AND RECREATION

PROJ MGR: DJS	REVIEWED BY: DJS	CHECKED BY: DJS
DESIGNED BY: CFR	DRAWN BY: CFR	SCALE: N.T.S
DATE: 7/18/2013	PROJECT NO. 18802.38	REVISION NO.

FIG OR DWG 1
SHEET NO. 1 OF 1

Attachment B: Conceptual Design Plans



NOTE:

1. THIS PLAN IS BASED ON AND ON-THE-GROUND SURVEY PERFORMED BY THE BSC GROUP INC. BETWEEN SEPTEMBER AND NOVEMBER, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD '83 MASSACHUSETTS STATE PLANE COORDINATE SYSTEM AND VERTICAL DATUM IS BASED ON NAVD '88 AS ESTABLISHED BY GPS OBSERVATIONS.

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION

Dewberry®

280 SUMMER ST.,
10TH FLOOR
BOSTON, MA 02210
(617) 695-3400
(617) 695-3310

TOWN OF DEDHAM
MASSACHUSETTS

DEDHAM DEPARTMENT
OF PUBLIC WORKS

SEAL

KEY PLAN

SCALE

0 10 20
SCALE: 1" = 10'

No.	DATE	BY	Description

REVISIONS

DESIGNED BY PAR
DRAWN BY PAR
CHECKED BY PAR
DATE FEBRUARY 2016

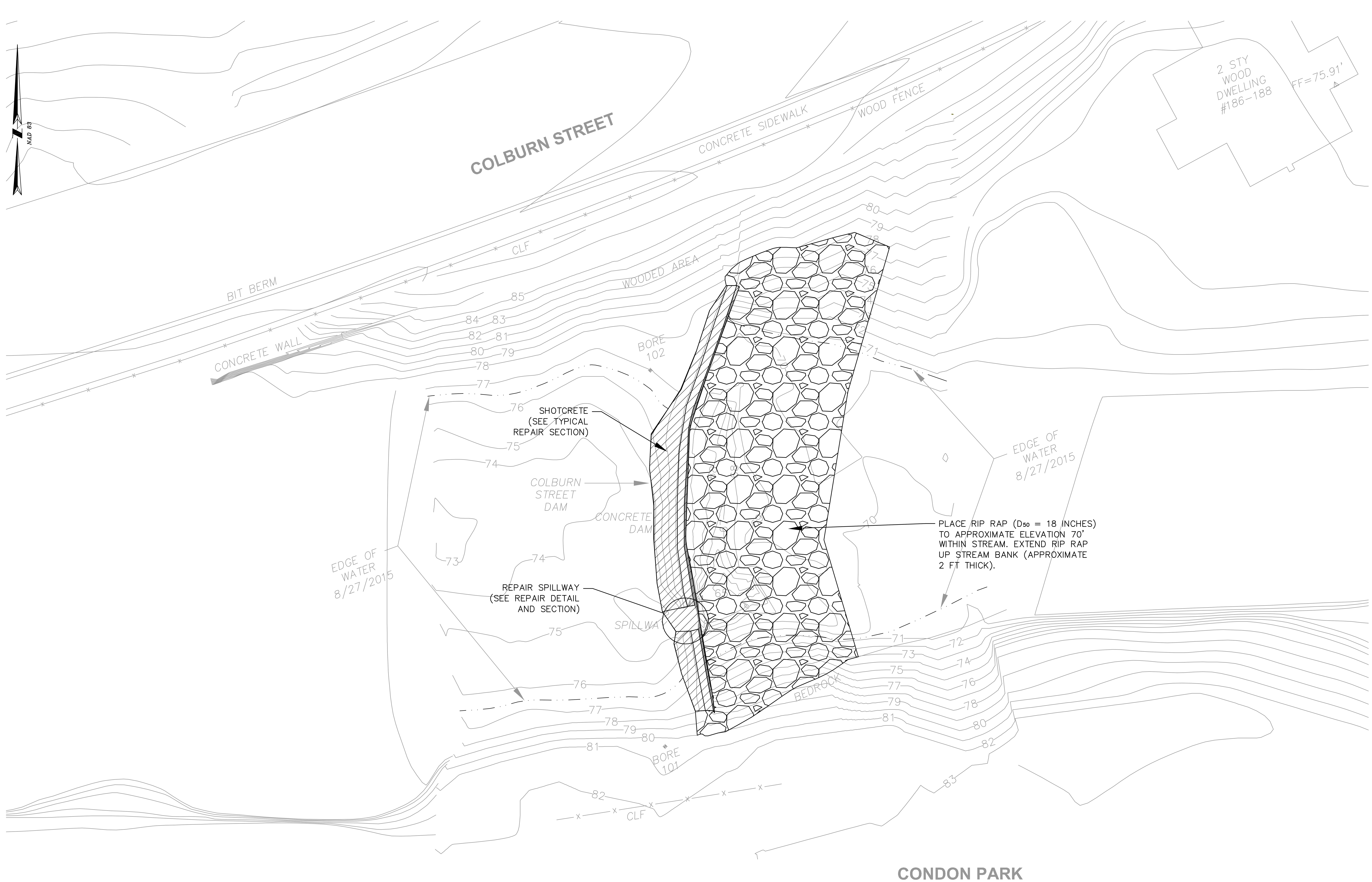
TITLE

EXISTING
CONDITIONS
PLAN

PROJECT NO. 50075664

C-101

SHEET NO. 1 OF 3



NOTES:
1. PRIOR TO FINAL DESIGN, WATER LEVEL SHALL BE LOWERED TO FURTHER EVALUATE THE SEDIMENT DEPTHS AND LOCATIONS ON THE UPSTREAM FACE OF THE CONCRETE DAM.
2. PRIOR TO FINAL DESIGN, THE DOWNSTREAM FACE OF THE DAM SHALL BE PROBED AND FURTHER INVESTIGATED TO DETERMINE THE EXISTING VOIDS WITHIN THE DAM AND DEPTH OF ROCK IN THE DAM.

CONCEPTUAL DESIGN
NOT FOR CONSTRUCTION

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BOSTON, MA 02210
(617) 695-3400
(617) 695-3310

TOWN OF DEDHAM
MASSACHUSETTS

DEDHAM DEPARTMENT
OF PUBLIC WORKS

SEAL

KEY PLAN

SCALE

0 10 20
SCALE: 1" = 10'

No.	DATE	BY	Description

REVISIONS

DESIGNED BY PAR
DRAWN BY PAR
CHECKED BY PAR
DATE FEBRUARY 2016

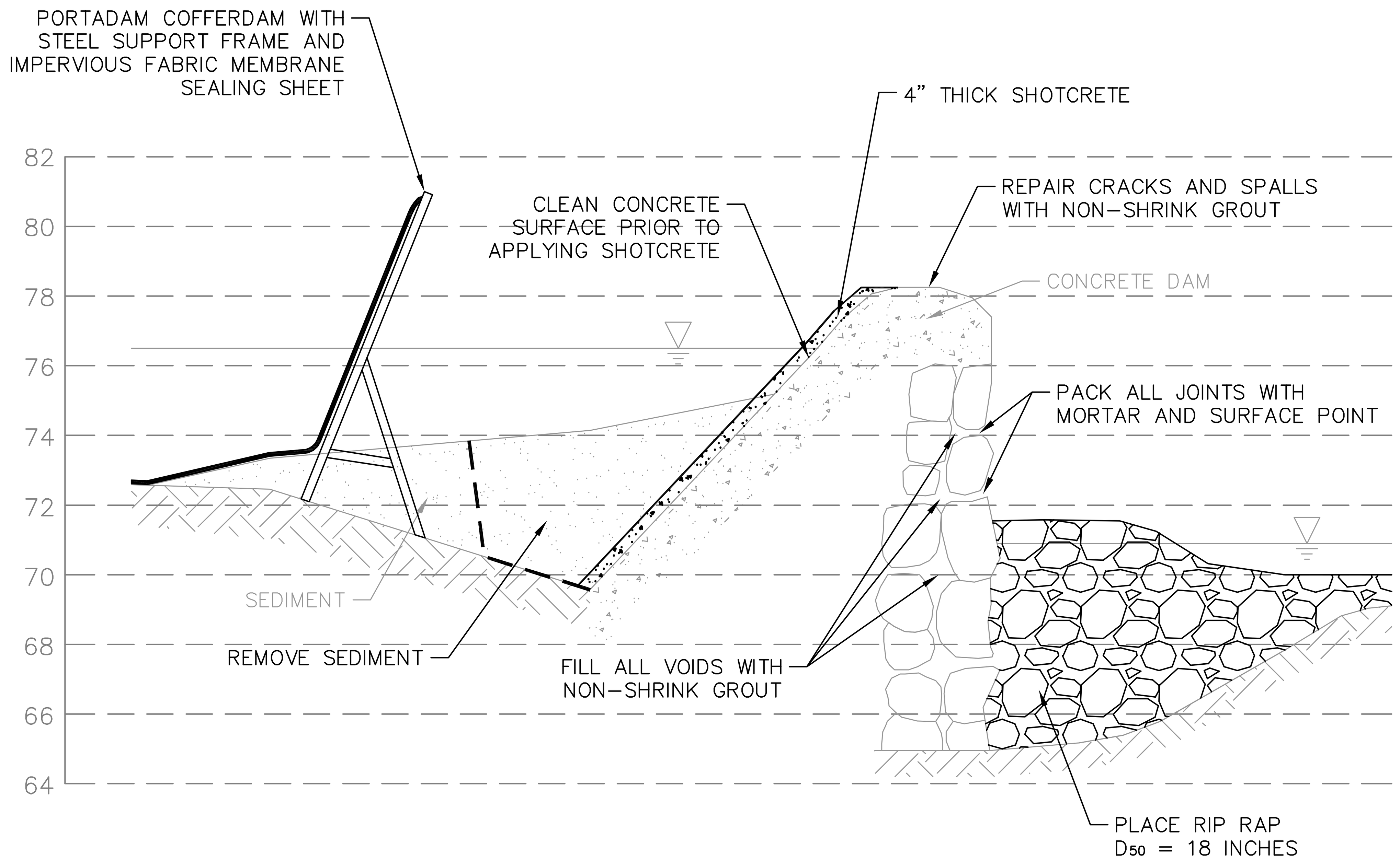
TITLE

PROPOSED
CONDITIONS
PLAN

PROJECT NO. 50075664

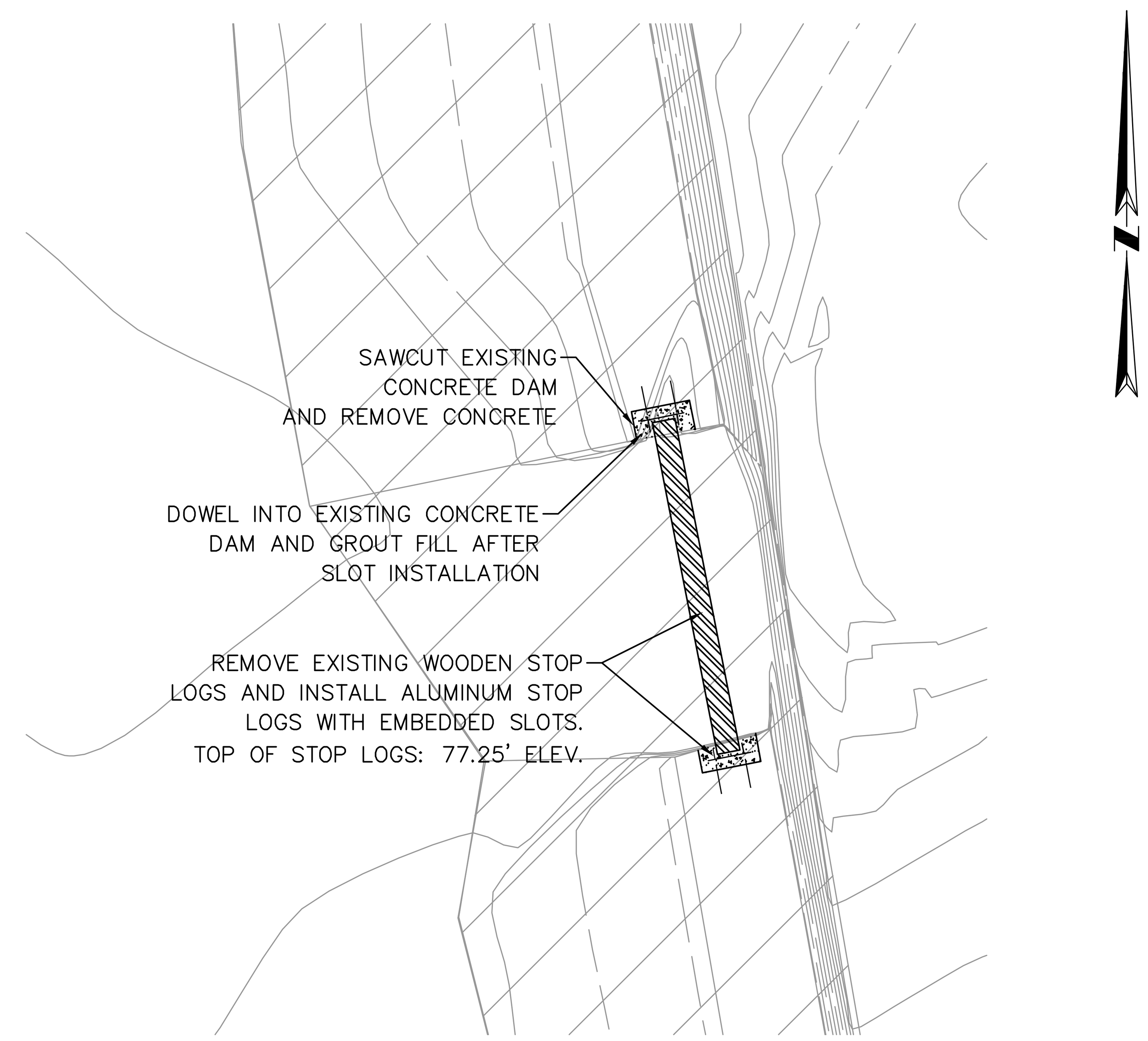
C-102

SHEET NO. 2 OF 3



CONCRETE DAM TYPICAL SECTION

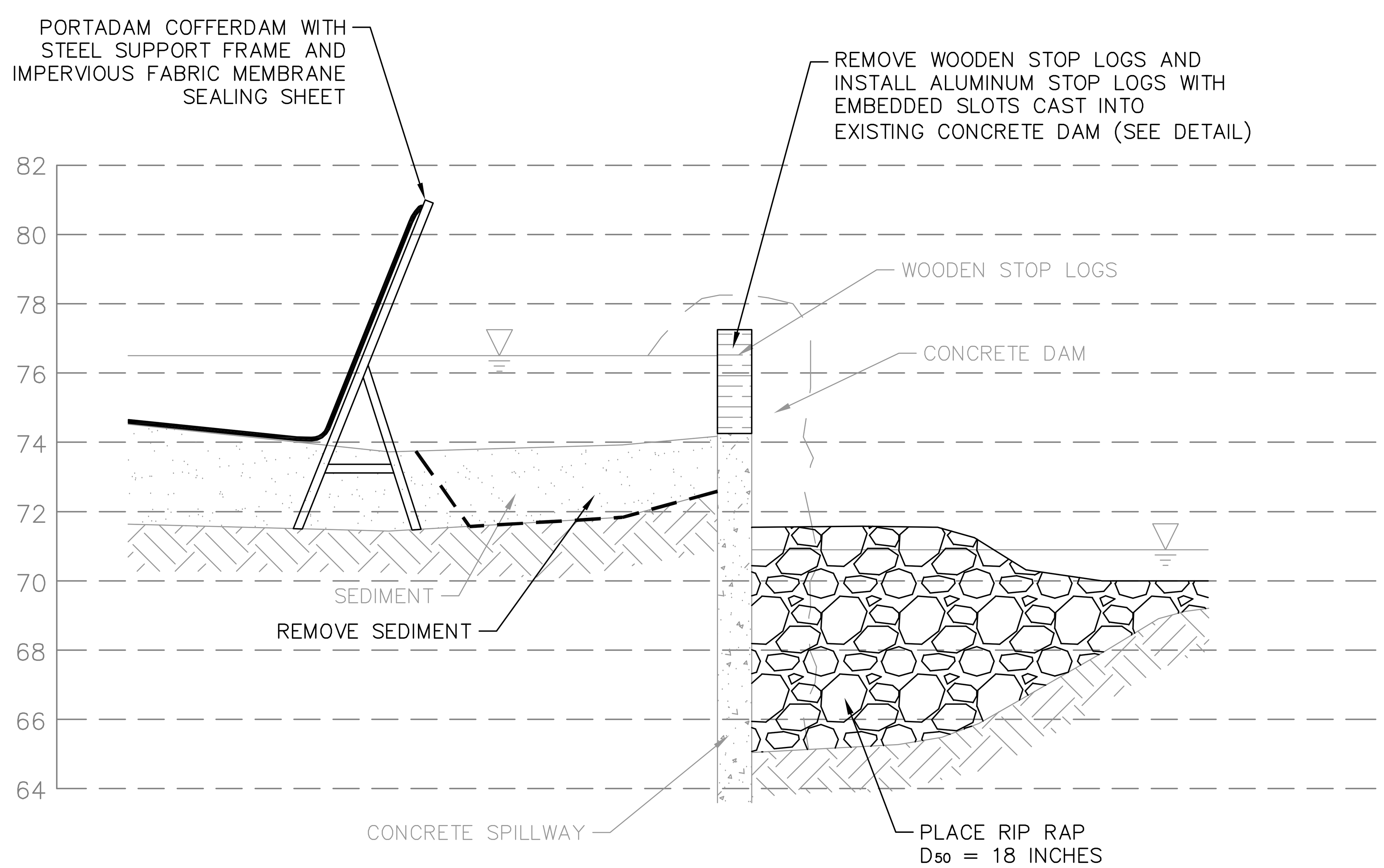
N.T.S.



SPILLWAY REPAIR DETAIL

0 2 4

SCALE: 1" = 2'



SPILLWAY TYPICAL SECTION

N.T.S.

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(617) 695-3310

TOWN OF DEDHAM
MASSACHUSETTS

DEDHAM DEPARTMENT
OF PUBLIC WORKS

SEAL

KEY PLAN

SCALE

No.	DATE	BY	Description

REVISIONS

DESIGNED BY PAR

DRAWN BY PAR

CHECKED BY PAR

DATE FEBRUARY 2016

TITLE

TYPICAL SECTIONS AND DETAILS

PROJECT NO. 50075664

C-501

SHEET NO. 3 OF 3

Attachment C: HEC-RAS Results Table

Profile Output Table - Breach

File Options Std. Tables User Tables Locations Help

HEC-RAS Plan: SunnyDay River: MotherBrook Reach: 1 Profile: Max WS

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Vel Chnl	Vel Total	Volume
			(cfs)	(ft)	(ft)	(ft/s)	(ft/s)	(acre-ft)
1	14917.57	Max WS	30.00	77.31	78.88	1.42	1.42	76.88
1	14678.47	Max WS	30.00	76.48	78.50	1.28	1.28	76.76
1	14437.47	Max WS	30.00	75.65	78.28	0.87	0.86	76.60
1	14278.06	Max WS	30.00	75.10	78.24	0.48	0.48	76.42
1	14239.99		Culvert					
1	14209.94	Max WS	30.00	74.87	78.24	0.51	0.51	76.33
1	14152.98	Max WS	30.00	74.65	78.24	0.24	0.24	76.15
1	14034.54	Max WS	30.00	74.26	78.23	0.18	0.18	75.64
1	13968.54	Max WS	30.00	74.03	78.23	0.09	0.09	75.27
1	13908.55	Max WS	30.00	73.83	78.23	0.13	0.13	74.88
1	13835.49	Max WS	30.00	73.58	78.23	0.08	0.08	74.37
1	13747.6	Max WS	30.00	73.27	78.23	0.06	0.06	73.52
1	13687.09	Max WS	30.00	73.06	78.23	0.05	0.05	72.80
1	13625.62	Max WS	30.00	72.85	78.23	0.05	0.05	71.94
1	13531.66	Max WS	30.00	72.53	78.23	0.05	0.05	70.57
1	13396.96	Max WS	30.00	72.06	78.23	0.05	0.05	68.58
1	13275.77	Max WS	30.00	71.64	78.23	0.03	0.03	66.26
1	13136.06	Max WS	30.00	71.16	78.23	0.02	0.02	62.58
1	12829.26	Max WS	30.00	70.10	78.23	0.08	0.08	51.95
1	12730.73	Max WS	30.00	69.76	78.23	0.09	0.09	49.62
1	12645.91	Max WS	30.00	69.47	78.23	0.05	0.05	48.71
1	12574.01	Max WS	30.00	69.22	78.23	0.05	0.05	47.73
1	12529.86	Max WS	30.00	69.07	78.23	0.05	0.05	47.14
1	12519.78		Inl Struct					
1	12509.7	Max WS	30.00	69.00	70.80	0.46	0.46	47.03
1	12453.66	Max WS	30.00	68.88	70.73	0.99	0.99	46.96
1	12407.11	Max WS	30.00	68.78	70.64	1.16	1.16	46.93
1	12322.99	Max WS	30.00	68.60	70.48	1.09	1.09	46.88
1	12228.89	Max WS	30.00	68.40	70.03	2.41	2.41	46.84
1	12157.11	Max WS	30.00	68.24	69.82	1.09	1.09	46.81
1	12100.13	Max WS	30.00	68.12	69.69	1.60	1.60	46.78
1	11980.08	Max WS	30.00	67.86	69.16	1.89	1.89	46.73
1	11884.15	Max WS	30.00	67.65	68.85	1.08	1.05	46.68
1	11793.84	Max WS	30.00	67.46	68.60	1.40	1.40	46.63
1	11695.85	Max WS	30.00	67.25	68.04	1.60	1.48	46.58
1	11544	Max WS	30.00	66.02	67.41	0.87	0.71	46.47
1	11346.18	Max WS	29.95	64.43	67.30	0.05	0.05	44.95
1	11136.8	Max WS	29.96	62.73	67.30	0.08	0.08	42.40
1	11083.69		Bridge					
1	11032.8	Max WS	29.96	61.89	67.30	0.05	0.05	41.37
1	10812.21	Max WS	29.98	60.11	67.30	0.02	0.02	35.75
1	10599.8	Max WS	30.00	58.40	67.30	0.02	0.02	27.49
1	10368.25	Max WS	30.02	56.52	67.30	0.01	0.01	16.30
1	10117.64	Max WS	30.05	54.50	67.30	0.01	0.01	1.61
1	10083.22		Inl Struct					
1	10051.66	Max WS	30.05	51.09	51.46	2.56	2.56	0.30

Figure 1: Sunny Day Non-Breach HEC-RAS Results

Profile Output Table - Breach

File Options Std. Tables User Tables Locations Help

HEC-RAS Plan: SD - Breach River: MotherBrook Reach: 1 Profile: Max WS

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Vel Chnl	Vel Total	Volume
			(cfs)	(ft)	(ft)	(ft/s)	(ft/s)	(acre-ft)
1	14917.57	Max WS	30.00	77.31	78.88	1.41	1.41	97.35
1	14678.47	Max WS	30.00	76.48	78.50	1.28	1.28	97.23
1	14437.47	Max WS	30.00	75.65	78.28	0.87	0.86	97.07
1	14278.06	Max WS	30.00	75.10	78.24	0.48	0.48	96.89
1	14239.99		Culvert					
1	14209.94	Max WS	30.00	74.87	78.24	0.51	0.51	96.79
1	14152.98	Max WS	30.00	74.65	78.24	0.24	0.24	96.62
1	14034.54	Max WS	30.00	74.26	78.23	0.18	0.18	96.11
1	13968.54	Max WS	30.00	74.03	78.23	0.09	0.09	95.73
1	13908.55	Max WS	30.00	73.83	78.23	0.13	0.13	95.34
1	13835.49	Max WS	30.00	73.58	78.23	0.08	0.08	94.84
1	13747.6	Max WS	30.00	73.27	78.23	0.06	0.06	93.99
1	13687.09	Max WS	30.00	73.06	78.23	0.05	0.05	93.27
1	13625.62	Max WS	30.00	72.85	78.23	0.05	0.05	92.41
1	13531.66	Max WS	30.00	72.53	78.23	0.05	0.05	91.03
1	13396.96	Max WS	29.99	72.06	78.23	0.05	0.05	89.04
1	13275.77	Max WS	29.99	71.64	78.23	0.03	0.03	86.73
1	13136.06	Max WS	30.00	71.16	78.23	0.02	0.02	83.05
1	12829.26	Max WS	29.99	70.10	78.23	0.08	0.08	72.41
1	12730.73	Max WS	29.99	69.76	78.23	0.09	0.09	70.08
1	12645.91	Max WS	29.99	69.47	78.23	0.05	0.05	69.18
1	12574.01	Max WS	29.99	69.22	78.23	0.05	0.05	68.19
1	12529.86	Max WS	29.99	69.07	78.23	0.05	0.05	67.61
1	12519.78		Inl Struct					
1	12509.7	Max WS	1426.68	69.00	76.20	3.04	2.92	67.49
1	12453.66	Max WS	1414.21	68.88	76.01	4.28	3.46	66.92
1	12407.11	Max WS	1396.24	68.78	75.74	5.30	3.88	66.51
1	12322.99	Max WS	1360.06	68.60	75.31	5.32	4.12	65.84
1	12228.89	Max WS	1259.79	68.40	74.03	7.74	5.65	65.24
1	12157.11	Max WS	1244.08	68.24	73.75	5.98	3.93	64.78
1	12100.13	Max WS	1208.41	68.12	73.38	5.93	4.57	64.33
1	11980.08	Max WS	1202.94	67.86	72.40	7.13	4.89	63.63
1	11884.15	Max WS	1183.06	67.65	71.55	6.95	4.88	63.09
1	11793.84	Max WS	1073.88	67.46	71.03	5.42	3.10	62.48
1	11695.85	Max WS	1029.71	67.25	70.53	4.43	2.46	61.26
1	11544	Max WS	941.00	66.02	69.13	6.02	3.07	59.49
1	11346.18	Max WS	401.50	64.43	68.65	0.41	0.40	56.26
1	11136.8	Max WS	386.31	62.73	68.65	0.73	0.73	52.19
1	11083.69		Bridge					
1	11032.8	Max WS	386.31	61.89	68.65	0.50	0.50	50.74
1	10812.21	Max WS	385.18	60.11	68.65	0.21	0.21	43.63
1	10599.8	Max WS	384.08	58.40	68.65	0.17	0.17	33.63
1	10368.25	Max WS	382.89	56.52	68.65	0.14	0.14	20.42
1	10117.64	Max WS	381.38	54.50	68.65	0.12	0.12	3.21
1	10083.22		Inl Struct					
1	10051.66	Max WS	381.10	51.09	52.56	4.92	4.92	1.58

Figure 2: Sunny Day Breach HEC-RAS Results

Profile Output Table - Breach

File Options Std. Tables User Tables Locations Help

HEC-RAS Plan: P100yr River: MotherBrook Reach: 1 Profile: Max WS

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Vel Chnl	Vel Total	Volume
			(cfs)	(ft)	(ft)	(ft/s)	(ft/s)	(acre-ft)
1	14917.57	Max WS	1499.76	77.31	87.05	4.26	3.65	157.64
1	14678.47	Max WS	1499.56	76.48	86.80	4.50	3.08	155.18
1	14437.47	Max WS	1499.16	75.65	86.69	2.86	1.75	151.43
1	14278.06	Max WS	1498.84	75.10	86.32	4.67	4.33	148.93
1	14239.99		Culvert					
1	14209.94	Max WS	1473.05	74.87	80.56	14.11	13.75	148.57
1	14152.98	Max WS	1489.33	74.65	81.06	6.16	6.01	148.19
1	14034.54	Max WS	1492.29	74.26	81.30	2.64	2.48	146.71
1	13968.54	Max WS	1492.93	74.03	81.32	1.93	1.82	145.62
1	13908.55	Max WS	1492.29	73.83	81.28	2.37	2.16	144.55
1	13835.49	Max WS	1492.28	73.58	81.28	1.90	1.88	143.30
1	13747.6	Max WS	1492.28	73.27	81.29	1.57	1.53	141.51
1	13687.09	Max WS	1492.82	73.06	81.29	1.23	1.20	139.96
1	13625.62	Max WS	1492.78	72.85	81.29	1.22	1.19	138.19
1	13531.66	Max WS	1492.27	72.53	81.28	1.24	1.21	135.51
1	13396.96	Max WS	1492.25	72.06	81.27	1.23	1.18	131.65
1	13275.77	Max WS	1492.23	71.64	81.28	0.87	0.85	127.45
1	13136.06	Max WS	1492.20	71.16	81.28	0.73	0.72	121.32
1	12829.26	Max WS	1492.12	70.10	81.16	2.40	2.35	104.90
1	12730.73	Max WS	1492.08	69.76	81.02	3.34	3.34	101.29
1	12645.91	Max WS	1492.07	69.47	81.11	1.77	1.75	100.00
1	12574.01	Max WS	1492.06	69.22	81.10	1.80	1.79	98.61
1	12529.86	Max WS	1492.05	69.07	81.10	1.80	1.76	97.76
1	12519.78		Inl Struct					
1	12509.7	Max WS	1492.04	69.00	76.41	3.07	2.94	97.52
1	12453.66	Max WS	1492.03	68.88	76.21	4.34	3.49	96.92
1	12407.11	Max WS	1492.02	68.78	75.93	5.39	3.90	96.48
1	12322.99	Max WS	1491.98	68.60	75.43	5.66	4.35	95.78
1	12228.89	Max WS	1491.81	68.40	74.52	7.57	4.82	95.08
1	12157.11	Max WS	1491.79	68.24	74.24	6.09	3.70	94.46
1	12100.13	Max WS	1491.78	68.12	73.85	6.33	4.73	93.89
1	11980.08	Max WS	1491.62	67.86	72.90	7.55	5.10	93.06
1	11884.15	Max WS	1489.64	67.65	72.13	7.31	4.88	92.40
1	11793.84	Max WS	1486.86	67.46	71.82	5.47	3.06	91.58
1	11695.85	Max WS	1483.94	67.25	71.52	4.17	2.18	89.53
1	11544	Max WS	1481.08	66.02	71.25	3.49	1.65	85.75
1	11346.18	Max WS	1479.95	64.43	71.11	0.89	0.85	78.77
1	11136.8	Max WS	1479.45	62.73	71.05	1.79	1.71	71.62
1	11083.69		Bridge					
1	11032.8	Max WS	1479.45	61.89	71.06	1.32	1.23	69.29
1	10812.21	Max WS	1479.36	60.11	71.07	0.60	0.59	59.01
1	10599.8	Max WS	1479.28	58.40	71.07	0.50	0.50	45.63
1	10368.25	Max WS	1479.18	56.52	71.06	0.43	0.43	28.56
1	10117.64	Max WS	1479.05	54.50	71.06	0.36	0.35	6.58
1	10083.22		Inl Struct					
1	10051.66	Max WS	1479.03	51.09	55.80	4.85	4.85	4.33

Figure 3: 100-yr Non-Breach HEC-RAS Results

Profile Output Table - Breach

File Options Std. Tables User Tables Locations Help

HEC-RAS Plan: P100yr - Breach River: MotherBrook Reach: 1 Profile: Max WS

Reload Data

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Vel Chnl	Vel Total	Volume
			(cfs)	(ft)	(ft)	(ft/s)	(ft/s)	(acre-ft)
1	14917.57	Max WS	1499.76	77.31	87.05	4.26	3.65	174.27
1	14678.47	Max WS	1499.56	76.48	86.80	4.50	3.08	171.81
1	14437.47	Max WS	1499.16	75.65	86.69	2.86	1.75	168.07
1	14278.06	Max WS	1498.84	75.10	86.32	4.67	4.33	165.57
1	14239.99		Culvert					
1	14209.94	Max WS	1498.38	74.87	80.54	14.40	14.04	165.21
1	14152.98	Max WS	1498.46	74.65	81.06	6.21	6.05	164.83
1	14034.54	Max WS	1498.10	74.26	81.29	2.65	2.49	163.35
1	13968.54	Max WS	1497.87	74.03	81.31	1.94	1.82	162.26
1	13908.55	Max WS	1497.68	73.83	81.27	2.39	2.17	161.19
1	13835.49	Max WS	1497.47	73.58	81.28	1.90	1.89	159.95
1	13747.6	Max WS	1497.24	73.27	81.28	1.58	1.54	158.16
1	13687.09	Max WS	1497.07	73.06	81.29	1.24	1.20	156.61
1	13625.62	Max WS	1496.91	72.85	81.28	1.23	1.20	154.84
1	13531.66	Max WS	1496.75	72.53	81.28	1.25	1.21	152.16
1	13396.96	Max WS	1494.15	72.06	81.27	1.23	1.19	148.30
1	13275.77	Max WS	1493.28	71.64	81.27	0.87	0.86	144.11
1	13136.06	Max WS	1492.20	71.16	81.27	0.73	0.72	137.99
1	12829.26	Max WS	1490.75	70.10	81.16	2.40	2.35	121.58
1	12730.73	Max WS	1486.61	69.76	81.02	3.33	3.33	117.98
1	12645.91	Max WS	1486.25	69.47	81.11	1.77	1.75	116.69
1	12574.01	Max WS	1485.95	69.22	81.10	1.79	1.79	115.30
1	12529.86	Max WS	1485.75	69.07	81.09	1.80	1.75	114.45
1	12519.78		Inl Struct					
1	12509.7	Max WS	3248.68	69.00	78.51	4.95	4.61	114.21
1	12453.66	Max WS	3246.12	68.88	78.13	6.81	5.27	113.36
1	12407.11	Max WS	3240.31	68.78	77.81	7.72	5.22	112.70
1	12322.99	Max WS	3084.93	68.60	76.92	8.34	6.02	111.60
1	12228.89	Max WS	2998.02	68.40	76.36	8.20	4.04	110.24
1	12157.11	Max WS	2983.46	68.24	76.14	6.86	3.46	108.86
1	12100.13	Max WS	2970.92	68.12	75.64	7.89	4.97	107.68
1	11980.08	Max WS	2955.81	67.86	74.37	10.36	6.78	106.26
1	11884.15	Max WS	2880.91	67.65	73.14	10.67	6.84	105.31
1	11793.84	Max WS	2832.76	67.46	72.78	7.78	4.26	104.19
1	11695.85	Max WS	2612.17	67.25	72.43	5.37	2.77	101.20
1	11544	Max WS	2384.25	66.02	72.09	4.45	2.06	95.87
1	11346.18	Max WS	2346.87	64.43	71.93	1.25	1.17	87.36
1	11136.8	Max WS	2313.86	62.73	71.82	2.51	2.37	79.15
1	11083.69		Bridge					
1	11032.8	Max WS	2313.86	61.89	71.82	1.87	1.72	76.51
1	10812.21	Max WS	2309.67	60.11	71.85	0.86	0.85	65.13
1	10599.8	Max WS	2305.51	58.40	71.85	0.73	0.72	50.60
1	10368.25	Max WS	2300.63	56.52	71.85	0.63	0.62	32.22
1	10117.64	Max WS	2294.37	54.50	71.85	0.53	0.51	8.63
1	10083.22		Inl Struct					
1	10051.66	Max WS	2292.62	51.09	56.90	5.92	5.92	6.01

Figure 4: 100-yr Breach HEC-RAS Results

Attachment D:
Geotechnical Report, March 2016, prepared by GEI Consultants, Inc.

Consulting Engineers and Scientists

March 31, 2016
Project 1510430

Mr. Michael Pelletier
Dewberry
280 Summer Street, 10th Floor
Boston, MA 02210

Dear Mr. Pelletier:

Re: Geotechnical Services
Colburn Street Dam
Dedham, Massachusetts

This letter presents the results of our subsurface investigations and geotechnical services for the Colburn Street Dam located in Dedham, Massachusetts. This letter presents our evaluation of the dam and recommendations for remediation.

Scope

Our scope consisted of the following:

- Performed a subsurface exploration consisting of:
  - Two test borings to observe soil and bedrock conditions.
  - Hand-held probes on the upstream side of the dam to evaluate sediment depths and the upstream geometry of the dam.
- Performed a stability analysis of the dam for the normal pool and design flood cases.
- Developed recommendations for improving the condition of the dam.
- Prepared this letter report presenting the results of the subsurface explorations and our recommendations.

Mr. Peter Garvey of Dewberry authorized our work with a signed contract dated August 12, 2015.

Site and Project Description

Colburn Street Dam is a stone masonry structure located on Mother Brook in Dedham, Massachusetts. Mother Brook is a stream which conveys water from the Charles River to the Neponset River. The dam is about 200 feet east of the intersection of Colburn Street and Bussey Street and adjacent to Condon Park, which has a baseball field and small playground. The major use of the reservoir appears to be recreational. The dam is approximately 95 feet long and 9 feet high at its shortest point, and 13 feet high at its tallest point. During periods of lower flow, water passes

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400 Unicorn Park Drive, Woburn, MA 01801
781.721.4000 fax: 781.721.4073

through a sluiceway notch, which is about 2 feet deep and 4 feet wide with the stop logs installed. During periods of higher flow, the dam is inundated.

We understand that based on a July 2013 inspection, the condition of the dam was downgraded to "fair" and the hazard classification of the dam was changed to "Significant" following Massachusetts Department of Conservation and Recreation (DCR) Office of Dam Safety guidelines. The downgraded condition of the dam was based on several deficiencies including downstream scour, seepage through the masonry face of the dam, large voids between masonry stones, and leakage through the stop logs in the sluiceway.

The new topographic survey performed for this project generally agrees with the findings of the July 2013 inspection. The stream bed downstream of the dam is generally at about El. 70. The survey data shows that the material downstream of the dam has been scoured to El. 67 along the face of the dam, and El. 65 in front of the spillway. Water levels in the reservoir and Mother Brook, as measured on August 27, 2015, were at El. 76.5 and El. 71 respectively.

All elevations in this report are referenced to North American Vertical Datum of 1988 (NAVD 88).

Exploration Program *(Borings)*

GeoLogic, Inc., of Norfolk, Massachusetts, drilled two borings (B101 and B102) on the banks of the brook from August 26 to 28, 2015. A GEI field engineer monitored the drilling and visually classified soil and bedrock samples in the field. Borings were located by taping from existing site features, and ground surface elevations at the borings were estimated based on existing plans. Boring locations are shown in the survey plan in Appendix A. Boring logs are provided in Appendix B.

The borings were advanced using a track-mounted CME-45 drill rig and drive and wash methods. Soil was sampled continuously from ground surface to the top of bedrock. Twenty feet of rock core was obtained in B101, and 25 feet in B102. Following termination of the borehole, drill cuttings were used to backfill the hole.

Exploration Program *(Hand-Auger Probes)*

GEI performed 25 hand probes on the upstream side of the dam on September 15, 2015 to evaluate the depth to sediment and approximate geometry of the dam. A small boat was used to access these locations upstream of Mother Brook and along several lines parallel to the dam. Profiles developed from the auger probes are included in Appendix C.

Subsurface Conditions

The soil encountered in the borings consisted of 5.5 to 7.5 feet of silty sand with gravel, overlying highly fractured Granite bedrock.

The silty sand with gravel was generally described as well-graded brown sand with about 30 percent of the constituents being fine to coarse gravel, and 20 percent being non-plastic fines. N-values ranged from 4 to 17 blows per foot (bpf) which is indicative of a very loose to medium dense compactness.

Bedrock was encountered at about El. 75 in B101 and El. 70 in B102. Bedrock was cored at the depths of 7 to 26.5 in B101 and 10.5 to 35.5 in B102. The bedrock was generally described as moderately hard to hard, weathered, and severely jointed granite. The Rock Quality Designation (RQD) ranged from 0 to 47 percent which is indicative of poor to very poor rock mass quality. Core recovery ranged from 40 percent to 100 percent.

Groundwater Conditions

Groundwater was not measured in the boreholes upon completion, but is anticipated to fluctuate with the water levels in the reservoir and in the stream. As measured by BSC Group on August 27, 2015, water levels in the reservoir and stream were recorded to be El. 76.5 and El. 71.0 respectively.

Analysis

We evaluated the stability of the dam based on the following:

- Pool levels at normal pool and design flood (100-year flood).
- We modeled the masonry dam strength using an internal friction angle, $\phi = 45^\circ$ with a cohesion intercept, c = 2,000 pounds per square foot (psf).
- Stability analyses were performed using the Slope/W program.
- We conservatively ignored the passive resistance at the toe of the dam.
- We assumed that, when the resultant bearing force from the overturning stability analysis is located outside of the middle 1/3 of the base (i.e. the kern), a crack forms between the base of the spillway and the bedrock, and the pressure within the crack is equal to the full head in the reservoir along the length of the crack.

Our calculations are provided in Appendix D.

Conclusions

Based on our stability evaluation for the dam using the assumptions above, we concluded the following:

- The factors of safety against sliding for the 100-year flood is greater than the factor of safety of 1.5 required in FERC guidance document dated October 2002, *Gravity Dams*.
- The spillway structure is not likely to overturn during the 100-year flood event.
- The continuing leakage through the face of the dam creates some risk of the dam blocks shifting and becoming unstable over time.

Some preliminary recommendations for mitigating seepage and scour are discussed below.

Recommendations

The following options could be considered to mitigate the seepage through the face of the dam:

- Perform grouting of the stone blocks of the dam to reduce seepage.
- Install a geomembrane on the back face of the dam to reduce seepage.
- Placed large riprap to fill scour holes at toe of dam.

- We recommend additional investigations to observe the bottom of dam/rock interface be performed if remedial design is advanced.

Limitations

This letter was prepared for the use of Dewberry and the Town of Dedham, exclusively. Our recommendations are based on the project information provided to us at the time of this report and may require modification if there are any changes in the nature, design, or location of the proposed structure. We cannot accept responsibility for designs based on our recommendations unless we are engaged to review the final plans and specifications to determine whether any changes in the project affect the validity of our recommendations and whether our recommendations have been properly implemented in the design.

The recommendations in this report are based in part on the data obtained from the subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. We, therefore, recommend that GEI be engaged to make site visits during construction to: a) check that the subsurface conditions exposed during construction are in general conformance with our design assumptions and b) ascertain that, in general, the work is being performed in compliance with the contract documents.

Our professional services for this project have been performed in accordance with generally accepted engineering practices; no warranty, express or implied, is made.

We appreciate the opportunity to work with you on this project. Please call me at 781-721-4030 if you have any questions.

Sincerely,

GEI CONSULTANTS, INC.

Jeanne LeFebvre

Jeanne A. LeFebvre, P.E.
Geotechnical Project Manager

Michael P. Walker, P.E.
Senior Practice Leader

JAL:mrb

Enclosures: Appendix A – Survey Plan
Appendix B - Boring Logs
Appendix C – Hand Probe Data
Appendix D - Calculations

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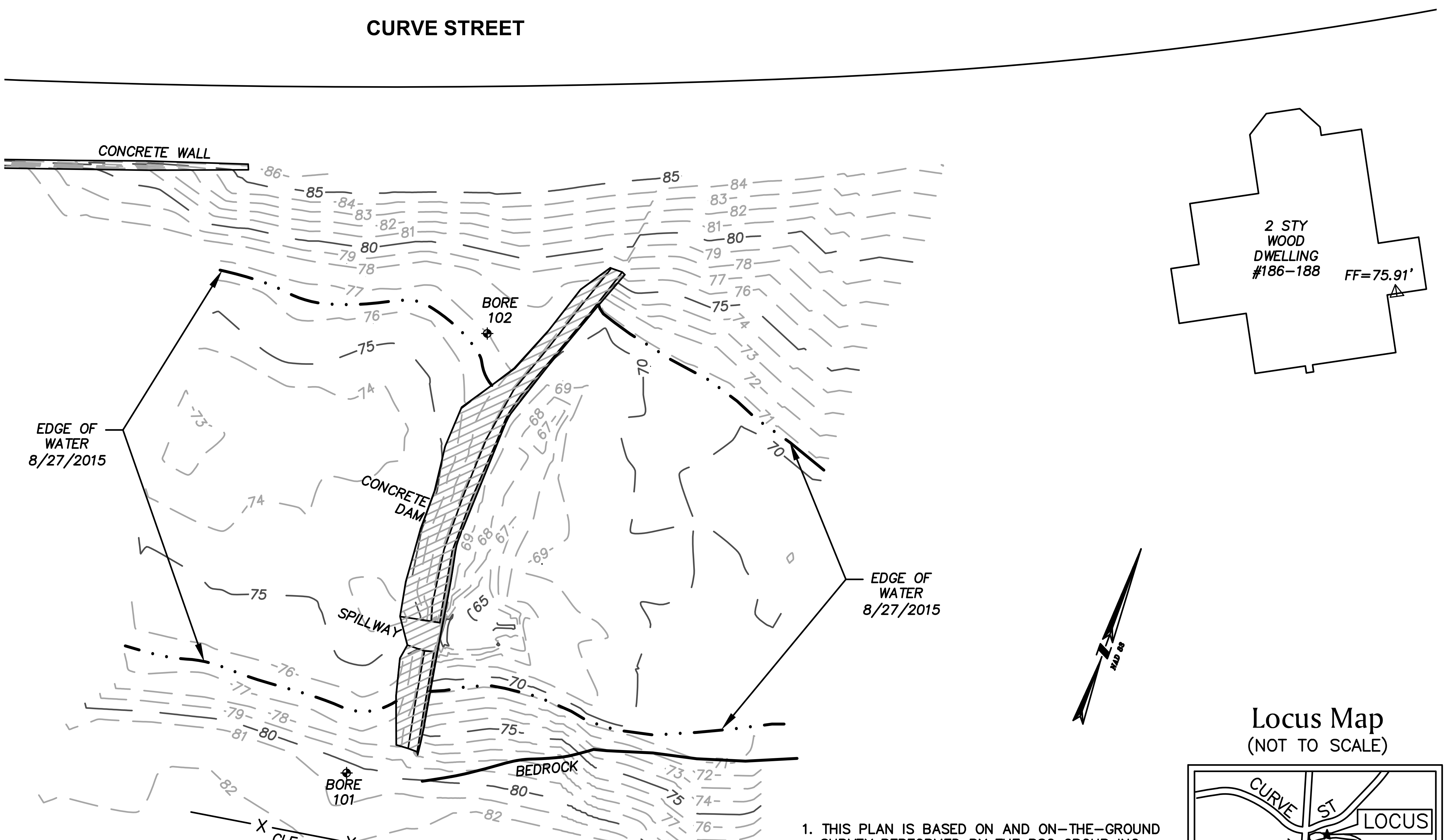
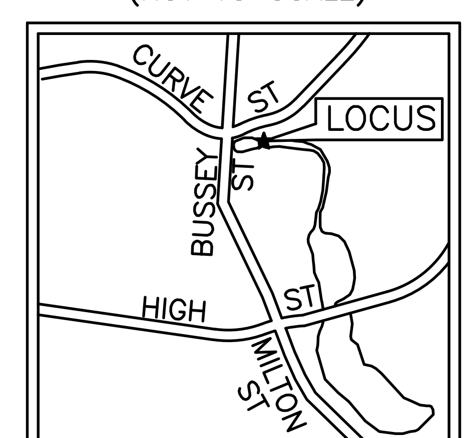
Appendix A

Survey Plan



1. THIS PLAN IS BASED ON AND ON–THE–GROUND SURVEY PERFORMED BY THE BSC GROUP INC. BETWEEN SEPTEMBER AND NOVEMBER, 2015.
2. HORIZONTAL DATUM IS BASED ON NAD '83 MASSACHUSETTS STATE PLANE COORDINATE SYSTEM AND VERTICAL DATUM IS BASED ON NAVD '88 AS ESTABLISHED BY GPS OBSERVATIONS.

Locus Map
(NOT TO SCALE)



EXISTING CONDITIONS PLAN

MOTHER BROOK DAM DEDHAM, MA

REVISIONS:

NO.	DATE	DESC.
1	11/16/15	Scour Topo & House Located

PREPARED FOR:

Dewbury-Goodkind, Inc.
280 Summer Street, 10th Floor
BOSTON, MA
02210-1131

BSC GROUP
803 Summer Street
Boston, Massachusetts
02127
617 896 4300

Scale: 1"=20'	Date: SEPT. 17, 2015
File: P:\1346400\S\D\1346400TOPO–REV1.dwg	

Dwg. No: REV1.dwg	Filed:	Sheet
Job No.: 1–3464.00	–––	1 of 1

Appendix B

Boring Logs

BORING INFORMATION

LOCATION: See Plan

GROUND SURFACE EL. (ft): NM | **DATE START/END:** 8/26/2015 - 8/27/2015

VERTICAL DATUM: | **DRILLING COMPANY:** Geologic, Inc.

TOTAL DEPTH (ft): 26.5 | **DRILLER NAME:** D. Sheldon

LOGGED BY: M.Perez-Canals | **RIG TYPE:** CME 45 Track Mounted

BORING B101

DRILLING INFORMATION

HAMMER TYPE: Automatic | **CASING I.D./O.D.:** 4 inch / 4.5 inch | **CORE BARREL TYPE:** NX

AUGER I.D./O.D.: NA / NA | **DRILL ROD O.D.:** NM | **CORE BARREL I.D./O.D.** NA / NA

DRILLING METHOD: Rotary Drilling with Casing

WATER LEVEL DEPTHS (ft): Not measured

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

Elev. (ft)	Depth (ft)	Sample No.	Sample Depth (ft)	Pen./Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Layer Name	Soil and Rock Description
		1	0 to 2	24/5	4-6-9-7			S1: SILTY SAND (SM): ~70% fine to medium sand, ~20% non plastic fines, ~10% fine to medium gravel, dark brown.
		2	2 to 4	24/11	14-8-9-4			S2: SILTY SAND WITH GRAVEL (SM): ~40% fine to coarse sand, ~35% fine to coarse gravel, 15% non plastic fines, dark brown.
	5	3	4 to 5.5	18/5	8-4-13-50/0"			S3: SILTY GRAVEL WITH SAND (GM): ~40% fine to coarse gravel, ~30% fine to coarse sand, ~20% non plastic fines, brown.
	10	1	7 to 12	60/56	22	Core barrel jammed, removed core. Core barrel jammed, removed core.	BEDROCK	C1: Grayish-pink, DEDHAM GRANITE, hard, weatherd joints from from gravel size to 5" apart, joints 45 to 90 degrees from horizontal. Core times (min/ft): ~6, ~14, ~18,~8,~11.
	15	2	12 to 17	60/57	8	Losing drill water. Core barrel jammed, removed core. Losing drill water. Core barrel jammed, removed core.	BEDROCK	C2: Grayish-pink, greenish-gray, DEDHAM GRANITE, hard, weathered joints from gravel size to 5" apart, joints 45 to 90 degrees from horizontal. Core times (min/ft): ~6, ~13, ~4,~10,~8.
	20	3	17 to 22	60/60	38	Core barrel jammed, removed core. Core barrel jammed, removed core. Core barrel jammed, removed core.	BEDROCK	C3: Grayish-pink, DEDHAM GRANITE, hard, weathered joints spaced from 1" to 7" apart, joints 0 to 90 degrees from horizontal. Core times (min/ft): ~4, ~9, ~11,~9,~12.
	25	4	22 to 26.5	54/54	43		BEDROCK	C4: Gryish-pink, DEDHAM GRANITE, hard weathered joints spaced from 1" to 9" apart, joints 0 to 45 degrees from horizontal. Last 10" of core very weathered. Core times (min/ft): ~7, ~7, ~8,~12,~20.
						Core barrel jammed, removed core.		Bottom of boring at depth 26.5 ft. Backfilled with drill cuttings.

NOTES:

PROJECT NAME: Colburn Street Dam

CITY/STATE: Dedham, Massachusetts

GEI PROJECT NUMBER: 1510430

GEI Consultants

BORING INFORMATION

LOCATION: See Plan
GROUND SURFACE EL. (ft): NM
VERTICAL DATUM:
TOTAL DEPTH (ft): 35.5
LOGGED BY: M. Perez-Canals
DATE START/END: 8/27/2015 - 8/28/2015
DRILLING COMPANY: Geologic, Inc.
DRILLER NAME: D. Sheldon
RIG TYPE: CME 45 Track Mounted

BORING B102

DRILLING INFORMATION

HAMMER TYPE: Automatic
AUGER I.D./O.D.: NA / NA
CASING I.D./O.D.: 3 inch / 3.5 inch
DRILL ROD O.D.: NM
CORE BARREL TYPE: NX
CORE BARREL I.D./O.D. NA / NA
DRILLING METHOD: Rotary Drilling with Casing
WATER LEVEL DEPTHS (ft): Not measured

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

Elev. (ft)	Depth (ft)	Sample No.	Sample Depth (ft)	Pen./ Rec. (in)	Blows per 6 in. or RQD	Drilling Remarks/ Field Test Data	Layer Name	Soil and Rock Description
		1	0 to 2	24/7	2-2-3-3			S1: SILTY SAND WITH GRAVEL (SM): ~40% fine to coarse sand, ~35% fine to medium gravel, ~15% non plastic fines, brown.
		2	2 to 4	24/12	5-4-6-8			S2: SILTY SAND WITH GRAVEL (SM): ~40% fine to coarse sand, ~30% fine to medium gravel, ~20% non plastic fines, some organics, brown.
	5	3	4 to 6	24/0	WOR-WOR-WOR-WOR	Casing fell and had to be pulled back. Possible void 4'-6'.		S3: No Recovery
		4	6 to 7.4	17/12	1-7-100/5"			S4: SILTY SAND WITH GRAVEL (SM): ~40% fine to coarse sand, ~35% fine to coarse gravel, ~15% non plastic fines, light brown-gray. Weathered rock on sampler tip.
	10	1	10.5 to 15.5	60/49	15	Sediment seeping through the downstream side of the dam.	BEDROCK	C1: Grayish-pink, DEDHAM GRANITE, hard, weathered joints from gravel size to 5" apart, joints at 0 to 45 degrees from horizontal. First 0-24" very weathered. Core times (min/ft): ~1, ~6, ~8,~6,~7.
	15	2	15.5 to 20.5	60/58	0	Core barrel jammed, removed core.		C2: Grayish-pink, DEDHAM GRANITE, hard, weathered joints from gravel size to 3" apart, at 0 to 90 degrees from horizontal. Core times (min/ft): ~6, ~8, ~12,~8,~13.
	20	3	20.5 to 25.5	60/60	0	Core barrel jammed, removed core. Sediment seeping through the downstream side of the dam from 18'-35.5'. Core barrel jammed, removed core.		C3: Similar to C2. Core times (min/ft): ~4, ~9, ~10,~5,~11.
	25	4	25.5 to 30.5	60/24	0	Core barrel jammed, removed core. Core barrel jammed, removed core.		C4: Gray, GABBRO, moderately hard, weathered joints from gravel size to 3" apart, at 0 to 45 degrees from horizontal. Core times (min/ft): ~7, ~8, ~5,~6,~7.
	30	5	30.5 to 35.5	60/57	47	Core barrel jammed, removed core. Core barrel jammed, removed core.		C5: Grayish-pink, greenish-gray, DEDHAM GRANITE, hard, weathered joints spaced from 1" to 14" apart, joints at 0 to 45 degrees from horizontal. Core times (min/ft): ~5, ~5, ~12,~7,~10.
	35					Material falling into the hole when core is removed.		Bottom of boring at depth 35.5 ft. Backfilled with drill cuttings

NOTES:

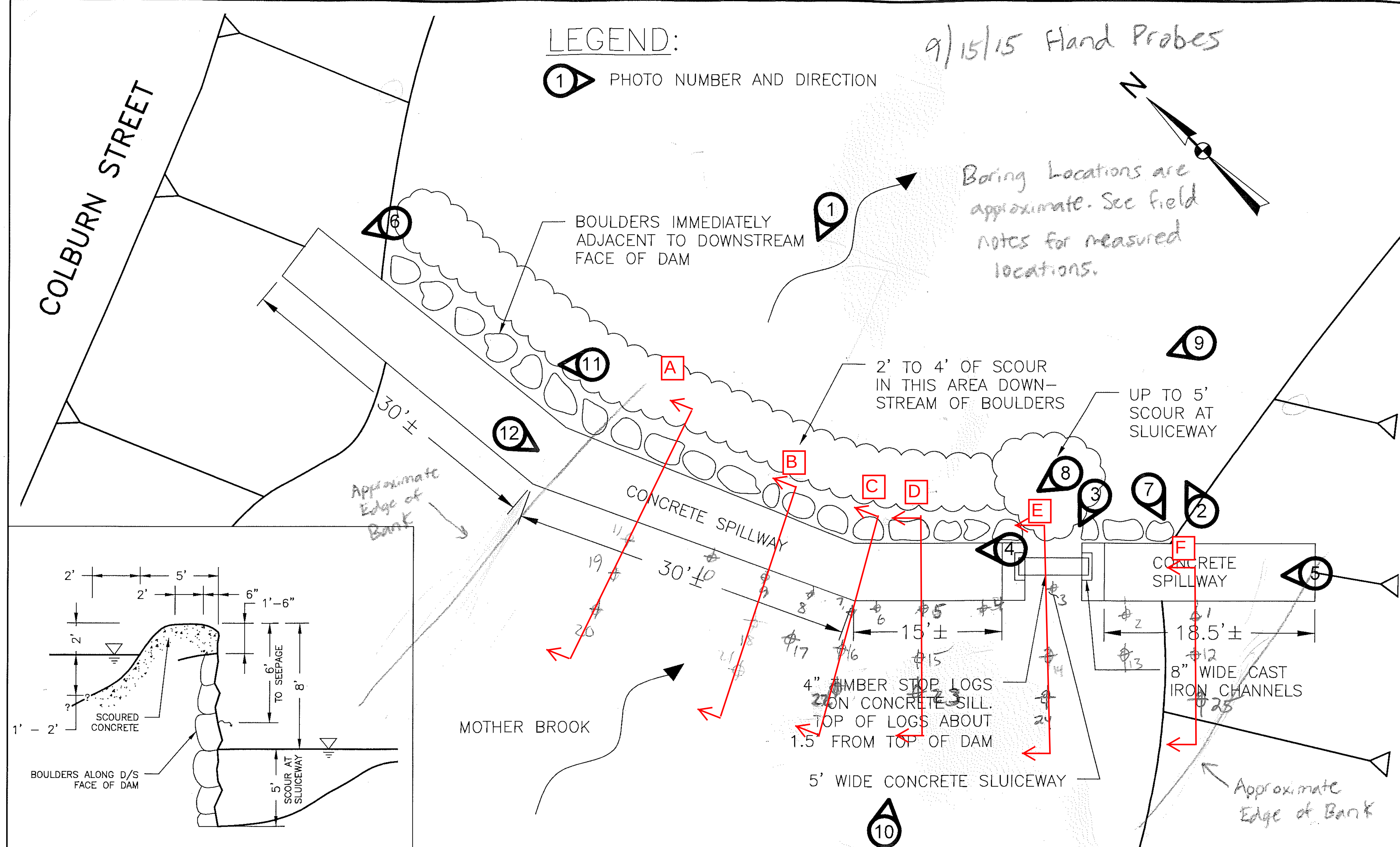
PROJECT NAME: Colburn Street Dam
CITY/STATE: Dedham, Massachusetts
GEI PROJECT NUMBER: 1510430

GEI Consultants

Appendix C

Probe Data

© 2013 – GZA GeoEnvironmental, Inc. GZA–J:\17,000–18,999\18802\18802–38.DJS\Figures\CADD\DAM INSPECTION SKETCH 1.dwg [Layout1] July 22, 2013 – 12:13pm connor.roman



NO.	ISSUE/DESCRIPTION	BY	DATE

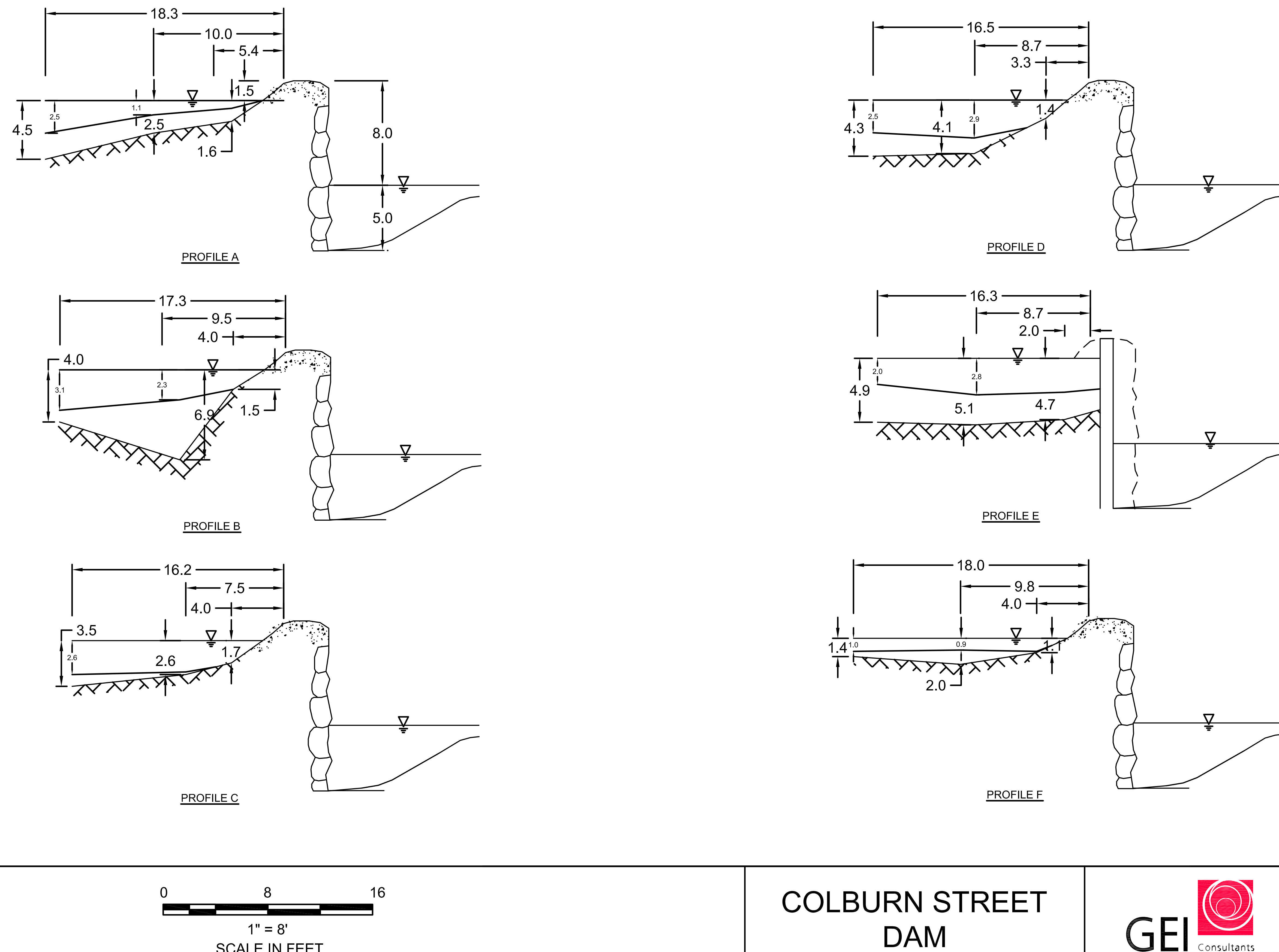
COLBURN STREET DAM
DEDHAM, MA

COLBURN STREET DAM INSPECTION
DAM SKETCH

PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
MASSACHUSETTS DEPARTMENT OF CONSERVATION AND RECREATION

PROJ MGR: DJS	REVIEWED BY: DJS	CHECKED BY: DJS	FIG OR DWG
DESIGNED BY: CFR	DRAWN BY: CFR	SCALE: N.T.S	1
DATE: 7/18/2013	PROJECT NO. 18802.38	REVISION NO.	SHEET NO. 1 OF 1



COLBURN STREET DAM

GEI Consultants

UPSTREAM X-SECTIONS

Appendix D

Calculations

Stability Analysis

Purpose:
Evaluate global and internal stability of the existing Colburn Street Dam. Calculate a factors of safety for the typical dam section (Section B) and a critical section near the sluice gate (Section D) based on the wall geometry and estimated soil properties. Evaluate factors of safety during Usual loading conditions (Maximum Storage Pool) and Unusual loading conditions (Flood Condition).

References:

[1] GZA GeoEnvironmental, Inc. Colburn Street Dam Follow-Up Inspection/Evaluation Report. July 15, 2013.
[2] Federal Energy Regulatory Commission (FERC). Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 3, Gravity Dams, Revised October 2002.
[3] Massachusetts Department of Conservation and Recreation (DCR) 302 CMR 10.00 – Dam Safety.
[4] USACE EM 1110-2-2200 - Gravity Wall Design; June 30, 1995
[5] Drawing: Colburn Street Dam Rehabilitation: Plans, Sections & Details Sheet No. 46, dated August, 1976, Prepared by Anderson-Nichols & Co. Inc.
[6] Drawing: Existing Conditions Plan, Prepared by BSC Group, Revision 1, dated November 16, 2015.
[7] McGregor, J.A., and Duncan, J.M. Performance and Use of the Standard Penetration Test in Geotechnical Engineering Practice. Virginia Polytechnic Institute and State University, Blacksburg, VA. 1998.

Approach:
We evaluated the dam considering three failure modes: (1) sliding at the base of the dam (at the bedrock/masonry interface, (2) internal stability (considering failures through the masonry), and (3) overturning. We considered bearing capacity to be acceptable by observation because the dam is most likely founded on bedrock.

We performed sliding and internal stability analyses using SLOPE/W, a limit-equilibrium stability analysis computer program developed by GEO-SLOPE International, Ltd. Stability was evaluated using the Spencer analysis method, which satisfies both moment and force equilibrium. Sliding between the bedrock and dam was defined using the block specified method. The entry and exit method was used to model an internal failure through the dam. We used interslice forces upstream and downstream of the dam from the sliding analysis to provide input active and passive forces for the overturning analysis. The overturning analysis was performed using rigid, free-body gravity analysis.

Criteria:
According to the Colburn Street Dam Inspection Report by GZA GeoEnvironmental [Ref. 1], the dam is classified as a “small” structure and the Hazard classification of the dam is “Significant.”

DCR Dam Safety Regulations [Ref. 2] provide recommended factors of safety (FSs) for calculations pertaining to shear-friction within a structure and the rock/concrete interface in the foundation (3.0 for usual loads and 2.0 for unusual loads). However, for our evaluation of sliding stability, we are conservatively ignoring the cohesion component of the bedrock/masonry interface. FERC [Ref. 3] provides a reduced FS for sliding along this interface, recognizing that, while cohesion does exist, it is difficult to quantify. The FERC-required FSs for sliding stability considering cohesion are 3.0 for usual loading and 2.0 for unusual loading, which are the same as recommended by DCR. If cohesion is neglected, then a FS of 1.5 is allowed for static cases, including the flood load case if the flood is not the PMF.

DCR does not provide recommended FSs for overturning analysis. Therefore, we used criteria in USACE Engineering Manual 1110-2-2200, Design of Gravity Dams [Ref. 4]. Instead of a FS, USACE requires that overturning be checked by evaluating the location of the resultant of the forces on the dam. For usual loading, the resultant must be within the middle third of the base. For unusual loading, the resultant must be within the middle half of the base.

The table below summarizes the criteria we used for the stability analyses.

		Criteria		
Scenario	Loading Condition	Masonry/Rock Interface Sliding FS	Overturning Moment Resultant Location	Internal Stability FS
Maximum Storage Pool – Pool El. 78.2	Usual	1.5	Within middle third	3.0
Flood Condition – Surcharge at El. 81.2	Unusual	1.5	Within middle half	2.0

Wall Geometry and Material Properties:

We developed the wall geometry for our analyses based on a repair drawing prepared by Anderson-Nichols [Ref. 5], survey performed by BSC Group [Ref. 6], and borings and hand probes performed by GEI.

According to the 1976 Plan, the top of is at El. 84.7± and is in reference to the Boston City Base. Converting to NAVD88 corresponds to approximately El. 78.2, which appears to be consistent with the survey performed by BSC. We assumed the dam is founded on rock at approximately El. 69 based on our observations of the dam.

The backslope of the dam was developed using measurements from the hand probes and projecting the backslope down to the assumed bedrock elevation. The retained soil behind the wall varied between about El. 74 and El. 76 based on the hand probes.

Pool/Surcharge and Tailwater Elevations were as follows:

Loading Condition	Scenario	Upstream Water El.	Downstream Water El.
Usual	Maximum Storage Pool	78.2 (Top of Dam)	71.0
Unusual	Flood Condition	81.2 (3' above T.O.D.)	72.0

The tailwater for the usual condition was based on the water level observed by BSC during their survey on August 27, 2015. The tailwater for the unusual condition was based on the FEMA Flood Insurance Rate Map (FIRM) for Norfolk County [Ref. 7]. The FIRM provides water surface elevations for the 100-year flood. We note that the 100-year flood elevation upstream of the dam is El. 81, which is close to the flood condition we evaluated (top of dam plus 3 feet).

We evaluated the dam at two cross sections, a typical dam cross section (called Section B), and the maximum section (called Section) D, which is close to the dam spillway, where deepest scour was observed. The mudline behind the wall was based on measurements taken from hand probes performed by GEI.

Material Properties:

Material properties are based on borings B101 and B102 located to the north and south ends of the dam, which were performed as part of this project and were observed by GEI. In general the borings encountered a layer of silty sand with gravel overlying highly fractured granite bedrock.

Silty Sand

We estimated friction angle (ϕ) based on a correlation with SPT N-values from McGregor and Duncan [Ref. 7] (see attached). SPT N-values in this layer generally ranged from 5 to 17 blows per foot, indicating a loose to medium dense soil. The average N-value in this layer, is 12 blows per foot. For the analysis, we assumed that $\phi = 32^{O}$ and that the total unit weight (γ_t) = 125 pcf.

Bedrock

Bedrock was modeled as an impenetrable material. The borings indicated that bedrock was encountered at El. 75.5 in B101 and El. 69.5 in B102 and was highly fractured. Based on BSC survey, the ground surface immediately downstream of the dam was at approximately El. 69, except at the spillway, where it was at about El. 65. We assumed that the bedrock under the dam was at El. 69. For both cross sections, we modeled the bedrock downstream of the dam based on BSC survey to account for scour.

Masonry Dam

According the 2013 GZA report, the masonry structure is in fair condition. Seepage was noted through the unmortared masonry face and stop logs at the sluiceway gate. Large voids were observed between the stones on the downstream face. For the analysis, we assumed that $\phi = 45^{\circ}$, c = 2,000 psf and that the total unit weight (γ_t) = 150 pcf.

Masonry / Bedrock Interface

We modelled the slip surface between the dam and rock interface as a thin, purely friction layer between bedrock and the dam.

For the analysis, we assumed $\phi = 45^{\circ}$ and the total unit weight (γ_t) = 150 pcf. As discussed above, we did not include a cohesion component to the strength of this interface.

Summary of material inputs:

Material	Unit Weight (pcf)	Material Model	Friction Angle φ_n (deg.)	Cohesion (psf)
Granular Material	125	Mohr-Coulomb	32	0
Masonry Dam	150	Mohr-Coulomb	45	2000
Rock/Masonry Interface	150	Mohr-Coulomb	45	0
Bedrock	--	Impenetrable	∞	∞

Other Loads:
According to USACE EM1110-2-2200 Section 3-3 Loads sub-section 3b, hydraulic jump of overflowing sections may reduce forces acting on the downstream face by as much at 60 percent. A horizontal load equivalent to 40 percent of the downstream hydraulic head was applied to the downstream face of the dam in the direction to counteract the tailwater pressure.

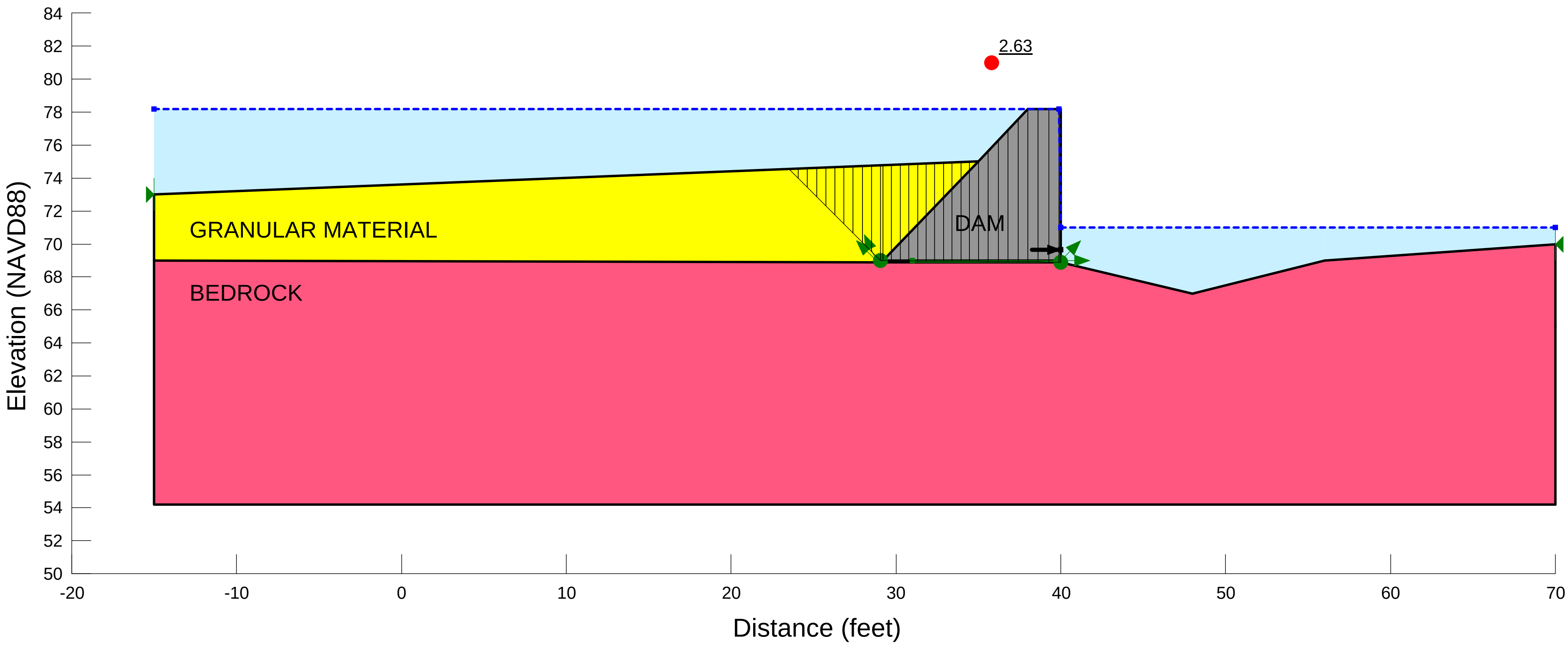
Results:

The calculated factors of safety for the stability analyses are as follows:

Section	Loading Condition	Sliding Analysis		Internal Structure Analysis		Overturning Analysis
		FS	Req'd. FS	FS	Req'd. FS	
Section B – Typical Section	Usual	2.63	1.5	4.30	3.0	Stable
	Unusual	1.99	1.5	3.24	2.0	Stable
Section D – Critical Section	Usual	2.54	1.5	4.28	3.0	Stable
	Unusual	1.88	1.5	3.05	2.0	Stable

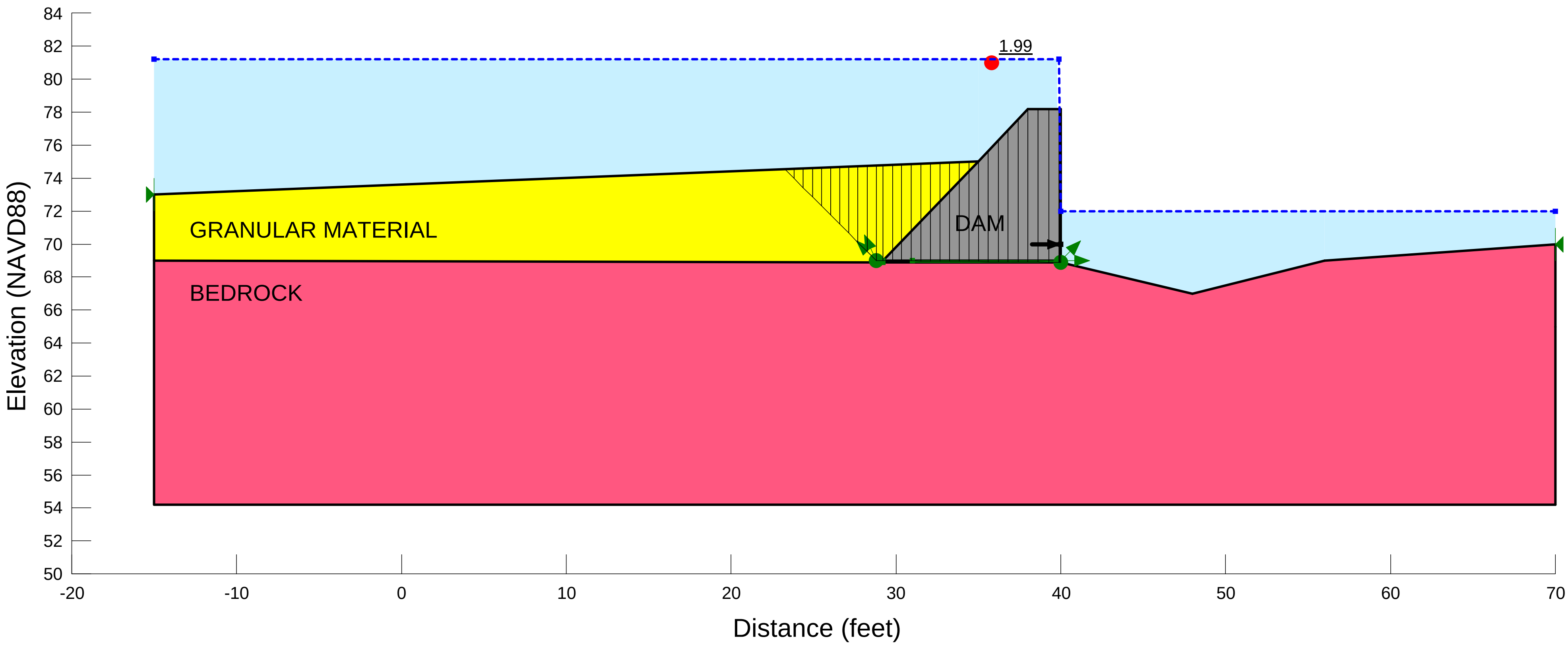
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Typical Section - Section B
Normal Pool - Slide Failure



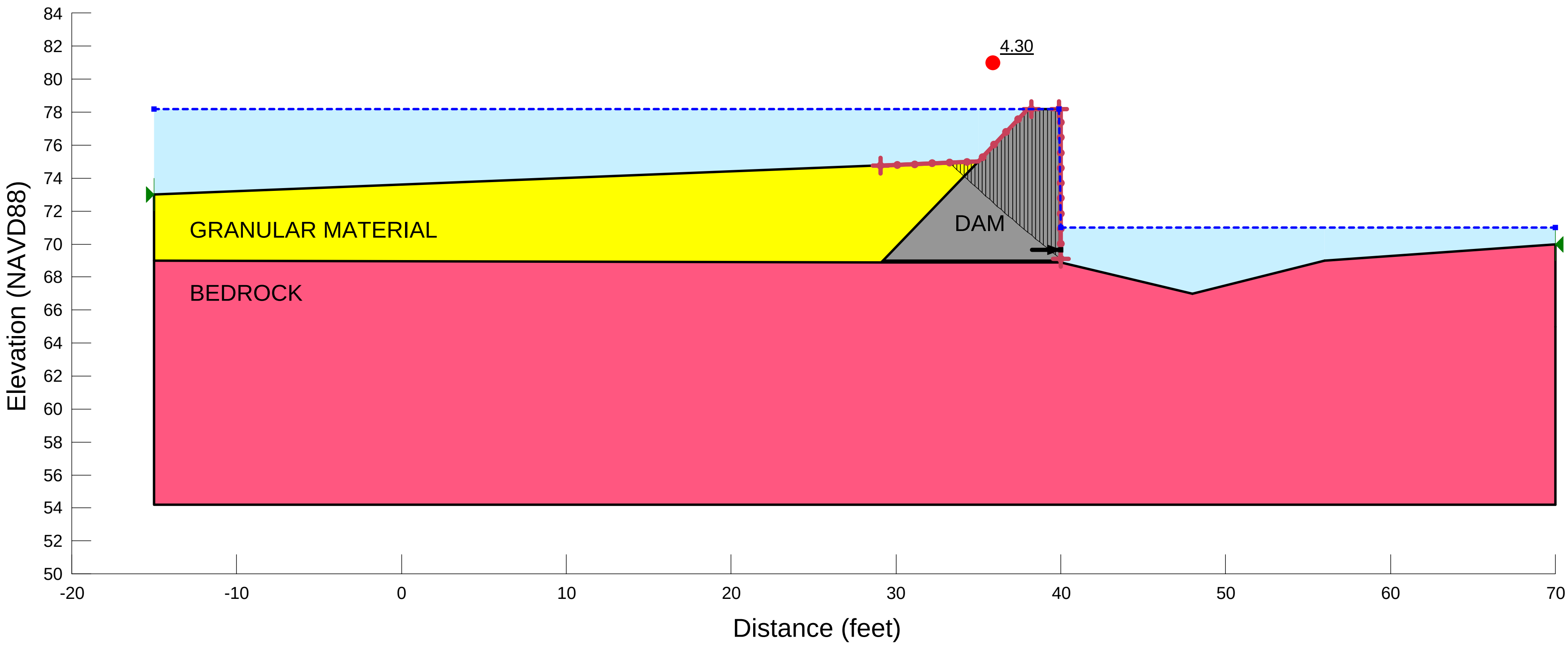
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Typical Section - Section B
Flood Condition - Slide Failure



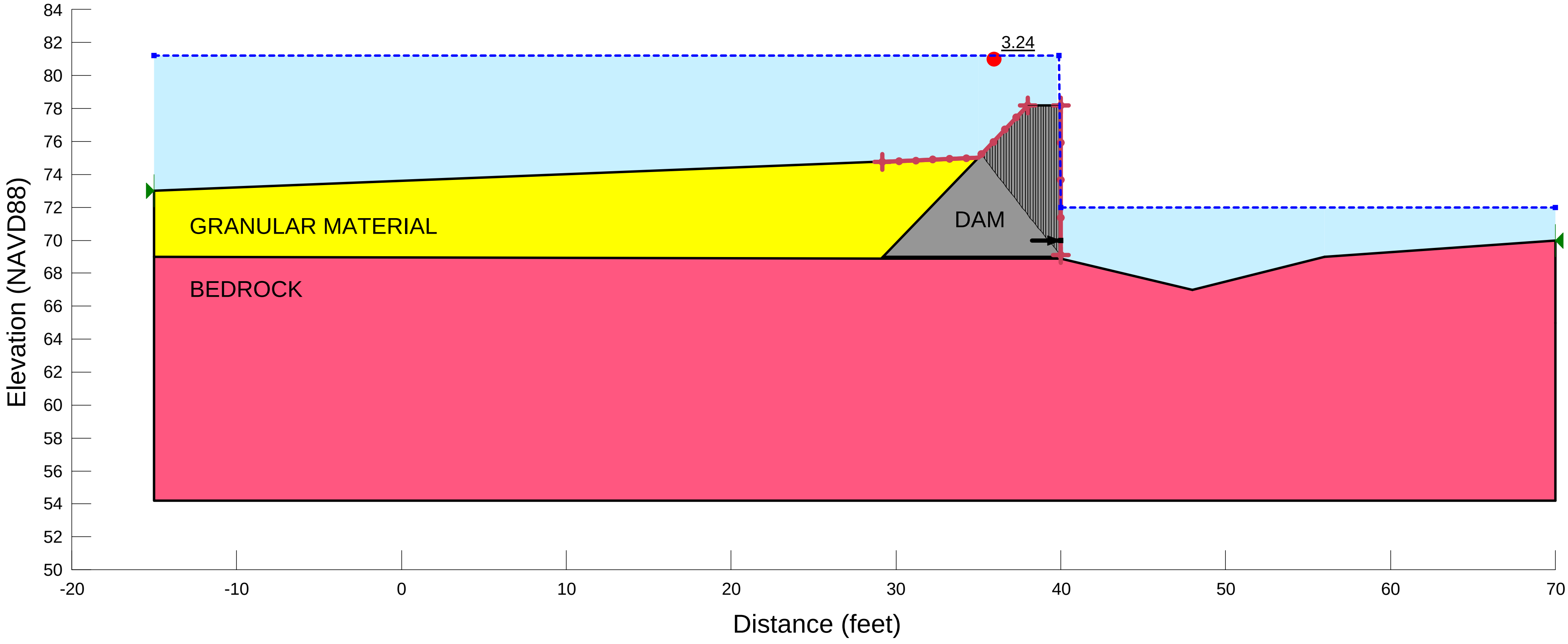
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Typical Section - Section B
Normal Pool - Dam Failure



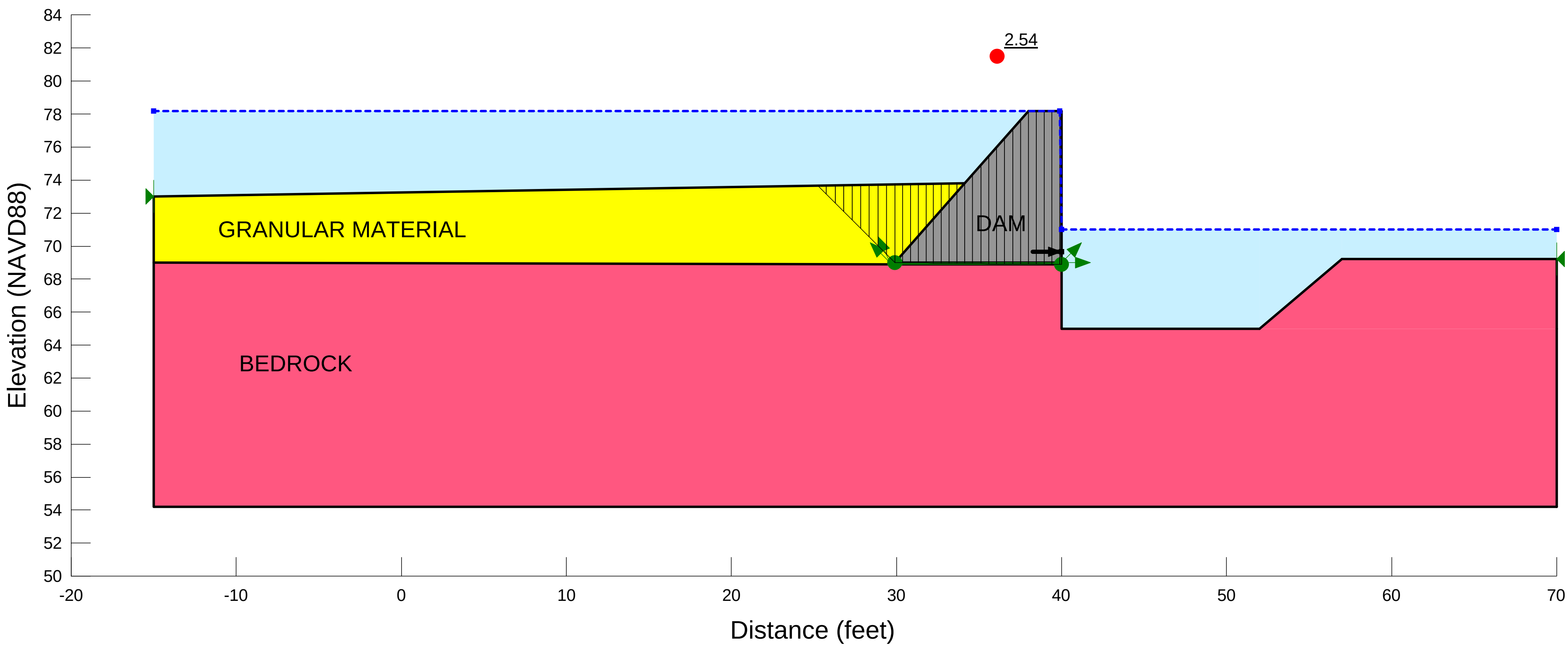
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Typical Section - Section B
Flood Condition - Dam Failure



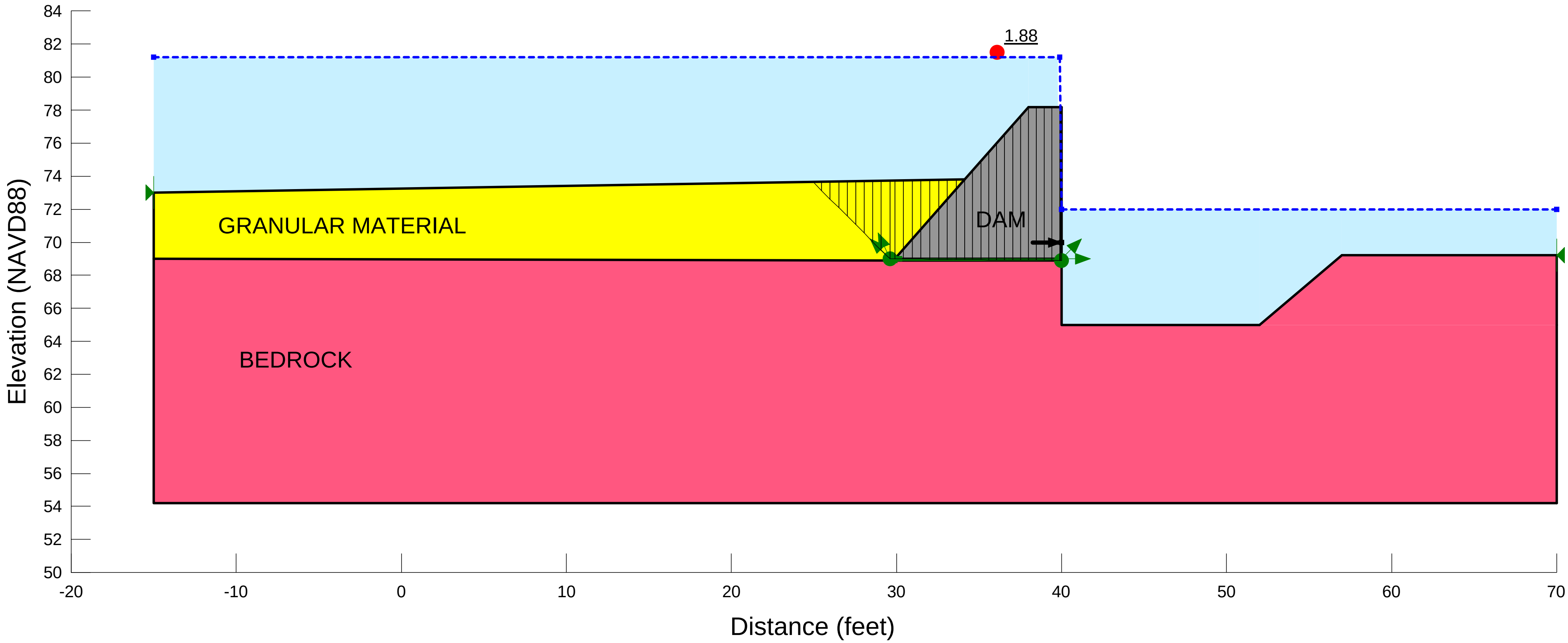
Colburn Street Dam
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Critical Section - Section D
Normal Pool - Slide Failure



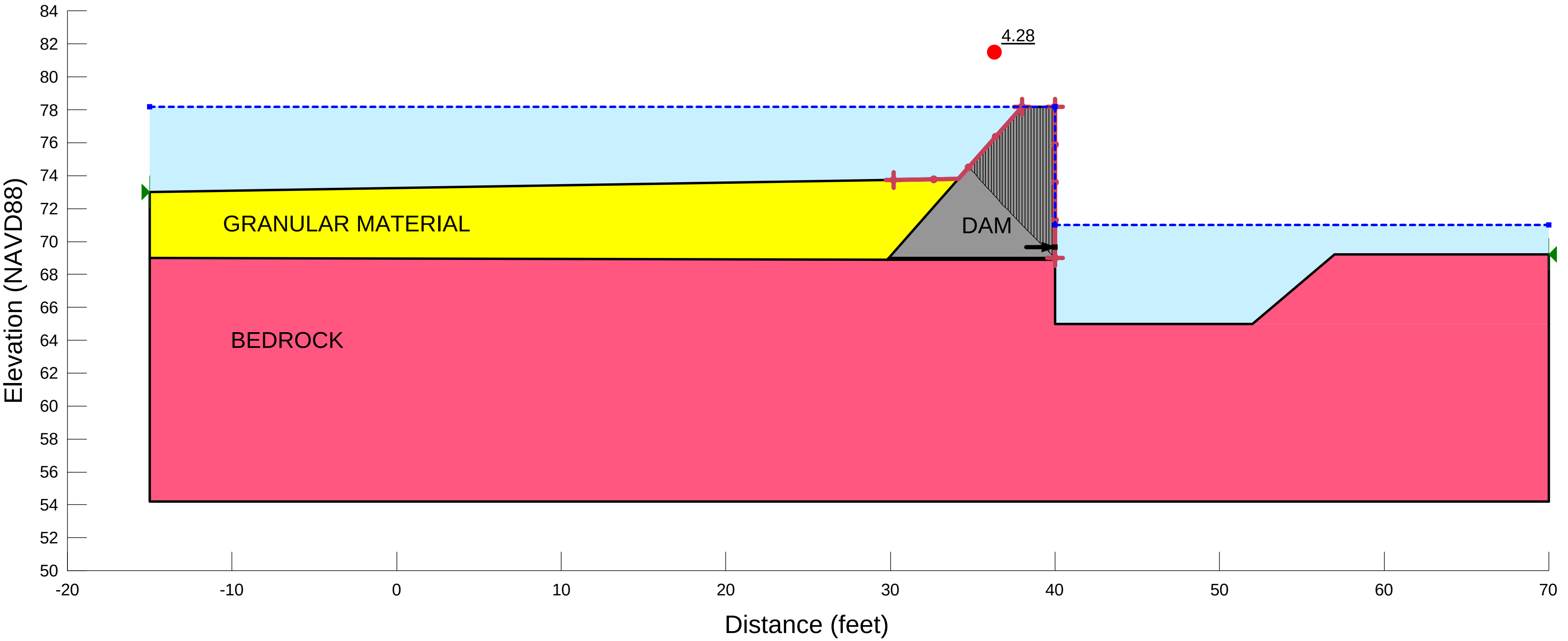
Colburn Street Dam
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Critical Section - Section D
Flood Condition - Slide Failure



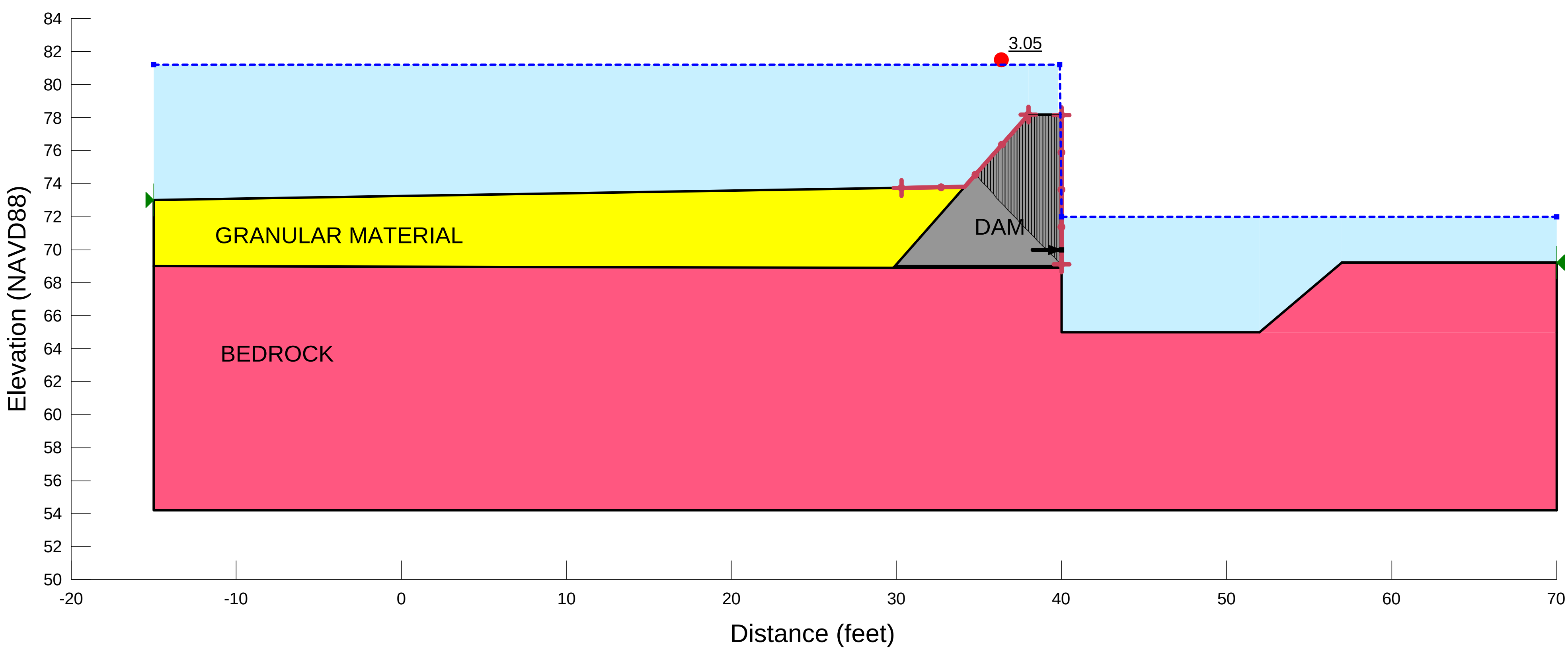
Colburn Street Dam
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Critical Section - Section D
Normal Pool - Dam Failure

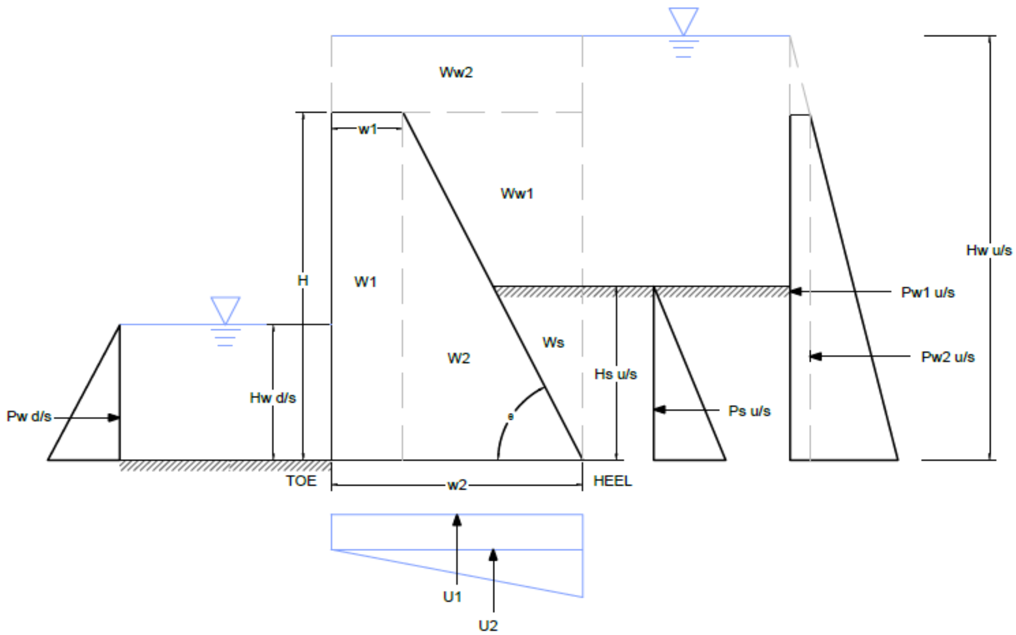


Colburn Street Dam
Dedham, Massachusetts
Dam Stability Analysis
Project No. 1510430
Method: Spencer

Critical Section - Section D
Flood Condition - Dam Failure



Generic Dam Geometry
As Built Geometery Unknown



Section B - Normal Pool Condition

Legend
Input Value
Calculated Value
Value from Slope/W

DAM GEOMETERY

Height, H (ft):	9.2
Angle of U/S Face, θ, (deg):	46
Angle of U/S Face, θ, (rad):	0.803
Bottom width, w2 (ft):	10.884
Top Width, w1 (ft):	2.000

SOIL GEOMETERY/PROPERTIES

Height of backfill, $Hs_{u/s}$ (ft):	6
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Active Earth Pressure Coeff, Ka:	0.307
Unit weight, ϒ (pcf):	125
Height of soil at toe, $Hs_{d/s}$ (ft):	0
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Passive Earth Pressure Coeff, Kp:	3.255
Unit weight, ϒ (pcf):	125
Foundation Material:	Bedrock
Friction angle, ϕ (deg):	55
Friction angle, ϕ (rad):	0.960

WATER CONDITIONS

Height of water U/S, $Hw_{U/S}$ (ft):	9.2
Height of water D/S, $Hw_{D/S}$ (ft):	2

GEI Consultants

Client: Town of Dedham
Project: Colburn Street Dam
Project No.: 1510430
Section B - Normal Pool Condition

Prepared By: A. Gradeski
Date: 12/30/2015
Checked By: J.LeFebvre
Date: 1/13/2016

Input Value
Calculated Value
Value from Slope/W
Hand Calculated

Force	Forces (lbs)				Moment Arm at Toe (ft)	Moment (ft-lbs)	
	Vertical		Horizontal				
	↓ +	↑ -	← +	→ -		↺ -	↻ +
W1	2760				1.000		2760.000
W2	6130.1923				4.961		30414.616
Ws	2172.7997				8.884		19303.885
Ww1	2550.16				8.884		22655.622
Ww2	0				5.442		0.000
Pp				0	0.000		0.000
Pw d/s				88	0.667		58.696
Pa			2352.2		3.067	7213.413	
U1		1358.365			5.442	7392.452	
U2		2445.057			7.256	17741.885	
	13,613	3,803	2,352	88		32,348	75,193
	ΣV =	**9,810**	**ΣH =**	**2,264**		**Total M(↻)**	**42,845**

NOTE: Calculation assumed water is overtopping.
Upsteam hydrostatic forces need adjustment is water level is below dam crest
Moment Arm for Pa unknown; assume acts at 1/3 Hw

Overturning

The Σ Vertical Forces		9,810	lbs
The Σ Moments (↺)		42,845	lb-ft
Resultant, R =	$\frac{\sum M}{\sum F_v}$	4.367609	ft
Resultant Location in Base:		Resultant INSIDE Central Third of Base	

OK!

Buoyancy

The Sum of Vertical Forces Down (ΣF_v↓)	13,613	lbs
The Sum of Vertical Forces Up (ΣF_v↑)	3,803	lbs
Factor of Safety to Buoyancy/Uplift	3.6	

OK!

Generic Dam Geometry
As Built Geometery Unknown
Flood Condition - 3 feet overtopping
Section B

Input Value
Calculated Value
Value from Slope/W

DAM GEOMETERY

Height, H (ft):	9.2
Angle of U/S Face, θ, (deg):	46
Angle of U/S Face, θ, (rad):	0.803
Bottom width, w2 (ft):	10.884
Top Width, w1 (ft):	2.000

SOIL GEOMETERY/PROPERTIES

Height of backfill, $Hs_{u/s}$ (ft):	6
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Active Earth Pressure Coeff, Ka:	0.307
Unit weight, ϒ (pcf):	125
Height of soil at toe, $Hs_{d/s}$ (ft):	0
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Passive Earth Pressure Coeff, Kp:	3.255
Unit weight, ϒ (pcf):	125
Foundation Material:	Bedrock
Friction angle, ϕ (deg):	55
Friction angle, ϕ (rad):	0.960

WATER CONDITIONS

Height of water U/S, $Hw_{U/S}$ (ft):	12.2
Height of water D/S, $Hw_{D/S}$ (ft):	3.2

GEI Consultants

Client: Town of Dedham
Project: Colburn Street Dam
Project No.: 1510430
Section B - Flood Condition

Prepared By: A. Gradeski
Date: 12/30/2015
Checked By: J.LeFebvre
Date: 1/13/2016

Input Value
Calculated Value
Value from Slope/W
Hand Calculated

Force	Forces (lbs)				Moment Arm at Toe (ft)	Moment (ft-lbs)	
	Vertical		Horizontal				
	↓ +	↑ -	← +	→ -		↺ -	↻ +
W1	2760				1.000		2760.000
W2	6130.1923				4.961		30414.616
Ws	2172.7997				8.884		19303.885
Ww1	2550.16				8.884		22655.622
Ww2	2037.5478				5.442		11088.678
Pp				0	0.000		0.000
Pw d/s				188	1.000		188.000
Pa			3004.3		4.067	12217.487	
U1		2173.384			5.442	11827.924	
U2		3056.322			7.256	22177.357	
	15,651	5,230	3,004	188		46,223	86,411
	ΣV =	**10,421**	**ΣH =**	**2,816**		**Total M(↻)**	**40,188**

NOTE: Calculation assumed water is overtopping.
Upsteam hydrostatic forces need adjustment is water level is below dam crest
Moment Arm for Pa unknown; assume acts at 1/3 Hw

Overturning

The Σ Vertical Forces	10,421	lbs
The Σ Moments (↺)	40,188	lb-ft
Resultant, R = $\frac{\sum M}{\sum F_v}$	3.856449	ft

Resultant Location in Base: Resultant INSIDE Central Half of Base

OK!

Buoyancy

The Sum of Vertical Forces Down ($\Sigma F_v \downarrow$)	15,651	lbs
The Sum of Vertical Forces Up ($\Sigma F_v \uparrow$)	5,230	lbs
Factor of Safety to Buoyancy/Uplift	3.0	

OK!

Generic Dam Geometry
As Built Geometery Unknown
Normal Pool Condition
Section D

Client:	Town of Dedham	Prepared By:	A. Gradeski
Project:	Colburn Street Dam	Date:	12/30/2015
Project No.:	1510430	Checked By:	J.LeFebvre
	Section D - Normal Pool Condition	Date:	1/13/2016

Input Value
Calculated Value
Value from Slope/W

DAM GEOMETERY

Height, H (ft):	9.2
Angle of U/S Face, θ, (deg):	49
Angle of U/S Face, θ, (rad):	0.855
Bottom width, w2 (ft):	9.997
Top Width, w1 (ft):	2.000

SOIL GEOMETERY/PROPERTIES

Height of backfill, $Hs_{u/s}$ (ft):	4.75
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Active Earth Pressure Coeff, Ka:	0.307
Unit weight, ϒ (pcf):	125
Height of soil at toe, $Hs_{d/s}$ (ft):	0
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Passive Earth Pressure Coeff, Kp:	3.255
Unit weight, ϒ (pcf):	125
Foundation Material:	Bedrock
Friction angle, ϕ (deg):	55
Friction angle, ϕ (rad):	0.960

WATER CONDITIONS

Height of water U/S, $Hw_{U/S}$ (ft):	9.2
Height of water D/S, $Hw_{D/S}$ (ft):	2

GEI Consultants

Client: Town of Dedham	Prepared By: A. Gradeski
Project: Colburn Street Dam	Date: 12/30/2015
Project No.: 1510430	Checked By: J.LeFebvre
Section D - Normal Pool Condition	Date: 1/13/2016

Input Value
Calculated Value
Value from Slope/W
Hand Calculated

Force	Forces (lbs)				Moment Arm at Toe (ft)	Moment (ft-lbs)	
	Vertical		Horizontal				
	↓ +	↑ -	← +	→ -		↺ -	↻ +
W1	2760				1.000		2760.000
W2	5518.2322				4.666		25747.038
Ws	1225.8301				8.664		10620.721
Ww1	2295.5846				8.664		19888.945
Ww2	0				0.000		0.000
Pp				0	0.000		0.000
Pw d/s				87.5	0.667		58.363
Pa			1885.4		3.067	5781.893	
U1		1247.68			4.999	6236.803	
U2		2245.824			6.665	14968.327	
	11,800	3,494	1,885	88		26,987	59,075
	ΣV =	**8,306**	**ΣH =**	**1,798**		**Total M(↻)**	**32,088**

NOTE: Calculation assumed water is overtopping.
Upsteam hydrostatic forces need adjustment is water level is below dam crest
Moment Arm for Pa unknown; assume acts at 1/3 Hw

Client:	Town of Dedham	Prepared By:	A. Gradeski
Project:	Colburn Street Dam	Date:	12/30/2015
Project No.:	1510430	Checked By:	J.LeFebvre
	Section D - Normal Pool Condition	Date:	1/13/2016

Overturning

The Σ Vertical Forces	8,306	lbs
The Σ Moments (↺)	32,088	lb-ft
Resultant, R = $\frac{\sum M}{\sum F_v}$	3.86317	ft

Resultant Location in Base: Resultant INSIDE Central Third of Base

OK!

Buoyancy

The Sum of Vertical Forces Down ($\Sigma F_v \downarrow$)	11,800	lbs
The Sum of Vertical Forces Up ($\Sigma F_v \uparrow$)	3,494	lbs
Factor of Safety to Buoyancy/Uplift	3.4	

OK!

Client:	Town of Dedham	Prepared By:	A. Gradeski
Project:	Colburn Street Dam	Date:	12/30/2015
Project No.:	1510430	Checked By:	
		Date:	

Generic Dam Geometry
As Built Geometery Unknown
Flood Condition - 3 feet overtopping
Section D

Input Value	
Calculated Value	
Value from Slope/W	

DAM GEOMETERY

Height, H (ft):	9.2
Angle of U/S Face, θ, (deg):	49
Angle of U/S Face, θ, (rad):	0.855
Bottom width, w2 (ft):	9.997
Top Width, w1 (ft):	2.000

SOIL GEOMETERY/PROPERTIES

Height of backfill, $Hs_{u/s}$ (ft):	4.75
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Active Earth Pressure Coeff, Ka:	0.307
Unit weight, ϒ (pcf):	125
Height of soil at toe, $Hs_{d/s}$ (ft):	0
Material Type:	Sand/Gravel
Friction angle, ϕ (deg):	32
Friction angle, ϕ (rad):	0.559
Passive Earth Pressure Coeff, Kp:	3.255
Unit weight, ϒ (pcf):	125
Foundation Material:	Bedrock
Friction angle, ϕ (deg):	55
Friction angle, ϕ (rad):	0.960

WATER CONDITIONS

Height of water U/S, $Hw_{U/S}$ (ft):	12.2
Height of water D/S, $Hw_{D/S}$ (ft):	3.2

GEI Consultants	Client: Town of Dedham	Prepared By: A. Gradeski
	Project: Colburn Street Dam	Date: 12/30/2015
	Project No.: 1510430	Checked By: J.LeFebvre
	Section D - Flood Condition	Date: 1/13/2016

Input Value
Calculated Value
Value from Slope/W
Hand Calculated

Force	Forces (lbs)				Moment Arm at Toe (ft)	Moment (ft-lbs)	
	Vertical		Horizontal				
	↓ +	↑ -	← +	→ -		↺ -	↻ +
W1	2760				1.000		2760.000
W2	5518.2322				4.666		25747.038
Ws	1225.8301				8.664		10620.721
Ww1	2295.5846				8.664		19888.945
Ww2	1871.5204				4.999		9355.205
Pp				0	0.000		0.000
Pw d/s				188	1.000		188.000
Pa			2423.5		4.067	9855.567	
U1		1996.288			4.999	9978.885	
U2		2807.281			6.665	18710.409	
	13,671	4,804	2,424	188		38,545	68,560
	ΣV =	**8,868**	**ΣH =**	**2,236**		**Total M(↻)**	**30,015**

NOTE: Calculation assumed water is overtopping.
Upsteam hydrostatic forces need adjustment is water level is below dam crest
Moment Arm for Pa unknown; assume acts at 1/3 Hw

Overturning

The Σ Vertical Forces	8,868	lbs
The Σ Moments (↺)	30,015	lb-ft
Resultant, R = $\frac{\sum M}{\sum F_v}$	3.3848	ft

Resultant Location in Base: Resultant INSIDE Central Half of Base

OK!

Buoyancy

The Sum of Vertical Forces Down ($\Sigma F_v \downarrow$)	13,671	lbs
The Sum of Vertical Forces Up ($\Sigma F_v \uparrow$)	4,804	lbs
Factor of Safety to Buoyancy/Uplift	2.8	

OK!

Attachment E: Conceptual Design Engineer’s Construction Cost Estimate

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CONCEPTUAL DESIGN - ENGINEER'S CONSTRUCTION COST ESTIMATE

PROJECT NAME:	**Colburn Street Dam Improvement Project**	DATE:	**02/17/16**
CLIENT:	**Department of Public Works**	PROJECT NO.:	**50075664**
MUNICIPALITY:	**Town of Dedham, Massachusetts**	CHECKED BY:	**PB**
COUNTY:	**Norfolk County**	PREPARED BY:	**MK**

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL AMOUNT
1	General Conditions - 10%	LS	1	$ 40,000.00	$ 40,000.00
2	Erosion and Sediment Control	LS	1	$ 15,000.00	$ 15,000.00
	Site Work				
3	Site Clearing and Grubbing	LS	1	$ 15,000.00	$ 25,000.00
4	Portadam Cofferdam (9' high frames, liner, hardware)	LF	110	$ 400.00	$ 44,000.00
5	Dewatering / Pumping	LS	1	$ 50,000.00	$ 50,000.00
6	Excavation in Cofferdam and Disposal	BCY	90	$ 70.00	$ 6,300.00
7	Clean Concrete Surface	SF	650	$ 10.00	$ 6,500.00
8	Concrete Surface Repair	SF	160	$ 15.00	$ 2,400.00
9	Shotcrete (Wet Mix) 4" Thick	SF	650	$ 40.00	$ 26,000.00
10	Rip Rap (D_{50} = 18")	CY	250	$ 275.00	$ 68,750.00
11	Saw Cut Concrete Walls (6" deep) at Spillway for Stop Log Installation	LF	12.0	$ 65.00	$ 780.00
12	Grout Fill after Slot Installation for Stop Logs	CF	3	$ 400.00	$ 1,200.00
13	Dowels for Grout for Stop Logs (6" long, 3/4" diameter)	EA	8	$ 50.00	$ 400.00
14	Aluminum Stop Logs with Lifting Device and Slot Installation	LS	1.0	$ 45,000.00	$ 45,000.00
15	Grout Voids in Dam	CF	1,000	$ 75.00	$ 75,000.00
16	Pack all Joints with Mortar and Surface Point	SF	900	$ 20.00	$ 18,000.00
17	Chain Link Fence (6' high)	LF	50	$ 20.00	$ 1,000.00
	SUB-TOTAL FOR MATERIAL AND LABOR				**$ 425,330.00**
	OVERHEAD - 0% OF LABOR & MATERIAL (OVERHEAD INCLUDED IN ABOVE UNIT PRICES)				**$ -**
	SUB-TOTAL:				**$ 425,330.00**
	PROFIT - 0% (PROFIT INCLUDED IN ABOVE UNIT PRICES)				**$ -**
	TOTAL ESTIMATED COST WITH OVERHEAD & PROFIT				**$ 425,330.00**
	DESIGN CONTINGNECY - 25%				**$ 106,332.50**
	CONSTRUCTION ALLOWANCE - 5%				**$ 21,266.50**
	BUDGET CONSTRUCTION COST				**$ 552,929.00**
			Rounded Total Construction Cost		**$ 560,000.00**