Addendum No. 1

August 22, 2025

Re: Dells Pond Outlet Dam Improvements Project
Town of Littleton

From: DuBois & King, Inc.
Charles Johnston, P.E.
6 Green Tree Drive
South Burlington, Vermont 05403
(802) 878-7661

To: Prospective Bidders

This Addendum forms part of the Contract Documents and provides additional information that may modify the original Bidding Documents issued for the Dells Pond Outlet Dam Improvements Project dated July 30, 2025. Acknowledge receipt of this Addendum in the space provided on Page 3 of the Bid Form. Failure to do so will subject the Bidder to possible disqualification.

I. Contract Document (Bid Document) Changes

Bid Form for Construction Contract

See attached updated Bid Form for changes. Changes summarized below:

- Added Bid Item #6: Concrete Demolition/Removal of Structures
- Updated numbering for Bid Items: Common Excavation, Riprap Class III, and Pond Drain System
- Added Bid Item 10: New Stone Block Wall Reconstruction
- Added Bid Item 11: Existing Stone Block Wall Reconstruction
- Updated numbering for Bid Items: Embankment Material, Pedestrian Bridge, Parking Lot Improvements, and Restoration of Surfaces.
- Article 6 – Time of Completion was modified for the Bidder to provide an expected schedule to complete construction.

UTILIZE ATTACHED BID FORM.

Section 01150 – Measurement and Payment

See attached updated Measurement and Payment section for changes. Changes summarized below:

- Bid Item 10: New Stone Block Wall Reconstruction
    - Removal of anchor coring, and reinforcing steel text

II. Questions & Answers

The following questions are from the Pre-Bid Meeting and questions received following the Pre-Bid Meeting.

Question 1: The Advertisement of Bid outlines the project has an expected duration of 90 days. Is this a requirement? There are conflicting lead times for certain portions of work, for example the bridge.

The Owner acknowledged that there are time constraints to finishing the project this year. The Bidder is directed to provide the best bid price and associated schedule to complete the work. Bids will be evaluated as outlined in the Bid Documents including but not limited to Bid Price, Qualifications, and Proposed Schedule. The Bid Form Article 6 was modified to provide room for the Bidder to outline expected schedule to be complete.

An example of a schedule is provided below:

Work behind the cofferdam/Work to be completed in dry to be completed by MON XX, YEAR. This includes the following items:

- Clearing and Grubbing
- Concrete Demolition/Removal of Structures
- Pond Drain System
- New Stone Block Wall Reconstruction
- Existing Stone Block Wall Reconstruction
- Common Excavation
- Riprap, Class III
- Embankment Material

Pedestrian Bridge work to be completed by MON XX, YEAR.

Parking Lot Improvements to be completed by MON XX, YEAR.

Substantial Completion to be achieved by MON XX, YEAR.

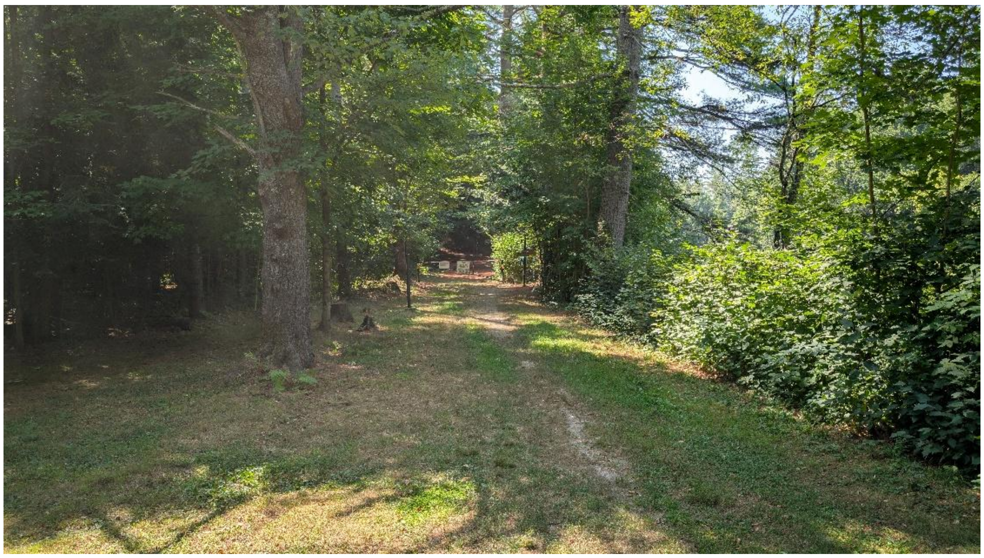
Final Completion to be achieved by MON XX, YEAR.

Question 2: Are there federal requirements for this project? Davis Bacon Wages, Buy American?

The project is locally funded by the Town of Littleton. No federal funds are currently being used, and the Owner is not aware of any federal requirements. As noted during the pre-bid meeting, if the bids exceed the Town's ability to fund the project, the Town may need to apply for funding via federal grant programs. If this is required, the project may be separated to prevent mixing of funds for portions of the project such as the steel pedestrian bridge.

Question 3: **How is the site accessed, and can the parking lots be closed for construction?**

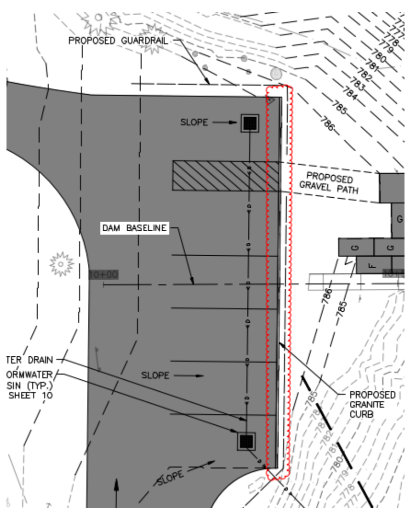
The site can be accessed in two ways. The primary access is from Dells Road on the left abutment of the dam (looking downstream). The secondary access is from NH-18 into the Dells Pond Park parking area and via a trail. There are gates that restrict vehicular access. The park will be closed for construction; however, the Town is looking to open the park as soon as possible, which is why a schedule is being requested.



Access from NH-18 through parking lot

Question 4: **Does the granite curbing go around the entire perimeter or just in front of the parking?**

The granite curbing is only intended along the front of the parking to prevent stormwater from flowing on dam crest and down slopes. See Plan Sheet 7.



Question 5: **Are there more specific specs for the storm drain system, inverts, pipe size, etc.?**

- Outlined in Section 3 on Plan Sheet 10, the storm drainpipe shall be a 6-inch diameter Schedule 40 PVC pipe.
- Outlined in Section 1 on Plan Sheet 10, the catch basins shall be a precast reinforced concrete riser with a steel grate and frame. All joints shall be watertight.
- The outlet invert is marked on Plan Sheet 7 to be Elevation 777.0-ft.
- The pipe slope is called out on Section 2 Plan Sheet 10 to be 1%.
- Section 2 Plan Sheet 10 inadvertently details the catch basin rims to be elevation 777.25-ft. This should read El. 787.25-ft. The pipe invert in the upstream catch basin is expected to be approximately Elevation 778.0-ft.

Question 6: **Confirm the size of the granite blocks?**

The new granite blocks are intended to be 6-foot long, 2-feet tall, and 2-feet wide. The intent is to reuse existing stone blocks on site where the existing wall is being modified, however, additional stone blocks will be necessary to form the new channel. All stone blocks will be mortared in place as outlined on Plan Sheet 9.

Question 7: **Will there be a technical specification available for the pedestrian bridge? Are there digital files for the design of the pedestrian bridge? Is there more information that is available for a fabricator to use to construct the bridge?**

Information related to the bridge is outlined on Plan Sheet 9. This is a delegate design item to the Contractor. The intent is that bridge will have the basic requirements outlined on Plan Sheet 9, however the bridge will be left to the Contractor to design. A submittal of the design from the Contractor will be provided to the Owner and Engineer prior to fabrication.

Question 8: **What is the required completion time for this project?**

The Owner is looking to complete this project as soon as possible. As outlined on the Permitting Disclaimer within the Bid Documents, there is a possible conflict with when the permits will be available and the US Army Corps of Engineers General Permit 'Time-of-Year Work Window'. As outlined in the pre-bid meeting, we encourage bidders to provide their best bid with a selected construction timeframe. The Owner acknowledges that due to the time constraint, construction may need to be delayed to the spring of 2026. Refer to Question 1.

Contractor shall review and be familiar with the NH Army Corps of Engineers General Permit, specifically Section IV. General Permit Condition 20.

https://www.nae.usace.army.mil/Portals/74/docs/regulatory/StateGeneralPermits/NH/NHGPs.pdf

Question 9: **AASHTO's guidance for pedestrian bridges only requires an H-5 vehicle for clear widths between 7 and 10 ft., please confirm that the pedestrian bridge needs to be designed based on an H-10 vehicle as stated in the notes on sheet 9.**

The bridge must have a H-10 loading capacity.

Question 10: **On sheet 9, there is indication that the pedestrian bridge is to generally match the appearance and type of the existing pedestrian bridge in Littleton which crosses over the Ammonoosuc River. The existing bridge is a bow string truss with a top cord cambered to meet the bottom cord on each end, the plans show a parallel cord truss, the existing bridge has vertical pickets, the plans are shoring horizontal safety rails, existing bridge has a wood deck, plans call for a concrete deck. Please clarify if the bridge should be as drawn on the plans or match the existing structure.**

The bridge is to be designed and supplied by the Contractor. The Owner has requested that the aesthetic of the bridge be similar to the bridge that crosses the Ammonoosuc River at Bridge Street in Littleton. The design of the bridge is not intended to match this bridge but to have a similar style, i.e. weather steel finish, guardrail, rub bar, etc. The graphics and depictions of the bridge on Plan Sheet 9 are to demonstrate the overall size and scope and are not intended to be the design of the bridge.

Question 11: **Will information be provided to size a bypass system during construction?**

The selected Contractor will receive the watershed modeling by DuBois & King that was created as part of the design for this project. Below are peak flow results for various storm events.

The watershed primarily flows from the north passing through a 60-inch diameter culvert into Dell's Pond Outlet Dam. An USGS StreamStats report has been included for this watershed. This provides the following seasonal flow information:

- January to March 15
  - 98% Flow – 0.49 cfs
  - 90% Flow – 0.73 cfs
  - 80% Flow – 0.97 cfs
- March 16 to May
  - 98% Flow – 1.2 cfs
  - 90% Flow – 2.6 cfs
  - 80% Flow – 3.8 cfs
- June to October
  - 98% Flow – 0.10 cfs
  - 90% Flow – 0.16 cfs
  - 80% Flow – 0.26 cfs

- November to December
    - 98% Flow – 0.50 cfs
    - 90% Flow – 1.16 cfs
    - 80% Flow – 1.73 cfs

In addition the watershed to the north, and smaller stream exist to the east. This is primarily controlled by the pond within the cemetery and the drainage system at the intersection of NH-18 and Farr Hill Road, which is reported to have a 15-inch diameter outlet pipe.

Recommend the bypass system at a minimum to continue to provide seasonal flows to downstream channel.

Question 12: There are borings and test pits shown on the plans. Can this information please be provided? We will need this information to design the concrete abutments.

The geotechnical report by Ward Geotechnical Consulting has been attached.

Question 13: How is excavation and backfill of the bridge abutments paid for?

Excavation of the earthen dam will be paid under Bid Item "Common Excavation" and backfill shall be paid under Bid Item "Embankment Material". If the design of the abutments, which is by the Contractor, requires modified subgrade material, this will be paid for under Bid Item "Pedestrian Bridge and Abutments".

Question 14: At the pre-bid meeting it was mentioned that the pedestrian bridge required a H-10 load rating and that it would be used for some light duty vehicles as needed. The notes call for it to be 8ft wide (the plans actually scale to only 5ft wide). Is this correct? That seems somewhat narrow if the Town wishes to drive over this bridge with light duty vehicles and ambulances.

As outlined on Plan Sheet 9, the bridge shall be 8-feet wide.

Question 15: The measurement and payment specifications include an item "Concrete Demolition/Removal of Structures" which does not appear in the bid tab. Was this missed? If not, where should the costs for demolition be included?

The bid tab has been modified to include this item.

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Question 16: **The measurement and payment specifications include items "New Stone Block Wall Reconstruction" and "Existing Stone Block Wall Reconstruction". We do not have either item in the bid tab but instead have a "Stone Block Wall Reconstruction". Is this item intended to capture the costs for the new block spillway and the cleaning/re-pointing of the existing block wall?**

The bid tab has been modified to include 'New Stone Block Wall Reconstruction' and 'Existing Stone Block Wall Reconstruction'. The intent is that portion of the spillway that is being modified by reusing of the existing stone blocks and augmented with additional granite blocks will be paid for under Bid Item 'New Stone Block Wall Reconstruction'. The cleaning and repointing of joints on the existing stone block wall will be paid for under Bid Item 'Existing Stone Block Wall Reconstruction'.

Question 17: **Note 1 of Detail D1 on sheet 9 says to apply a ¼" layer of cement grout between all joints of the granite blocks within the spillway section. ¼" wide? ¼" deep? This seems to be an odd dimension since the blocks may have gaps larger than ¼" depending on how they fit together. Perhaps this note intended to state, ¼" min layer of grout? Please clarify.**

This note is intended to state ¼-inch minimum layer of grout between stone blocks.

Question 18: **Note 2 of Detail D1 on sheet 9 says to pressure wash and inspect the downstream wall and repoint all joints with cement grout. Does this apply to the entire structure (ie all walls, spillway, etc)? What about the wall areas that will be buried in the new rip rap? This will be an expensive item to "repoint ALL joints". Perhaps consider a unit price item for repoint of stone masonry. Which pay item is this work to be included in?**

The repointing of the existing walls is intended to only be in location the existing concrete mortar is missing. The pressure wash is to remove vegetation (moss) and other debris from the face of the wall to confirm locations that require additional mortar.

Question 19: **Are we expected to chip out/remove any existing grout between existing stone masonry? Or are we only re-pointing the areas missing grout? Or re-pointing over existing grout as well?**

The repointing is only in locations missing grout. There is no need to repoint the wall that is below the riprap on the downstream toe.

Question 20: **Does the rip rap class 3 item include the cost of the concrete grout in the rip rap? If not, where is this included? What strength or type concrete/grout is intended for use on this application?**

The concrete to be applied to the riprap 5-feet downstream of the wall is intended to be a low strength flowable fill concrete mix. NHDOT Class F is acceptable for this item.

Question 21: **The measurement and payment section for the pond drain system refers to the "pressure pipe system" and "commissioning of the system to the Town". What is this in reference to? Sheet 11 of the plans refers to pressure testing of the drain pipe, which will not be physically possible given the test would need to include the existing section of pipe under the dam spillway. This pipe is an open ended drain pipe and not a pressure pipe. Not only is pressure testing not possible (and could risk the integrity of the existing pipe and dam), it is also unnecessary given its application. Please clarify.**

A leakage test will be required for the valve-vault to verify the installation of the gate valve and the link-seals. This will involve a plug in the 12-inch D.I. pipe upstream of the vault. No testing of the drainpipe upstream or downstream of the vault will be required. Commissioning involves a brief meeting with the Owner to demonstrate operation of the pond drain system, providing manufacturer's documentation for system, and handing over operating equipment to the Town.

III. Pre-Bid Conference Attendance List

Name	Number	Email
Charles Johnston – D&K	802-989-4402	cjohnston@dubois-king.com
Eric Oliver – Town of Littleton	603-575-9170	eoliver@townoflittleton.org
Cooper Gordon	603-325-8441	gordonservices@gmail.com
Cody Marsh	802-557-8829	cmarsh@ecivt.com
Barry Sleath	802-291-3921	bsleath@neilhdaniels.com
Clemente Varas	802-224-6176	cvaras@kingsburyco.com
Jared Urban	508-981-9413	jurban@rainforrent.com
Austin Hunt	802-730-3278	ahunt@gwtatro.com
Matthias Schram	802-793-2510	matthias@morganexcavationllc.com
Joe Abesamra	978-871-1272	jabesamra@tford.com
Nick Guckin	802-473-6093	nguckin@jpsicard.com
Mike Lynch	603-331-1832	lynch4ns@earthlink.net
Lucas Perez-Segnini	914-319-7973	Lperez-segnini@neinfrastructure.com
Keith Kane	603-752-1370	keith@raysnh.com
Sam Jeffers	603-331-1161	samuelclarkjeffers@hotmail.com

A list of plan holders that acquired plans from DuBois & King is located here: dubois-king.com/projects-bidding-active/

This document constitutes Addendum 1 for this Project.

BID FORM FOR CONSTRUCTION CONTRACT

The terms used in this Bid with initial capital letters have the meanings stated in the Instructions to Bidders, the General Conditions, and the Supplementary Conditions.

ARTICLE 1—OWNER AND BIDDER

1.01 This Bid is submitted to: **Town of Littleton, New Hampshire**

1.02 The undersigned Bidder proposes and agrees, if this Bid is accepted, to enter into an Agreement with Owner in the form included in the Bidding Documents to perform all Work as specified or indicated in the Bidding Documents for the prices and within the times indicated in this Bid and in accordance with the other terms and conditions of the Bidding Documents.

ARTICLE 2—ATTACHMENTS TO THIS BID

2.01 The following documents are submitted with and made a condition of this Bid:

A. Required Bid security;
B. List of Proposed Subcontractors;
C. List of Proposed Suppliers;
D. Evidence of authority to do business in the state of the Project; or a written covenant to obtain such authority within the time for acceptance of Bids;
E. Contractor's license number as evidence of Bidder's State Contractor's License or a covenant by Bidder to obtain said license within the time for acceptance of Bids;
F. Required Bidder Qualification Statement with supporting data; and

ARTICLE 3—BASIS OF BID—LUMP SUM BID AND UNIT PRICES

3.01 *Lump Sum Bids*

A. Bidder will complete the Work in accordance with the Contract Documents for the following lump sum (stipulated) price(s), together with any Unit Prices indicated in Paragraph 3.02:

1. Lump Sum Price (Single Lump Sum)

Item No.	Description	Bid Amount *Numerals*	*Words*
1	Mobilization/Demobilization	$	Dollars and Cents
2	Control of Water	$	Dollars and Cents
3	Traffic Control	$	Dollars and Cents
4	Erosion Prevention and Sediment Control	$	Dollars and Cents
5	Clearing and Grubbing	$	Dollars and Cents
6	Concrete Demolition/Removal of Structures	$	Dollars and Cents
9	Pond Drain System	$	Dollars and Cents
10	New Stone Block Wall Reconstruction	$	Dollars and Cents
11	Existing Stone Block Wall Reconstruction	$	Dollars and Cents
13	Pedestrian Bridge and Abutments	$	Dollars and Cents
14	Parking Lot Improvements	$	Dollars and Cents
15	Restoration of Surfaces	$	Dollars and Cents

3.02 *Unit Price Bids*

A. Bidder will perform the following Work at the indicated unit prices:

Item No.	Description	Estimated Quantity	Bid Unit Price	Bid Amount	
				Numerals	*Words*
7	Common Excavation	500 CY	$	$	Dollars and Cents
8	Riprap, Class III	320 TON	$	$	Dollars and Cents
12	Embankment Material	50 CY	$	$	Dollars and Cents

B. Bidder acknowledges that:

1. each Bid Unit Price includes an amount considered by Bidder to be adequate to cover Contractor's overhead and profit for each separately identified item, and

2. Estimated quantities are not guaranteed, and are solely for the purpose of comparison of Bids, and final payment for all Unit Price Work will be based on actual quantities, determined as provided in the Contract Documents.

3.03 *Total Bid Price (Lump Sum and Unit Prices)*

Description	Bid Amount	
	Numerals	*Words*
Total Bid Price (Total of all Lump Sum and Unit Price Bids)	$	Dollars and Cents

ARTICLE 4—BASIS OF BID—COST-PLUS FEE –LEFT BLANK INTENTIONALLY

4.01 NOT USED

ARTICLE 5—PRICE-PLUS-TIME BID –LEFT BLANK INTENTIONALLY

5.01 NOT USED

ARTICLE 6—TIME OF COMPLETION

6.01 Bidder agrees that the Work will be substantially complete and will be completed and ready for final payment in accordance with Paragraph 15.06 of the General Conditions on or before the dates or within the number of calendar days indicated in the Agreement.

6.02 The Bidder will provide an expected schedule to complete construction:

____
____
____
____
____
____
____
____

ARTICLE 7—BIDDER'S ACKNOWLEDGEMENTS: ACCEPTANCE PERIOD, INSTRUCTIONS, AND RECEIPT OF ADDENDA

7.01 *Bid Acceptance Period*

A. This Bid will remain subject to acceptance for 60 days after the Bid opening, or for such longer period of time that Bidder may agree to in writing upon request of Owner.

7.02 *Instructions to Bidders*

A. Bidder accepts all of the terms and conditions of the Instructions to Bidders, including without limitation those dealing with the disposition of Bid security.

7.03 *Receipt of Addenda*

A. Bidder hereby acknowledges receipt of the following Addenda:

Addendum Number	Addendum Date

ARTICLE 8—BIDDER'S REPRESENTATIONS AND CERTIFICATIONS

8.01 *Bidder's Representations*

A. In submitting this Bid, Bidder represents the following:

1. Bidder has examined and carefully studied the Bidding Documents, including Addenda.
2. Bidder has visited the Site, conducted a thorough visual examination of the Site and adjacent areas, and become familiar with the general, local, and Site conditions that may affect cost, progress, and performance of the Work.
3. Bidder is familiar with all Laws and Regulations that may affect cost, progress, and performance of the Work.
4. Bidder has carefully studied the reports of explorations and tests of subsurface conditions at or adjacent to the Site and the drawings of physical conditions relating to existing surface or subsurface structures at the Site that have been identified in the Supplementary Conditions, with respect to the Technical Data in such reports and drawings.
5. Bidder has carefully studied the reports and drawings relating to Hazardous Environmental Conditions, if any, at or adjacent to the Site that have been identified in the Supplementary Conditions, with respect to Technical Data in such reports and drawings.
6. Bidder has considered the information known to Bidder itself; information commonly known to contractors doing business in the locality of the Site; information and observations obtained from visits to the Site; the Bidding Documents; and the Technical Data identified in the Supplementary Conditions or by definition, with respect to the effect of such information, observations, and Technical Data on (a) the cost, progress, and performance of the Work; (b) the means, methods, techniques, sequences, and procedures of construction to be employed by Bidder, if selected as Contractor; and (c) Bidder's (Contractor's) safety precautions and programs.

7. Based on the information and observations referred to in the preceding paragraph, Bidder agrees that no further examinations, investigations, explorations, tests, studies, or data are necessary for the performance of the Work at the Contract Price, within the Contract Times, and in accordance with the other terms and conditions of the Contract.
8. Bidder is aware of the general nature of work to be performed by Owner and others at the Site that relates to the Work as indicated in the Bidding Documents.
9. Bidder has given Engineer written notice of all conflicts, errors, ambiguities, or discrepancies that Bidder has discovered in the Bidding Documents, and of discrepancies between Site conditions and the Contract Documents, and the written resolution thereof by Engineer is acceptable to Contractor.
10. The Bidding Documents are generally sufficient to indicate and convey understanding of all terms and conditions for performance and furnishing of the Work.
11. The submission of this Bid constitutes an incontrovertible representation by Bidder that without exception the Bid and all prices in the Bid are premised upon performing and furnishing the Work required by the Bidding Documents.

8.02 *Bidder's Certifications*

A. The Bidder certifies the following:
   1. This Bid is genuine and not made in the interest of or on behalf of any undisclosed individual or entity and is not submitted in conformity with any collusive agreement or rules of any group, association, organization, or corporation.
   2. Bidder has not directly or indirectly induced or solicited any other Bidder to submit a false or sham Bid.
   3. Bidder has not solicited or induced any individual or entity to refrain from bidding.
   4. Bidder has not engaged in corrupt, fraudulent, collusive, or coercive practices in competing for the Contract. For the purposes of this Paragraph 8.02.A:
      a. Corrupt practice means the offering, giving, receiving, or soliciting of anything of value likely to influence the action of a public official in the bidding process.
      b. Fraudulent practice means an intentional misrepresentation of facts made (a) to influence the bidding process to the detriment of Owner, (b) to establish bid prices at artificial non-competitive levels, or (c) to deprive Owner of the benefits of free and open competition.
      c. Collusive practice means a scheme or arrangement between two or more Bidders, with or without the knowledge of Owner, a purpose of which is to establish bid prices at artificial, non-competitive levels.
      d. Coercive practice means harming or threatening to harm, directly or indirectly, persons or their property to influence their participation in the bidding process or affect the execution of the Contract.

BIDDER hereby submits this Bid as set forth above:

Bidder:

____

(typed or printed name of organization)

By: ____

(individual's signature)

Name: ____

(typed or printed)

Title: ____

(typed or printed)

Date: ____

(typed or printed)

If Bidder is a corporation, a partnership, or a joint venture, attach evidence of authority to sign.

Attest: ____

(individual's signature)

Name: ____

(typed or printed)

Title: ____

(typed or printed)

Date: ____

(typed or printed)

Address for giving notices:

____

____

Bidder's Contact:

Name: ____

(typed or printed)

Title: ____

(typed or printed)

Phone: ____

Email: ____

Address:

____

____

____

Bidder's Contractor License No.: (if applicable) ______________________________

SECTION 01150 - MEASUREMENTS AND PAYMENT

PART 1 - GENERAL

1.01 DESCRIPTION

A. This Section covers the requirements for measurements and records for payment purposes and describes the items under which payments will be made for all Work performed under this Contract.

B. Items not specified or identified to be measured or paid for, but required to fully construct the Work as shown on the Drawings, shall be considered subsidiary to appropriate specified items at no change in the specified item cost.

1.02 MEASUREMENT REQUIREMENTS

A. Where payments will be made for removing existing materials, notify the Engineer so that they may inspect the materials to be removed, so that they may witness the measuring, and so that they may approve the record of measurements. All materials removed without conforming to the above procedures, and which Engineer cannot verify or substantiate, will not be paid for.

B. Maintain complete, neat, clean, and legible field notes for all measured items. Notes shall contain spaces for Contractor's and Engineer's signatures plus additional space for comments. An original and copy shall be made for all notes, and one copy shall be turned over to Engineer daily. The Engineer's signature shall not be construed as an acceptance of the Work, or the measurements made, but shall mean that he was present when the measurements were made.

1.03 SUBMITTALS

A. See Section 01300.

B. Field notes of all measurements for payment purposes delivered to Engineer daily.

1.04 <u>SCHEDULING</u>

A. Notify Engineer, as far in advance as possible, of the making of measurements so that the Engineer may observe existing conditions, work being performed, and measurements being made.

B. Allow for and afford Engineer ample time, space and equipment to observe measurements and to verify measurements and elevations.

PART 2 - PRODUCTS

2.01 <u>GENERAL</u>

A. Provide all labor, materials, facilities, levels, measuring devices, and all other equipment necessary to properly and accurately perform all measurements for payment purposes.

PART 3 - EXECUTION

3.01 <u>GENERAL REQUIREMENTS & STIPULATIONS</u>

A. Perform all measuring required under this Section.

B. No separate payments will be made for Work under this Contract except for the pay items stipulated in this PART 3. All costs in connection with the Work shall be included in one or more of the pay items as appropriate, see Subsection 1.02B for additional definition.

C. Each pay item shall be full compensation for all costs in connection with the item, including but not limited to:

1. The furnishing of all materials, labor, equipment, tools, and all incidentals.
2. The installation of all materials, equipment, facilities, accessories and appurtenant items.
3. The proper share of overhead and profit.
4. All testing requirements. The Contractor shall retain an independent and qualified material testing firm to conduct all testing. The testing firm shall be submitted to the Engineer for approval. The costs for testing shall not be paid for directly, but shall be considered subsidiary to all items that require testing (ie: concrete, soils, compaction, pressure testing, etc.)

5. All dust control measures.

6. All related and incidental Work and items necessary or required to complete the Work.

7. Record and maintain record drawings throughout the Project construction period.

D. Each pay item which specifically involves excavation shall be considered to include full compensation for:

1. Backfilling with suitable excavated material in compacted lifts unless paid for under other items.

2. Excavation in earth, all excavation except for solid bedrock excavation.

3. Disposal of any surplus.

4. Installation and removal of sheeting and bracing.

3.02 MEASUREMENT & PAYMENT ITEMS

A. The names of the following items are abbreviated forms of the Bid Items as contained in the Bid Form. The names, as shown below or on the Bid Form, shall not be construed to represent a complete description of all of the Work included under such items and are provided only as a means of identification and for ease of conversation.

Item 1: MOBILIZATION/DEMOBILIZATION

Lump sum, no measurement required.

Payment shall be lump sum for work items necessary for the movement of personnel, equipment and materials to the Project Site, establishment of all field offices and related facilities, attendance at all meetings, notifications to Owner, Engineer, abutters, utility owners / providers, NHDOT, NHDES, and all other necessary items to perform the Work on the Project and for all other costs and operations which must be performed prior to the beginning of the Work. In addition, this pay item will include all costs associated with movement of personnel, equipment and materials from the Project Site by final acceptance of the Project and all cleanup work and restoration of surfaces as required herein.

The first forty percent (40%) of the lump sum shall be paid once all personnel, equipment and materials necessary to initiate excavation and foundation preparation are on site. The next 40% shall be paid in equal monthly payments based on the Contractor's approved construction schedule, the next 10% following approval of Substantial Completion, and the final 10% shall be paid following Owner's acceptance of punch list completion and cleanup.

Item 2: CONTROL OF WATER

Lump sum, no measurement required.

Payment shall be lump sum for complete control of water, including preparation and submittal of a control of water plan to the Owner and Engineer for review and acceptance, installation and subsequent removal of all temporary cofferdams, temporary diversion pipes, pumping for excavations, placement, handling and maintenance of pumps and any other miscellaneous work necessary to keep the work area dry.

The first 10 percent (10%) of the lump sum shall be paid once the submittal of a control of water plan to the Owner and Engineer has been reviewed and accepted. The next 30 percent (30%) of the lump sum shall be paid once the temporary cofferdam and appurtenances are in place and properly functioning. The next 40 percent (40%) shall be paid in equal monthly payments based on the Contractor's approved schedule to Substantial Completion, and the final 20 percent (20%) paid at the acceptance of Substantial Completion and removal of all control of water devices.

Item 3: TRAFFIC CONTROL

Lump sum, no measurement required.

Payment shall be lump sum for installation and maintenance of Contractor's traffic control plan during construction as specified in the Drawings, as required by the Town of Littleton, and any traffic control standards by the NHDOT.

The first 40 percent (40%) of the lump sum shall be paid once the traffic control measures are initially installed and the remaining 60% shall be paid at the completion of the Project and demobilization.

Item 4: EROSION PREVENTION & SEDIMENT CONTROL

Lump sum, no measurement required.

Payment shall be lump sum for the installation, maintenance and removal of erosion control and slope protection measures to prevent the discharge of turbid water and / or sediment from the Project Site as shown on the Drawings. Items included in this pay item include (not limited to): All erosion control measures identified or noted on the Drawings and in the Specifications, required by permits and the associated work to install, and maintenance and removal of the erosion prevention and sediment control measures.

Item 5: CLEARING AND GRUBBING

Lump sum, no measurement required.

Payment shall be lump sum for the removal and off-site disposal of vegetation, cutting of trees, removal of roots, stumps, rocks, muck, and other objectionable materials deleterious to the Work as shown on the Drawings and / or as directed by the Engineer.

Item 6: CONCRETE DEMOLITION / REMOVAL OF STRUCTURES

Lump sum, no measurement required.

Payment shall be lump sum for the demolition / removal and off-site disposal of the existing vehicular bridge and abutments, pond drain intake structure, downstream cut stone masonry walls, and other associated structural components required to be removed to construct the proposed improvements as shown on the Drawings and / or as directed by the Engineer.

Item 7: COMMON EXCAVATION

Measure cubic yardage of material excavated to acceptable subgrade and material hauled / disposed of from the site for the construction of the Work based on the payment limits on the Drawings. Excavation also includes the removal and disposal of pond sediment within the limits shown, removal of material to construct terraced wall sections, and removal of material for installation of riprap material. The compaction of materials and proof-rolling of subgrade under embankments and structures with approved vibratory compaction equipment shall not be paid for separately, but shall be considered subsidiary to this pay item.

All costs associated with the installation, maintenance and removal of temporary shoring and earth support work required by the Contractor to safely support the excavated slope shall not be paid for directly, but shall be considered subsidiary to this pay item.

Payment shall be per cubic yard of material excavated as specified.

Item 8: RIPRAP, CLASS III

Record and tally the actual tonnage of stone fill delivered to the site and placed, as indicated by the Drawings and directed by the Engineer.

Payment shall be per ton of stone fill material placed in the outlet channel, as determined by the tally of truck slips submitted to the Engineer.

Item 9: POND DRAIN SYSTEM

Lump sum, no measurement required.

Payment shall be lump sum for all improvements described for the Pond Drain System, as shown on the plans, including (not limited to): replacement of the pond drain pipe from plugged location to new upstream intake, new concrete valve vault, and ductile iron piping and valves.

Work under this item shall also include any necessary excavation to expose the existing pond drain from the plugged location upstream to the existing intake structure.

Work under this item shall also include furnishing and installation of ductile iron pipe, gate valves, valve operators, bends, tees, elbow, intake strainer, and any other necessary components to construct the new pond drain piping system.

Work under this item shall also include the pressure pipe system; and commissioning of the system to the Town of Littleton.

Work under this item shall also include the design, furnishing and installation of the concrete valve vault and associated hardware including the trash rack, valve stem extension and guides, and telescoping valve wrench.

The first 20 percent (20%) of the lump sum shall be paid once the submittal of a pond drain system plan to the Owner and Engineer has been reviewed and accepted. The next 60 percent (60%) of the lump sum shall be paid once the pond drain system is in place and properly functioning. The final 20 percent (20%) shall be paid after commissioning of the system to the Town of Littleton and the acceptance of Substantial Completion.

Item 10: NEW STONE BLOCK WALL RECONSTRUCTION

Measure per cubic foot for furnishing, transporting, handling, and placing the materials specified including new or reused wall stones, concrete grout, and for furnishing all labor, tools, equipment, and incidentals necessary to complete the work.

Measurement not to exceed the measurements shown on the plans or as authorized by the Resident Engineer. Vertical dimension limits will be from the top of the foundation to the top of the new block installed. Horizontal dimension limits for width will be from each end of the reconstructed wall as shown in the Plan. Horizontal dimension limits for depth will be neat line that defines the front face of the block to the neat line that defines the back face of the new block. Contractor shall layout the blocks and confirm with Resident Engineer on dimensions prior to installation of next elevation of blocks.

Item 11: EXISTING STONE BLOCK WALL RECONSTRUCTION

Lump sum, no measurement required.

Payment shall be lump sum for pressure washing and cleaning of the existing stone wall to remain, repointing of existing joints with concrete grout, and other associated components required to repair the wall as shown on the Drawings and / or as directed by the Engineer.

Item 12: EMBANKMENT MATERIAL

Measure the cubic yardage of embankment material required to be placed, compacted, and tested as indicated by the payment limits shown on the Drawings and / or directed by the Engineer. Laboratory confirmation testing for compliance with the gradation specification and field density tests are considered subsidiary to this item.

Payment shall be per cubic yard of embankment material placed, shaped, compacted, and tested in the Dam.

Item 13: PEDESTRIAN BRIDGE

Lump sum, no measurement required.

Payment shall be lump sum for the design, furnishing, and installation of the pedestrian bridge system including the concrete foundation, steel bridge, and gravel access connecting the paved parking lot to the existing trail, as shown on the Drawings and / or as directed by the Engineer.

The first 20 percent (20%) of the lump sum shall be paid once the submittal of a pedestrian bridge plan to the Owner and Engineer has been reviewed and accepted. The lump sum shall be paid in the following for completed portions of the Work:

- 20 percent (20%) for installation of the concrete abutments,
- 10 percent (10%) for the installation of the gravel path,
- 40 percent (40%) for the installation of the pedestrian bridge

The final 10 percent (10%) will be paid after the completion of the pedestrian bridge and any repairs required for restoring surfaces as directed by the Owner or Engineer.

Item 14: PARKING LOT IMPROVEMENTS

Payment shall be lump sum for complete installation of the new paved parking spaces as described herein and as shown on the plans including (not limited to): Removal and off-site disposal of trees, stumps, roots, muck, and other objectionable materials deleterious to the Work, clearing and grubbing, rough grading, compaction proof-rolling of the prepared subgrade, placement of geotextile fabric, drainage and ditching, placement and grading of compacted crushed gravel, installation of new catch basins, including all required foundation preparation, installation of storm drain piping, installation of pavement, grading, placement, and all other work, including loam, seed, lime, fertilizer, mulch required to construct the improvements as described herein and / or as directed by the Engineer.

The first 40 percent (40%) of the lump sum shall be paid once the rough grading of the parking lot has been completed and the area has been proof-rolled compacted and catch basin structures and associated storm drain piping have been installed.

The second 40 percent (40%) of the lump sum shall be paid once parking area has been paved, compacted and tested, and adjacent disturbed areas have been top soiled, seeded, and temporary erosion control measures installed and ready to receive granular material.

The remaining 20% shall be paid upon acceptance of substantial completion, removal of equipment from the paved surfaces, and completion of punch list items.

Item 15: RESTORATION OF SURFACES

Lump sum, no measurement required.

Payment shall be lump sum for restoring all surfaces to their original condition. These areas include (not limited to): pavement surface along Dells Road, access parking lot off Dells Road, staging areas, etc. Work shall include, as required by the Engineer, repair of and clean sweeping of pavement, applying loam / topsoil, seed, and hay mulch, biodegradable temporary erosion control blankets, coir rolls, and other required items to restore the landscape surface as shown on the Drawings and other areas disturbed due to the activities of the Project as directed by the Engineer.

END OF SECTION

Ward Geotechnical
Consulting, PLLC

Geotechnical Investigation: Dells Pond Dam

Littleton, New Hampshire
NHDES Dam No. 140.10

Submitted to:
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THE STATE OF NEW HAMPSHIRE
CRAIG F. WARD
No. 6776
LICENSED
PROFESSIONAL ENGINEER

Date: December 2015
Project: 14830

Craig F. Ward, P.E.
Principal

Table of Contents

Section 1 – Project Description

1.1 Site and Project Description

The project involves the evaluation and design of the reconstruction of the Dells Pond Dam in Littleton, New Hampshire. The location of the dam is shown on Figure 1. A site plan of the dam is shown on Figure 2.

The dam, which was constructed by New Hampshire Fish & Game circa 1936, is considered a High Hazard structure by the New Hampshire Department of Environmental Services Dam Bureau (Dam Bureau). The dam is an approximately 125-foot-long earthen embankment dam with a cut stone masonry spillway. The embankment on either side of the spillway is supported along its downstream side by cut stone masonry retaining walls with a maximum exposed height of about 12 feet. The spillway is approximately 20 feet long with an exposed height of approximately 14 feet. Cut stone masonry steps at the toe of the spillway serve to dissipate the energy of the flow over the spillway and provide scour protection. The spillway walls are cut stone masonry retaining walls that extend through the width of the embankment and support a one lane steel girder bridge with a wooden deck. The spillway channel has cut stone masonry training walls that extend to a distance of approximately 34 feet (right training wall) and 40 feet (left training wall) downstream of the spillway. The spillway channel has a concrete apron that extends to about 23 feet from the toe of the spillway. The dam has a 12-inch-diameter low level outlet pipe that passes through the spillway. The valve used to operate the low level outlet is located in a wooden valve pit on the bottom of the pond about 45 feet upstream of the spillway.

The dam is located on town-maintained conservation land in a primarily residential area on Dells Road. A wooden deck is located on the edge of the pond on the upstream slope of the left embankment. A hiking trail begins at a paved parking lot on the west (right) abutment of the dam, crosses the spillway bridge and the crest of the embankment, and continues east into the woodland. The area generally slopes downward from north to south and is covered primarily by pine trees. Based on soil mapping provided on the National Resource Conservation Service (NRCS) website, the site contains primarily glacial outwash deposits of sand and gravel with varying amounts of fines (soils passing the no. 200 sieve).

A Letter of Deficiency (LOD) was issued by the Dam Bureau on March 23, 2011. The most significant deficiency identified in the LOD is that the spillway has inadequate capacity and cannot pass the required design storm event without overtopping. Other items identified in the LOD include the need to regrade the upstream embankment face and establish hearty grass cover, remove brush growing along the downstream side of the embankment, replace mortar missing from the cut stone masonry retaining and training walls, repair erosion damage along the downstream groins, investigate seepage through the cut stone masonry training walls, and conduct breach analyses and update the Emergency Action Plan.

The Town of Littleton has engaged the design team to investigate the deficiencies identified in the LOD and design the reconstruction of the dam. The design team consists of Headwaters Hydrology, PLLC (HH), DuBois & King, Inc. (D&K), and Ward Geotechnical Consulting, PLLC (WGC). HH's role is to conduct the necessary hydrologic and hydraulic analyses, address wetland issues, and provide survey services. D&K is responsible for addressing civil and structural issues and for preparation of design drawings and specifications for the reconstruction. WGC is responsible for geotechnical investigations and analyses and to develop recommendations for geotechnical aspects of the design of the reconstruction.

Based on hydrologic and hydraulic analyses conducted by HH, the reconstruction must include raising the dam by about 2 feet so the spillway can pass the design storm event with at least 1 foot of freeboard. Two options for raising the dam are being considered. One option would be to place compacted fill to raise the embankment by 2 feet. The other option would be to install steel sheet pile walls along the upstream slope of the embankment. The sheet pile walls would create parapet walls at the required elevation of the top of the dam, and could be faced with concrete for aesthetic purposes. Both options would require that the existing bridge and spillway walls also be raised by at least 2 feet.

1.2 Review of Existing Information

We have reviewed information obtained by D&K from the files of the Dam Bureau and the New Hampshire Department of Transportation (NHDOT). This information includes the following:

- LOD issued by the Dam Bureau on March 23, 2011, which is discussed in Section 1.1 (above).

- Inspection reports prepared by the Dam Bureau for inspections completed November 23, 2010 (which formed the basis for the LOD) and August 16, 2012. The deficiencies noted in these inspection reports are similar to those identified in the LOD.

- The Phase I Inspection Report for the dam prepared by the Army Corps of Engineers, dated April 1979. In general, the Phase I report identifies the same deficiencies identified in the LOD. Additional deficiencies noted included the partial collapse of the downstream portions of the downstream training walls and the inoperable low level outlet. It was also noted in the report that a nearby resident indicated that several truckloads of clay were placed on the upstream face of the dam about 4 or 5 years before the Phase I inspection.

- An inspection report prepared by the NHDOT for the spillway bridge on September 30, 2013. The report indicates that the steel superstructure was in fair condition, although a 20% section loss in the steel girders due to corrosion was noted. The deck was in good condition and the substructure was rated as satisfactory.

No original design drawings or as-built drawings of the dam or the spillway bridge were found in the files of the Dam Bureau or the NHDOT. D&K also contacted the New Hampshire Fish & Game Department and the New Hampshire Public Utilities Commission, but original design or as-built drawings could not be located.

1.2 Terminology

For the purpose of this report, we have adopted the following terminology:

- The directions "left" and "right" are based on the observer looking downstream. For example, the portion of the embankment east of the spillway is referred to as the left embankment and the portion of the embankment west of the spillway is referred to as the right embankment.

- The "spillway walls" are the cut stone masonry walls immediately upstream of the spillway, which are perpendicular to the embankment and support the spillway bridge.

- The "upstream training walls" are the masonry walls that extend upstream from the spillway walls along the toe of the upstream embankment.

- The "downstream training walls" are the masonry walls that line the spillway channel downstream of the spillway.

- The "embankment retaining walls" are the cut stone masonry walls that support the downstream sides of the left and right embankments.

- Elevations are referenced to the National Geodetic Vertical Datum of 1929.

Section 2 – Site Observations

Craig Ward of WGC visited the site on the following occasions:

- Mr. Ward visited the site on May 18, 2015, to observe the condition of the dam and to stake the locations of proposed borings for Dig Safe utility clearance. At the time of the site visit, the impoundment was at approximately normal pool.

- Mr. Ward was on site from June 1 to June 3, 2015, to observe the boring program. The impoundment was at approximately normal pool during the boring program.

- Mr. Ward visited the site on September 17, 2015, with Robert Durfee of D&K and Corey Clark of the Dam Bureau to observe the condition of the dam, to assist Mr. Durfee of D&K with field measurements, and to observe the excavation of test pits logged by Mr. Durfee. At the time of the site visit, the Town had lowered the level of the impoundment (by pumping) to about 1 foot below the level of the spillway crest to facilitate observation of the spillway. The test pits were excavated to observe the upper portions of the cut stone masonry embankment retaining walls and the right spillway wall.

Observations made during the site visits are described below:

1. Several sinkholes were observed on the upstream slope of the embankment behind the upstream training walls. These sinkholes appear to have been formed by erosion of the backfill soils through joints or gaps in the stone masonry due to infiltration of surface runoff and/or fluctuations of the pond level. The sinkholes are larger and better defined behind the right upstream training wall, where the back of the masonry wall is visible through one of the sinkholes. The sinkholes behind the left upstream training walls are smaller and appear as depressions in the ground surface.

2. The upstream training walls are poor condition. Several of the cut stones are misaligned and appear to have settled or been undermined.

3. The downstream ends of the downstream training walls have been undermined and have partially collapsed. The partially collapsed portions are downstream of the concrete apron that extends about 23 feet downstream of the spillway.

4. Seepage was observed from the lower portions of the downstream training walls, within about 20 feet of the spillway. The upper seepage lines on the walls appear to be no more than about 1 foot above the water level in the spillway channel at the time of the observations. During the site visit on September 17, 2015 (with the impoundment level about 1 foot below the crest of the spillway), the upper seepage

line along the right upstream training wall was no more than about 1 foot above the concrete apron and seepage was visually estimated to be no more than 3 gallons per minute (gpm). Seepage at that time was significantly less along the left training wall than that observed along the right training wall. No evidence of sediments was observed along the apron at the base of the training walls and the seepage appeared to be clear. With the impoundment drawn down, there appeared to be a small amount of seepage from the downstream face of the spillway.

5. During the site visits when the impoundment was at approximately normal pool level, seepage was observed at the junction between the left embankment retaining wall and the left spillway wall at approximately elevation 775 feet. The seepage at that time was flowing at a rate of about 1 to 2 gpm. During the site visit on September 17, 2015, when the impoundment was about 1 foot below normal pool, no active seepage was observed from this area, but the area was moist.

   No other seepage was observed along the downstream faces of the embankment retaining walls, or from the ground surface downstream of the retaining walls.

6. During the site visit when the impoundment was lowered to about 1 foot below normal pool, a hole was observed immediately behind the upstream end of the left spillway wall (at its junction with the left upstream training wall). The hole, which had the appearance of an animal burrow, is about 4 to 6 inches in diameter and located just below normal pool elevation (and just above the impoundment level at the time of the visit). The orientation of pine needles at the entrance of the hole suggests water flow into the hole has occurred in the past.

   Note that the hole is located directly upstream of the seepage at the junction of the left embankment retaining wall and the left spillway wall, which was observed at times when the impoundment was at normal pool (item 5, above). The fact that this seepage stopped when the impoundment was lowered enough to expose the hole suggests that the hole is the source of the seepage.

7. During the visit when the impoundment was drawn down and little water was flowing over the concrete apron, two 4-inch-diameter PVC pipes were observed extending from below the downstream end of the apron. This appears to indicate that a drainage system was installed beneath the apron. Although the drain pipes were under water, one of the pipes appeared to flowing, indicating at least some functionality of the drainage system below the apron. It was not determined whether or not the other pipe was flowing. No sediments were observed at the outlets of the pipes.

8. While on site when the impoundment was drawn down, Mr. George McNamara of the Littleton Public Works Department indicated that the Town attempted to lower the impoundment for our inspection by opening the valve at the upstream end of the

low level outlet but found that the low level outlet pipe had a blockage about 18 feet upstream of the downstream end of the pipe. Therefore, the impoundment had to be lowered using pumps.

9. Brush and saplings were observed along areas downstream of the left and right embankment retaining walls. Mature pine trees were observed on the right and left abutments, within about 10 to 20 feet of the ends of the embankment. Trees were also observed behind the downstream portions of the downstream training walls.

Section 3 – Subsurface Investigation

3.1 Boring Program

WGC engaged New England Boring Contractors, Inc. to drill three borings at the site. The boring program was conducted from June 1 through June 3, 2015, under the observation of WGC. The three borings (B1, B2, and B3) were drilled in the crest of the dam embankment at the locations shown on Figure 2. Logs of the borings are provided in Appendix A.

The borings were drilled to depths ranging from 37.7 to 53 feet below the crest of the embankment using cased wash boring drilling techniques. Split-spoon sampling with standard penetration tests (SPTs) was typically performed at intervals of 5 feet, or less. One of the borings (B3) was advanced to refusal on bedrock, and a 5-foot-long bedrock core sample was drilled. The other two borings (B1 and B2) were advanced to refusal on either bedrock or a boulder, and a tricone roller bit was used to advance these borings 1.5 feet (B2) and 2 feet (B1) into the bedrock or boulder.

Falling head permeability tests were conducted below the phreatic surface (upper line of seepage) in borings B1 and B2. The test intervals were 16 to 18 feet below the embankment crest in B1, and 18 to 20 feet below the embankment crest in B2.

A 2-inch-diameter PVC observation well was installed to a depth of 41.9 feet below the crest of the embankment in B2. A steel roadbox protective casing was installed flush to the ground surface.

After completion of drilling, the boreholes for B1 and B3 were filled with cement grout using a tremie.

3.2 Test Pit Program

On September 17, 2015, with the impoundment lowered by about 1 foot, 5 shallow test pits were excavated along the crest of the dam at the locations shown on Figure 2. The test pits were excavated to depths ranging from about 2.3 to 4 feet below the crest by Town personnel and logged by D&K. The primary goal of the test pits was to observe the thickness of the tops of the embankment retaining walls and the right spillway wall and to observe the lateral extent of the right end of the right embankment retaining wall. The soil conditions in the test pit excavations were observed by WGC and are reflected in the logs prepared by D&K, which are provided in Appendix B.

3.3 Geophysical Testing

WGC engaged Radar Solutions International, Inc. (Radar Solutions) to conduct geophysical testing at the dam in attempt to estimate the depth to the bottom of the maximum section of the left embankment retaining wall and the thicknesses of both walls at various locations and depths. Geophysical testing could not be performed for the spillway and spillway walls due to water flow over the spillway (the impoundment was at about normal pool during geophysical testing). Radar Solutions utilized various geophysical testing methods including ground penetrating radar (GPR), parallel seismic (seismic), and sonic echo (sonic) techniques. The GPR survey was performed at various sections along the downstream face of the embankment retaining walls to estimate wall thickness vs. depth. The sonic method was performed to estimate the depth to the bottom of the maximum section of the left embankment retaining wall (section that includes boring B2). The seismic method was also performed to estimate the depth to the bottom of the maximum section of the left embankment retaining wall, with seismic energy strikes at the top of the wall and vibrations measured using a geophone at various depths within the observation well installed in B2. The results of the geophysical testing is provided the report *Geophysical Characterization of Dells Pond Dam, GPR, Parallel Seismic, and Sonic Echo Surveys, Littleton , New Hampshire*, prepared by Radar Solutions, dated August 6, 2015, and the addendum to the report dated December 22, 2015 (reports provided under separate cover).

Section 4 – Subsurface Conditions

4.1 Soil, Groundwater, and Bedrock Conditions

The subsurface soil, groundwater, and bedrock conditions encountered in the borings and test pits are described below. The subsurface conditions through the maximum section of the embankment (through B2) are also shown on Figure 3. Subsurface conditions are known only at the boring and test pit locations, and conditions at other locations may differ from those described in this report.

The subsurface conditions encountered in the borings and test pits are described below, from the ground surface, down:

Embankment Fill (Unified Soil Classification System SW, SP, SP-SM, and ML) – Embankment fill was encountered in all of the borings. The thickness of the embankment fill layer ranges from 14 to 17 feet at the boring locations. The constituents of the embankment fill are variable, but consist primarily of sand and sand with gravel, with varying amounts of fines (soil passing the no. 200 sieve) and occasional cobbles and boulders. However, an approximately 5-inch-thick layer of sandy silt was encountered in the upper 2 feet of the fill in B1. SPT N-values in the fill ranged from 5 to 20 blows per foot, indicating that the fill is loose to medium dense.

Sand with Gravel, Sand, Silty Sand, and Silty Sand with Gravel (SW, SP, and SM) - A deposit of natural granular soils with variable fines contents was encountered beneath the embankment fill at the boring locations. The soils in this deposit consist primarily of sand and sand with gravel, but silty sand and silty sand with gravel (as well as an approximately 2-inch-thick sandy silt lens) were also observed in the deposit. SPT N-values in the deposit ranged from 9 to 22 blows per foot, indicating that the deposit is loose to medium dense. Falling head permeability tests performed at depth intervals of 16 to 18 feet in B1 and 18 to 20 feet in B2 indicated permeability values of 7.3×10^{-4} and 1.2×10^{-3} centimeters per second (cm/s), respectively. The deposit shows evidence of stratification and was probably deposited as glacial outwash.

Silty Sand with Gravel and Sandy Silt with Gravel (SM and ML) – A layer of silty sand with gravel and sandy silt with gravel was encountered beneath the outwash deposit in the borings. The layer consists primarily of silty sand with gravel, but transitions to sandy silt with gravel near the bottom of the layer at B2. SPT N-values in the layer ranged from 27 to greater than 100 blows per foot, indicating that the layer is medium dense to very dense. The layer was probably deposited as glacial till.

Bedrock – Bedrock was encountered beneath the glacial till layer at a depth of approximately 47 feet below the crest of the embankment in B3, where a 5-foot-long bedrock core sample was drilled. The bedrock observed in the core sample consists of fresh to slightly weathered, coarse grained, igneous or metamorphic rock, probably a granodiorite. Joints in the core sample are near horizontal and dipping from 30° to 45° from the horizontal, at spacings ranging from less than 1 inch (in the more highly fractured upper 1 foot of the sample) to 17 inches. The rock quality designation (RQD) of the core sample is approximately 76%.

Either bedrock or a boulder was encountered at a depth of 35.7 feet below the embankment crest in B1 and at a depth of 43 feet below the embankment crest in B2. Core samples were not obtained in these borings to determine whether bedrock or boulders were encountered. The borings were advanced 1.5 feet (B2) and 2 feet (B1) into the boulder or bedrock surface using a tricone roller bit.

Groundwater – A groundwater observation well was installed to a depth of 41.9 feet below the embankment crest in B2. When measured on June 3, 2015 (about 17 hours after well installation), the groundwater level in the well was 14.7 feet below the crest of the embankment (groundwater elevation approximately 770.3 feet). The impoundment was at approximately normal pool level at the time of the groundwater level measurement.

During the site visit on September 17, 2015, when the impoundment had been lowered to about 1 foot below normal pool, the groundwater level measured in the observation well was 15.1 feet below the embankment crest, corresponding to approximately elevation 769.9 feet.

4.2 Cut Stone Masonry Walls

As previously indicated, no original design or as-built drawings of the dam were located during D&K's file review. Therefore, no information regarding the geometry of the portions of the cut stone masonry walls that are below grade is available, and the thicknesses and the depths to the bottoms of the walls are not known. In order to obtain additional information concerning the geometry of the walls for use in our seepage and stability analyses, test pits were excavated to observe the upper portions of the embankment retaining walls and the right spillway wall, and geophysical testing was conducted to estimate the depth to the bottom of the maximum section of the left embankment retaining wall and the variation of the thickness of various sections of the embankment retaining walls with depth.

Embankment Retaining Walls

Based on the observations of exposed portions of the walls and the results of the test pits and geophysical testing, we developed assumed wall depth and geometry of the maximum section of the left embankment retaining wall (section through B2) for use in our seepage and stability analyses. Our assumed section was based on the following:

- The downstream face of the maximum section of the left embankment retaining wall is battered at approximately 13° to the vertical over the approximately 12-foot exposed height.

- The tops of the embankment retaining walls observed in the test pits typically ranges in thickness from about 2 to 4 feet, except in TP1, which was excavated at the right end of the right wall (near the abutment). In TP1 the top of the wall is approximately 1.5 feet wide, but the back of the wall steps out an additional 1.2 feet approximately 1.5 feet below the top of the wall.

- The backs of the retaining walls observed in the test pits are battered or stepped, such that the thickness of the walls increases with depth. The test pits were too shallow to accurately measure the angle of the batter.

- The GPR survey along the exposed faces of the retaining walls indicates that the walls increase in thickness with depth, as would be expected given that the front and back faces of the walls are battered. The thickness of the walls at the level of the ground surface at the toes of the wall appears to range from about 5.5 feet, where the ground surface at the toes of the walls rises (leaving less exposed height), to about 8 or 9 feet, where the ground surface at the toes of the walls is lower (leaving larger exposed height). At the maximum section of the wall (the section including B2), the wall appears to be about 8 feet thick at the level of the ground surface at the toe of the wall.

- The sonic and parallel seismic surveys appear to indicate that the overall height of the wall at its maximum section (section including B2) is in the range of 26.5 to 28.5 feet, with the depth of the bottom of the wall approximately 13.5 to 15.5 below the ground surface at the toe of the wall. This corresponds to about 7.6 to 9.6 feet below the level of the top of the concrete apron in the spillway channel.

  While it is possible that the maximum section of the wall is embedded this deep, thereby founding the wall on the dense, less permeable, glacial till soils (beneath the more permeable outwash) and providing a more effective seepage cutoff, we are somewhat skeptical due to the effort that would be required to excavate that deep (about 9.5 to 11.5 feet below original ground) in granular soils below groundwater. Also, the depth to the bottom of the embankment fill observed in B2, which was drilled about 10 feet behind the face of the wall, is only about 17 feet below the crest of the embankment. If the wall were embedded to a depth of 26.5 to 28.5 feet below the crest, we would expect the depth to the bottom of the fill at B2 to be greater than 17 feet, unless the excavation was very steeply cut or sheeting was used for excavation support. Therefore, for the purposes of the stability and seepage analyses, we have assumed that that the maximum section of the wall bears at approximately the level of the bottom of the embankment fill at B2, or about 4 feet below the existing ground surface at the toe of the wall at the maximum section.

For our seepage and stability analyses, we elected to use a more conservative interpretation of the maximum retaining wall section than that indicated solely by the geophysical surveys. As indicated above, the depth to the bottom of the wall at the maximum section was assumed to be about 17 feet below the embankment crest, or approximately 4 feet below the existing ground surface at the toe of the wall (even through the geophysical testing indicated an embedment depth of 26.5 to 28.5 feet below the crest). Also, although the GPR survey indicated that the thickness of the bottom of the exposed portion of the wall at this section is about 8 feet, we opted to use an assumed width of 7 feet at the bottom of the wall (at a depth of 4 feet below the existing ground surface at the toe). Our assumed geometry for the maximum retaining wall section for use in our seepage and stability analyses is shown on Figure 3 and is summarized below:

- Top width of the retaining wall of 2 feet.
- Width of the base of the retaining wall of 7 feet.
- Depth of the base of the retaining wall of 17 feet below the crest of the embankment (about 4 feet below the existing ground surface at the toe).
- Batter at the face of the wall of about 13°. Using the assumed geometry described above, the batter of the back of the wall is approximately 4.7°.

Spillway

Geophysical surveys were not conducted for the spillway because at the time of the geophysical field work the impoundment was at approximately normal pool and water was flowing over the spillway. Therefore, the spillway geometry used for our stability and seepage analyses was based primarily on observations of exposed portions of the spillway. The bottom of the spillway was assumed to extend about 4 feet below the top of the lowest step at the toe of the spillway. The back batter of the spillway was assumed to be the same as that assumed for the maximum section of the embankment retaining wall. The assumed spillway geometry for use in our seepage and stability analyses is shown on Figure 4 and is summarized below:

- Top width of the spillway crest of 2.75 feet.
- Depth of the base of the spillway of about 4 feet below the level of the top step at the toe.

- Batter at the face of the spillway of about 9°. Batter of the back of the spillway of 4.7° (same as assumed for embankment retaining wall).

- Width of the base of the retaining wall of 11.8 feet (based on exposed dimensions, assumed back batter, and assumed embedment depth).

Section 5 – Seepage Analyses

5.1 General

Seepage analyses were performed for the maximum embankment retaining wall section and for the spillway at normal pool and design flood conditions. The assumed wall geometries for these sections, which are shown on Figures 3 and 4, are described in section 4.2 of this report. The soil profiles shown on Figures 3 and 4 are based on the results of the borings. Normal pool and design flood head water and tailwater elevations were provided by HH.

The permeability (K) value for the outwash sand and sand with gravel deposit is based on falling head permeability tests that were conducted in the borings. Permeability values of the other soils and materials were assumed based on published values for similar materials. Where appropriate, the assumed values were varied within reasonable limits to provide results that agree with site observations of seepage (or absence of seepage) and observation well measurements. The assumed and measured permeability values used in the final analyses are as follows:

- Embankment Fill: $K = 2 \times 10^{-4}$ cm/s
- Sand and Sand with Gravel (Outwash): $K = 1 \times 10^{-3}$ cm/s (based on falling head permeability testing)
- Silty Sand with Gravel (Glacial Till): $K = 1.5 \times 10^{-5}$ cm/s
- Bedrock: assumed impervious
- Masonry Wall: $K = 5 \times 10^{-6}$ cm/s
- Sediment (spillway only): $K = 1 \times 10^{-4}$ cm/s

The analyses were conducted using the SEEP/W computer program (part of the GeoStudio 2012 suite of geotechnical software). Plots of the results of the seepage analyses are provided in Appendix C. Note that elevations shown on the plots are 740 feet less than the actual elevations (NGVD).

5.2 Maximum Embankment Retaining Wall Section

Seepage analyses were conducted for the maximum embankment retaining wall section (section including B2) for normal pool and design flood conditions. The results of the analyses are provided in Appendix C and discussed below:

Normal Pool Condition

The analyses for the normal pool condition were performed assuming existing conditions. As shown on the seepage analysis plot on Figure C.1 (Appendix C), a constant head boundary condition was assigned for the head water along the submerged portion of the upstream slope of the embankment. A constant head boundary condition was assigned for the tailwater along the spillway channel (beyond the concrete apron in the spillway channel), assuming that groundwater flows toward the spillway channel. The position of the phreatic surface (upper line of seepage) between the constant head boundary conditions was determined by the SEEP/W analysis.

In our review of the results of the analysis, it was noted that the predicted normal pool phreatic surface at the location of the well in B2 (about 10 feet behind the face of the wall) was about 3.2 feet higher than the actual phreatic surface measured in the well during normal pool conditions (14.7 feet below embankment crest, or approximately elevation 770.3 feet). In our opinion, this is due to fact that the section containing B2 is located only about 11 feet from the spillway channel, causing the actual phreatic surface in this area to be lowered by drainage into the spillway channel and/or the drain that underlies the apron. To evaluate the influence of drainage to the apron underdrain, we conducted an additional analysis in which the apron underdrain is modeled as a constant head boundary with the pressure head equal to the tailwater elevation. The plot for this seepage analysis provided on Figure C.2 (Appendix C) shows the predicted phreatic surface lowered to about 0.8 feet above the actual phreatic surface measured in the well in B2 during normal pool conditions, which is considered reasonable agreement.

The influence of drainage to the spillway channel on the phreatic surface is likely to be less pronounced along sections located farther from the spillway. In our opinion, the phreatic surfaces (and seepage pressures) along sections of the embankment retaining wall located farther from the spillway are probably more closely approximated by the initial seepage analysis that doesn't include drainage to the apron underdrain (Figure C.1, Appendix C). Therefore, we have elected to use the phreatic surface and seepage pressures predicted by the initial seepage analysis (without the apron underdrain) for our stability analyses. The higher predicted phreatic surface and seepage pressures result in more conservative stability analyses.

The plots for both of these seepage analyses indicate that the seepage emerges from the downstream face of the embankment retaining wall below the existing ground surface, which agrees with observations of the wall during normal pool conditions. The plots also indicate a relatively small exit gradient to the constant head boundary in the spillway channel.

Design Flood Condition

The analysis for the design flood condition was performed for proposed conditions in which the dam is raised to provide at least 1 foot of freeboard above the design flood impoundment level. As previously indicated in this report, two options are being considered for raising the dam. One option is to raise the embankments by placing fill. The other option is to install sheet pile walls along the upstream slopes of the embankments. The sheet pile walls would act as a partial seepage cutoff, resulting in a lower phreatic surface and smaller seepage pressures than if the dam were raised by placing fill on the embankments. Therefore, for our seepage analysis, we've elected to assume that the dam would be raised by placing fill on the embankments. The higher phreatic surface and seepage pressures associated with this option result in more conservative stability analyses.

As shown on the seepage analysis plot on Figure C.3 (Appendix C), a constant head boundary condition was assigned for the head water along the submerged portion of the upstream slope of the embankment. A constant head boundary condition was assigned for the tailwater, which intersects the existing ground surface several feet downstream of the toe of the embankment retaining wall. The position of the phreatic surface between the constant head boundary conditions was determined by the SEEP/W analysis.

Note that preliminary seepage analyses indicated that seepage during the design flood would break out on the ground surface immediately downstream of the embankment retaining wall. Therefore, for the final seepage analysis, a filtered drainage berm along the toe of the embankment retaining wall was included to contain and control seepage.

The plot for the seepage analysis, which is provided on Figure C.3 (Appendix C), indicates that the seepage would be contained by the drainage berm. The plot also indicates a relatively small exit gradient to the downstream constant head boundary.

5.2 Spillway Section

Seepage analyses were conducted for the spillway section for normal pool and design flood conditions. The results of the analyses are provided in Appendix C and discussed below:

Normal Pool Condition

The analysis for the normal pool condition was performed assuming existing conditions. As shown on the seepage analysis plot on Figure C.4 (Appendix C), a constant head boundary condition was assigned for the head water along crest of the spillway and the surface of the soil behind the spillway. A constant head boundary condition was assigned for the tailwater at the apron underdrain in the spillway channel. The position of the phreatic surface (upper line of seepage) between the constant head boundary conditions was determined by the SEEP/W analysis.

The seepage analysis plot on Figure C.4 (Appendix C) indicates that the seepage emerges from the downstream face of the spillway, which agrees with observations of the spillway when the impoundment had been lowered. The plot also indicates a relatively small exit gradient to the constant head boundary at the apron underdrain in the spillway channel.

Design Flood Condition

The analysis for the design flood condition was performed for existing conditions, since the proposed raising of the dam would not significantly affect seepage conditions at the spillway section.

As shown on the seepage analysis plot on Figure C.5 (Appendix C), the constant head boundary conditions were the same as those assumed for the normal pool condition except that the design flood head water and tailwater pressures were used. The position of the phreatic surface between the constant head boundary conditions was determined by the SEEP/W analysis.

The seepage analysis plot on Figure C.5 (Appendix C) indicates that the seepage emerges from the downstream face of the spillway. The plot also indicates a relatively small exit gradient to the constant head boundary at the apron underdrain in the spillway channel.

Section 6 – Stability Analyses

5.1 General

Stability analyses were conducted for the maximum embankment retaining wall section and the spillway section for the following load cases:

- Normal Pool – Head water in impoundment at spillway crest elevation 778.7 feet; tailwater in spillway channel at elevation 764.6 feet.

- Design Flood – Head water at elevation 785.7 feet; tailwater at elevation 771.3 feet. Design flood head water and tailwater elevations were provided by HH.

- Seismic – Pseudostatic analyses using a peak horizontal ground acceleration of 0.12g applied under normal pool head water and tailwater conditions. The peak horizontal ground acceleration of 0.12g is based on information provided on the USGS website for the ASCE 7-10 method.

The phreatic surfaces and seepage pressures used in the stability analyses are approximately those predicted by the seepage analyses described in Section 5 of this report. The seepage analysis plots on Figures C.1 and C.4 were used for the normal pool case. The seepage analysis plots on Figure C.3 and C.5 were used for the design flood case.

As part of the stability analyses, the liquefaction potential of the soils below the normal pool phreatic surface were evaluated using the method described in *Liquefaction Resistance of Soils: Summary Report from the 1996 and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, Journal of Geotechnical and Geoenvironmental Engineering, April 2001. Our evaluation indicates that the soils that underlie the site are not susceptible to liquefaction under design earthquake conditions.

5.2 Maximum Embankment Retaining Wall Section

Both global and external (or local) stability analyses were conducted for the maximum embankment retaining wall section for the existing and proposed conditions shown on Figure 3. The stability of the existing embankment was analyzed for the normal pool case. The stability of the embankment for proposed conditions was analyzed for the normal pool, design flood, and seismic load cases.

The proposed conditions used in the analyses are for the option in which the embankment would be raised by 2 feet by the placement of compacted fill. As indicated in Section 5, the option of raising the dam by installing sheet pile walls along the upstream slopes of the embankments would result in a lower phreatic surface and smaller seepage pressures.

Therefore, for our stability analysis, we've elected to assume that the dam would be raised by placing fill on the embankments. The higher phreatic surface and seepage pressures associated with this option result in more conservative stability analyses.

Global Stability Analyses

Global stability analyses were conducted for existing conditions and for the proposed conditions both with and without the placement of the drainage berm along the toe of the retaining wall. As indicated in Section 5 if this report, the drainage berm is necessary to control seepage along the toe of the retaining wall for the design flood condition. For the stability analyses that include the berm, the berm was assumed to be 2.5 feet thick and extend a distance of 20 feet from the toe of the wall.

The required minimum factors of safety for each load case are provided in *Stability of Earth and Rock-Fill Dams*, U.S. Army Corps of Engineers, EM-1110-2-1902, dated April 1, 1970.

The global stability analyses were performed using the SLOPE/W computer program (part of the GeoStudio suite of geotechnical software) using the Morgenstern-Price method. The results of the global stability analyses are shown on the plots provided on Figures D.1 through D.7 (Appendix D) and summarized in Table 1. The results of the global stability analyses indicate the following:

- The factor of safety (FS) for the existing maximum section indicates adequate stability for the normal pool load case (FS = 1.5).

- For the proposed conditions without the drainage berm, the maximum section would not be adequately stable for the normal pool and design flood load cases. The factors of safety for the normal pool load case (FS = 1.4) and the design flood load case (FS = 1.1) do not meet the minimum required factors of safety.

- For the proposed conditions with the drainage berm, the maximum section would be adequately stable for all load cases. The factors of safety for the normal pool case (FS = 1.8), the design flood case (FS = 1.4), and the seismic load case (FS = 1.3) meet or exceed the required minimum factors of the safety.

External (Local) Stability Analyses

External stability analyses were conducted for existing conditions and for the proposed conditions assuming the drainage berm would be placed along the toe of the retaining wall. External stability analyses for proposed conditions without the berm were not conducted because the global stability analyses indicate that the berm is needed for the maximum section to be adequately stable.

The external stability analyses of the retaining wall section were performed by hand calculation. Active earth pressure was assumed to be mobilized by the soil behind the wall. At rest earth pressure was assumed for the soil along the toe of the wall for the normal pool load condition. Half of the available passive earth pressure (i.e., a factor of safety of 2 was applied to the passive pressure coefficient) was assumed to be mobilized by the soil at the toe of the wall for the design flood and seismic load conditions.

The required minimum sliding and overturning criteria are provided in *Stability Analysis of Concrete Structures*, U.S. Army Corps of Engineers, EM 1110-2-2100, dated December 2005. Although the retaining wall is constructed of cut stone masonry instead of concrete, we consider the stability criteria to be appropriate for all gravity structures. In selecting the appropriate stability criteria, the normal pool, design flood, and seismic load cases were assumed to be the Usual Load Condition, the Unusual Load Condition, and the Extreme Load Condition (as described in the publication), respectively.

The results of the external stability analyses are provided in Table 2. The results of the sliding analyses are expressed in terms of factor of safety. The results of the overturning analyses are expressed in terms of the distance from the toe of the wall base to the resultant vertical force. As shown in Table 2, the external stability criteria are met or exceeded for all of the cases analyzed.

5.3 Spillway Section

External (local) stability analyses were conducted for the spillway section shown on Figure 4. The stability of the spillway was analyzed for the normal pool, design flood, and seismic load cases. These analyses apply to both existing and proposed conditions since the proposed raising of the dam does not explicitly include modifications to the spillway. However, as discussed later in this section, modification to the spillway will be necessary to satisfy stability criteria.

The conventional two dimensional stability analysis is based on plane strain conditions for which it is assumed that the structure or embankment represented by the section being analyzed is very long relative to the depth of the section. The spillway is 20 feet long with an exposed height of about 14 feet (assumed overall height of at least 18 feet). Also, the spillway is flanked by approximately 20-foot-long spillway walls, which add support to the spillway. Therefore, in our opinion, the plain strain assumption on which the two-dimensional SLOPE/W global stability analysis is based is not valid and a global stability analysis was not performed.

The external (or local) stability analysis also based on plane strain conditions. However, unlike a two dimensional global stability analysis, the external analyses can readily take into consideration the support offered by the spillway walls flanking the spillway. For our external stability analyses, we included this support by modeling the spillway walls as deadmen. To model the walls as deadmen, we included the horizontal restraint provided at the ends of the spillway by the spillway walls due to interface friction between the walls and the soils on both sides of the walls. We neglected friction along the bases of the spillway walls. Also, the weight of the spillway walls was not included as vertical stabilizing forces in the analyses, which would require the connection between the spillway and spillway walls to resist large moments.

The external stability analyses of the spillway section were performed by hand calculation. Due to the rigid restraint of the spillway by the spillway walls, at rest earth pressures were assumed for the soils behind and along the toe of the spillway. At rest earth pressure was also assumed for the soil along both sides of the spillway walls because these walls are buttressed by the spillway and the bridge. As previously indicated, the spillway walls were assumed to provide horizontal restraining forces at the ends of the spillway (due to interface friction between the walls and adjacent soils).

As for the external stability analysis for the embankment retaining wall, the required minimum sliding and overturning criteria used for our evaluation of the spillway are provided in *Stability Analysis of Concrete Structures*, U.S. Army Corps of Engineers, EM 1110-2-2100, dated December 2005.

The results of the external stability analyses are provided in Table 3. The results of the sliding analyses are expressed in terms of factor of safety. The results of the overturning analyses are expressed in terms of the distance from the toe of the spillway base to the resultant vertical force.

As shown in Table 3, the external stability criteria are met or exceeded for all of the cases analyzed except the sliding criteria for the design flood case. The calculated factor of safety for sliding during the design flood is about 1.16, compared to a required minimum factor of safety of 1.3. Further analyses indicate that an additional horizontal restraining force of approximately 45 kips would be needed to meet the sliding criteria during the design flood. This required restraining force could be provided by installing soil anchors. The required restraining force could also be provided by constructing a concrete berm with a submerged weight of at least 4.1 kips per foot of spillway along the upstream or downstream face of the spillway. Placement of the berm on the upstream face (or back) of the spillway would be preferable since this would also increase overturning resistance and could improve the structural connection between the spillway and the spillway walls. The dowel connection between the upstream face of the spillway and the concrete berm must be strong enough to support the weight of the berm.

Section 7 – Recommendations

We have developed preliminary design recommendations based on the results of our subsurface investigations and seepage and stability analyses:

- Based on our stability and seepage analyses, both options being considered for raising the dam to safely pass the design flood (i.e., raising the embankments by placing compacted fill, or installation of sheet pile walls on the upstream embankment slopes) are technically feasible. However, the option involving the construction of sheet pile walls along the upstream slopes of the embankments has several advantages from a geotechnical engineering perspective. With the sheet pile wall option, the dam would be more stable because this option wouldn't increase the height of the soil behind the embankment retaining walls, which would increase driving forces acting on the walls. In addition, the sheet pile walls would act as partial seepage cutoffs, lowering the phreatic surface through the embankments, thereby improving embankment and embankment retaining wall stability. Also, the sheet pile wall would probably reduce seepage through the downstream training walls.

- Berms must be constructed along the downstream toes of the embankment retaining walls to provide seepage control and improve stability of the embankments and embankment retaining walls. The berms should be 2.5 feet thick and extend from the toes of the retaining walls to distance of at least 20 feet from the toes of the walls. The berms should consist of a minimum 1.5-foot-thick layer of crushed stone underlying a minimum 1-foot-thick layer of drainage stone. The crushed stone should meet the requirements of No. 89 stone per Section 702 of the New Hampshire Department of Transportation Standard Specifications for Road and Bridge Construction, 2010 (NHDOT Specifications). The drainage stone should meet the requirements of Class C Stone per item 585.3 of the NHDOT Specifications. If the subgrade soils beneath the berms consist of sand or sand with gravel with low fines content (as expected), the No. 89 stone will provide adequate filtration. However, if the subgrade soils are found to be silty, it will be necessary to provide a minimum 1-foot-thick layer of concrete sand (per ASTM C-33) between the No. 89 stone and subgrade. It this is the case, the thickness of the No. 89 stone layer may be deceased to 1 foot.

  Note that the need for the berms was based on seepage and stability analyses conducted for the option in which the embankments would be raised by placing 2 feet of compacted fill. If the sheet pile wall option is selected instead, the phreatic surface through the embankments would be lowered and it might be possible that the berms could be eliminated. That said, placement of the berms would enhance seepage control and stability and should be considered even if the sheet pile wall option is selected. We could re-evaluate the need for the berms during final design if the sheet pile wall option is selected.

- The stability of the spillway must be increased to satisfy the sliding criteria under the design flood load case. The stability of the spillway could be increased by the installation of soil anchors or the placement of a concrete berm on the upstream side of the spillway. Soil anchors if used should provide a total allowable horizontal force of 45 kips. If a berm is used, it should have a submerged weight of at least 4.1 kips per foot of spillway. The berm must be connected to the spillway and spillway walls by dowels designed to support the weight of the concrete berm.

- The low level outlet is currently not functional and should be replaced.

- Proper drainage of seepage beneath the concrete apron is critical to the stability of the spillway, especially during flood conditions. Therefore, although the apron is in reasonably good condition, we recommend that it be removed and reconstructed with a filtered underdrain. The underdrain should consist of a minimum 1.5-foot-thick layer of crushed stone meeting the requirements of No. 89 per Section 703 of the NHDOT Specifications. If the subgrade soils beneath apron consist of sand or sand with gravel with low fines content (as expected), the No. 89 stone will provide adequate filtration. However, if the subgrade soils are found to be silty, it will be necessary to provide a minimum 1-foot-thick layer of concrete sand (per ASTM C-33) between the No. 89 stone and subgrade. It this is the case, the thickness of the No. 89 stone layer may be deceased to 1 foot. The underdrain should include at least three 4-inch-diameter PVC perforated or slotted drainage pipes laid within the crushed stone layer. The pipes should be parallel to the direction of channel and discharge to the channel downstream of the apron.

- The hole observed behind the upstream end of the left spillway wall probably contributes to the seepage observed at the intersection of the left spillway wall and left embankment retaining wall (see items 5 and 6 of Section 2). The hole should either be grouted or the area of the hole should be excavated and replaced by compacted fill during reconstruction.

- The upstream training walls should be reconstructed with filtered drains.

- The partially collapsed portions of the downstream training walls beyond the apron should be reconstructed. Both walls should include filtered drains with drainage pipes to discharge the intercepted seepage to the spillway channel at the downstream ends of the walls.

- The embankment retaining walls, spillway, spillway walls, and downstream retaining walls should be repointed with mortar where necessary.

- All trees and brush should be removed within 15 feet of the embankment, embankment retaining walls, and downstream and upstream training walls.

Section 8 – Limitations

Our preliminary design recommendations are based on the project information provided to us at the time of this report and may require modification if there are any project changes. Our current agreement for the project includes consultations during design and review of final design documents so that we can 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 borings, test pits, and geophysical surveys performed at the site. The nature and extent of variations in subsurface conditions 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 recommend that WGC be engaged to make several site visits during construction to:

1. Check that the subsurface conditions exposed during construction are in general conformance with our design assumptions.

2. Ascertain that, in general, the work is being performed in compliance with the contract documents and our recommendations.

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

Tables

Ward Geotechnical
Consulting, PLLC

TABLE 1 - RESULTS of GLOBAL STABILITY ANALYSES
Maximum Embankment Retaining Wall Section
Dells Pond Dam
Littleton, New Hampshire

LOAD CASE	FACTOR OF SAFETY Fsmin	FS
Existing Conditions		
Normal Pool	1.5	1.5
Proposed Conditions - Embankment Height Increased by 2 feet:		
Normal Pool - without berm at toe	1.5	1.4
Normal Pool - with 2.5' thick berm at toe		1.8
Design Flood - without berm at toe	1.4	1.1
Design Flood - with 2'.5 thick berm at toe		1.4
Seismic Load - without berm at toe	1.0	1.2
Seismic Load - with 2.5' thick berm at toe		1.3

Notes:

1. Minimum factors of safety are provided in Stability of Earth and Rock-Fill Dams, U.S. Army Corps of Engineers, EM 1110-2-1902, 1 April 1970 (REF).
2. The global stability analyses were conducted using the Morgenstern-Price method with the Slope/W computer program.
3. The Normal Pool and Design Flood Cases for proposed conditions without a berm at the toe of the wall do not meet the stability criteria. With the addition of a 2.5-foot-thick by 20-foot-long berm at the toe of the wall, the minimum factors of safety are met for all load cases.

TABLE 2 - RESULTS of EXTERNAL (LOCAL) STABILITY ANALYSES
Maximum Embankment Retaining Wall Section
Dells Pond Dam
Littleton, New Hampshire

LOAD CASE	SLIDING Factor of Safety FS_{min}	FS	OVERTURNING d_{min} (feet)	d (feet)	BEARING PRESSURE (KSF)
Existing Conditions					
Normal Pool	1.7	2.0	2.33	3.1	2.5
Proposed Conditions - Embankment Height Increased by 2' & 2.5' x 20' berm at toe:					
Normal Pool	1.7	1.7	2.33	2.6	2.8
Design Flood	1.3	1.4	1.75	2.3	2.1
Seismic Load	1.1	1.8	0.00	1.1	5.0

Notes:

1. Minimum sliding and overturning stability criteria are provided in Stability Analysis of Concrete Structures, U.S. Army Corps of Engineers, EM 1110-2-2100, 1 December 2005 (REF). Although the wall is constructed of dressed granite block masonry instead of concrete, the stability criteria provided in REF are considered appropriate.

2. Load case categories per REF for critical structures:
   Usual Load Condition is Normal Pool
   Unusual Load Condition is Design Flood
   Extreme Load Condition is Seismic Load

3. Overturning criteria and results are expressed in terms of the distance (d) from the toe of the wall base to the resultant vertical force. For the resultant to be within the middle third of the 7-foot-wide base (i.e., entire base in compression), d must be at least 2.33 feet. For the resultant to be within the middle half of the base, d must be at least 1.75 feet. For the resultant to be within the base, d must be at least 0.

TABLE 3 - RESULTS of EXTERNAL (LOCAL) STABILITY ANALYSES
Spillway Section
Dells Pond Dam
Littleton, New Hampshire

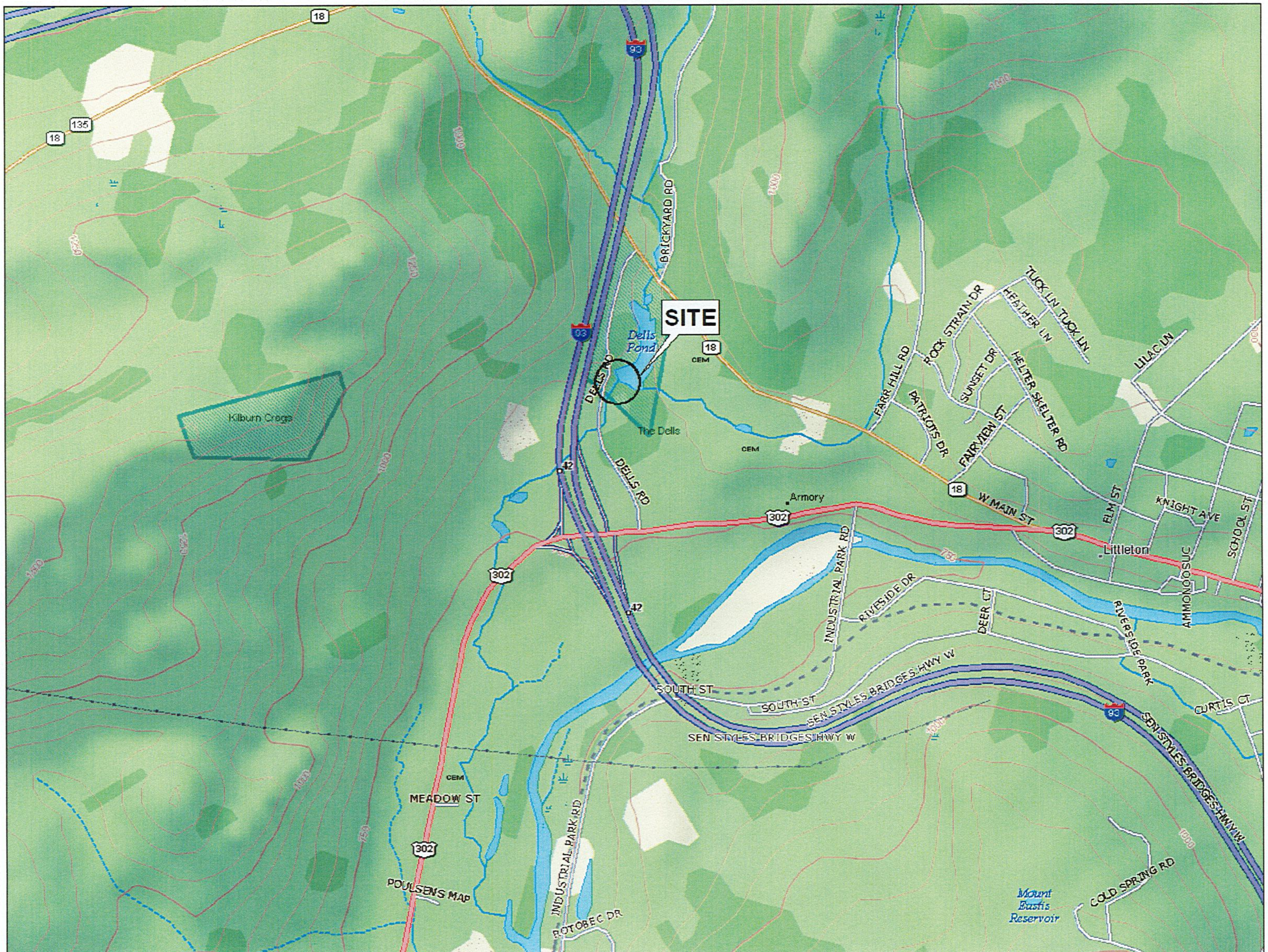
LOAD CASE	SLIDING Factor of Safety FS_{min}	FS	OVERTURNING d_{min} (feet)	d (feet)
Normal Pool	1.7	1.8	3.93	9.5
Design Flood - see Note 5	1.3	**1.16**	2.95	3.2
Seismic Load	1.1	1.6	0.00	7.8

Notes:

1. Minimum sliding and overturning stability criteria are provided in Stability Analysis of Concrete Structures, U.S. Army Corps of Engineers, EM 1110-2-2100, 1 December 2005 (REF). Although the wall is constructed of dressed granite block masonry instead of concrete, the stability criteria provided in REF are considered appropriate.
2. Load case categories per REF for critical structures:
   Usual Load Condition is Normal Pool
   Unusual Load Condition is Design Flood
   Extreme Load Condition is Seismic Load
3. Overturning criteria and results are expressed in terms of the distance (d) from the toe of the wall base to the resultant vertical force. For the resultant to be within the middle third of the assumed 11.8-foot-wide base (i.e., entire base in compression), d must be at least 3.93 feet. For the resultant to be within the middle half of the base, d must be at least 2.95 feet. For the resultant to be within the base, d must be at least 0.
4. Stability analyses of the spillway include resistance provided by interface friction between the 20-foot-long spillway walls and the backfill against the spillway walls.
5. The Design Flood Case does not meet the sliding stability criteria for "Unusual Load Condition" (Fs_{min} = 1.3), but does meet sliding stability criteria for the "Extreme Load Condition" (Fs_{min} = 1.1). An additional horizontal restraining force of approximately 45 kips would be needed to meet the sliding stability criteria for the "Unusual Load Condition". This could be accomplished by placing a concrete berm with a submerged weight of 4.1 kip per foot of spillway on the upstream or downstream face of the spillway. Soil anchors could also be considered to increase sliding stability.

Figures

Ward Geotechnical
Consulting, PLLC



MN (15.1° W)

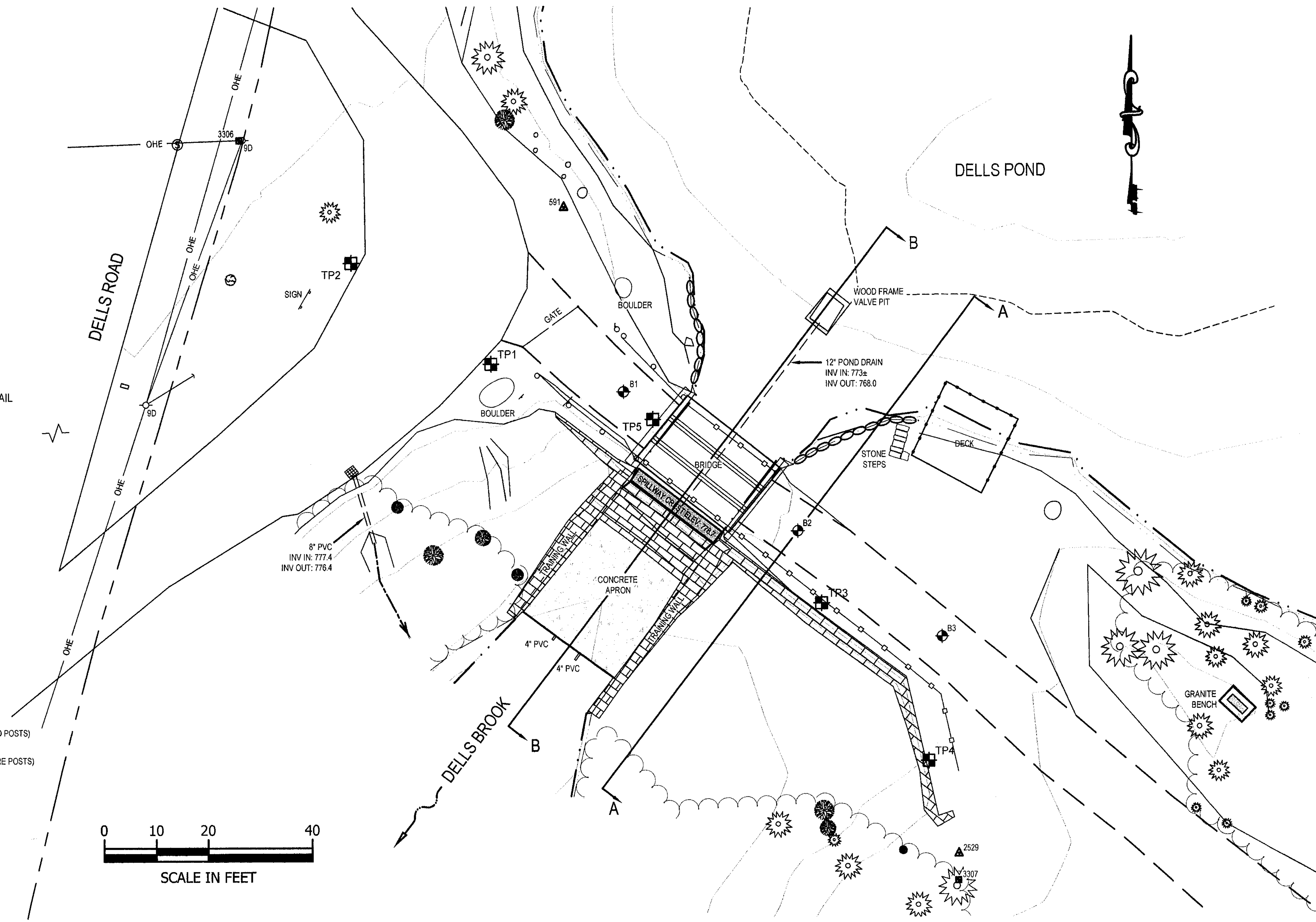
0 400 800 1200 1600 2000 2400 ft

Data Zoom 13-6

DuBois & King, Inc. Laconia, New Hampshire	DELLS POND DAM LITTLETON, NEW HAMPSHIRE	SITE LOCATION MAP
Ward Geotechnical Consulting, PLLC	WGC Project 14830	December 2015 Figure 1

LEGEND - EXISTING FEATURES

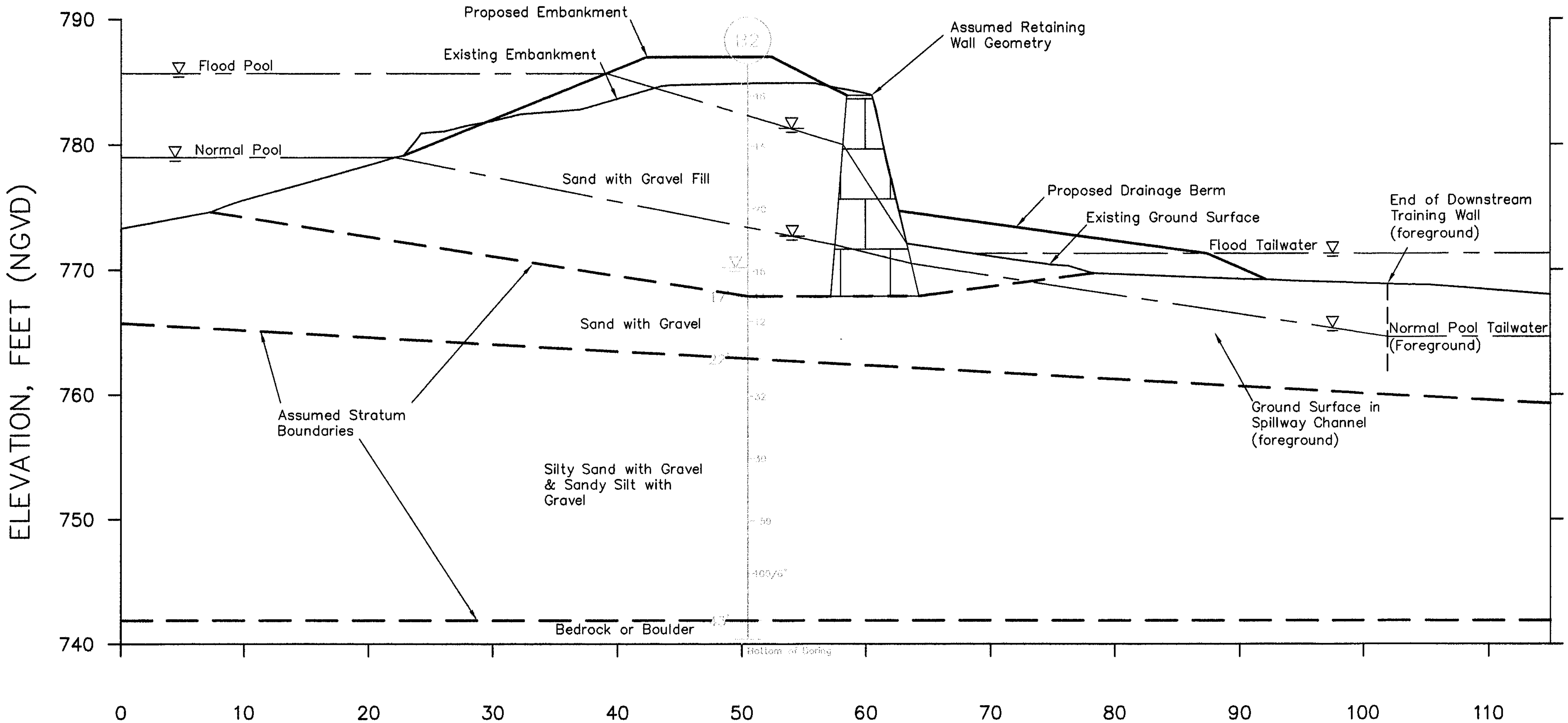
- 1' CONTOUR
- 5' CONTOUR
- PROPERTY LINE (APPROXIMATE LOCATION)
- WETLAND BOUNDARY
- EDGE OF WATER AT TIME O SURVEY (POND STAGE: 779.)
- STONE MASONRY
- CONCRETE
- ASPHALT PAVEMENT
- EDGE MIXED GRASS/GRAVEL TRAIL
- BOULDER WALL
- TREELINE
- SOFTWOOD TREE
- HARDWOOD TREE
- STUMP
- UTILITY POLE
- GUY WIRE
- OHE — OVERHEAD WIRES
- CATCH BASIN
- DRAINAGE PIPE
- DRAINAGE SWALE
- WOOD FENCE/GUARDRAIL (ROUND POSTS)
- WOOD FENCE/GUARDRAIL (SQUARE POSTS)
- SI N
- CONTROL POINT
- BENCHMARK
- B1 GEOTECHNICAL BORING
- TP1 TEST PIT



Notes:

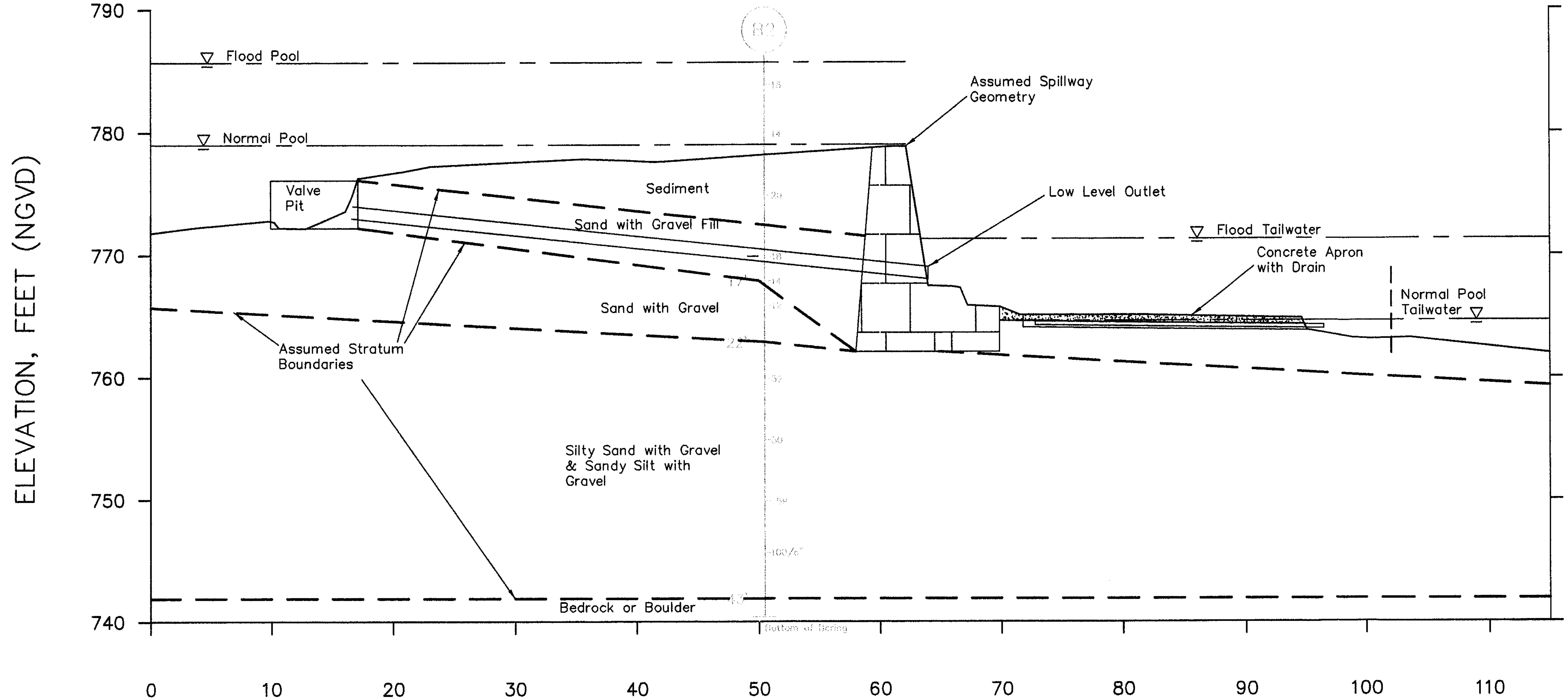
1. The borings were drilled by New Hampshire Boring, Inc. and observed by Ward Geotechnical Consulting, PLLC (WGC) from June 1 through June 3, 2015.
2. The test pits were excavated by the Town of Littleton and observed by WGC and DuBois & King, Inc. on September 17, 2015.
3. Base plan was provided by Headwaters Hydrology, PLLC in AutoCAD format for our use in preparing this boring location plan. Elevations are referenced to the National Geodetic Vertical Datum of 1929.

DuBois & King, Inc. Laconia, New Hampshire	DELLS POND DAM LITTLETON, NEW HAMPSHIRE	BORING & TEST PIT LOCATION PLAN
Ward Geotechnical Consulting, PLLC	WGC Project 14830	December 2015 Figure 2



SECTION A

DuBois & King, Inc. Laconia, New Hampshire	DELLS POND DAM LITTLETON, NEW HAMPSHIRE	SECTION A - MAXIMUM EMBANKMENT RETAINING WALL SECTION
Ward Geotechnical Consulting, PLLC	WGC Project 14830	December 2015 Figure 3



SECTION B

DuBois & King, Inc. Laconia, New Hampshire	DELLS POND DAM LITTLETON, NEW HAMPSHIRE	SECTION B - SPILLWAY
Ward Geotechnical Consulting, PLLC	WGC Project 14830	December 2015 Figure 4

Appendix A

Ward Geotechnical

Consulting, PLLC

Ward Geotechnical Consulting, PLLC

Project:	Dells Pond Dam
Location:	Littleton, New Hampshire
Client:	DuBois & King, Inc.
Project No.:	14830

Boring Log B1

Contractor:	New England Boring Contractors
Logged By:	Craig Ward
Drilling Dates:	6/3/2015
Drill Rig:	Mobile B-47 Truck
Groundwater Depth:	not measured
Date:	
GS Elevation:	785.5 feet
Datum:	NGVD
Boring Location:	Right of spillway - See Boring Location Plan

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	STRATUM
0	S1	4-4 6-7	24	18	4" Case & Wash Drove casing to 4' and drilled to 4'.	S1: 0-8": **Sand with Gravel (SP)** - fine to medium (some coarse) sand, 10%-20% subrounded & subangular gravel to 1/2", brown. 8"-10": **Silty Sand (SM)** - fine to medium sand, 10%-20% nonplastic fines, occasional subrounded gravel to 1/2", light brown. 10"-15": **Sandy Silt (ML)** - nonplastic fines, 10%-30% fine to medium sand, olive-brown. 15"-18": **Sand (SP)** - fine to medium sand, light brown.	Fill
5	S2	4-4 3-3	24	2	Poor recovery - probably pushed cobble with spoon. Pushed casing with head to ~6', then drove to 9'. Drilled to 9' - gravelly below ~7'. Losing water.	S2: **Gravel fragments** coated with fine to coarse Sand	Fill
10	S3	9-7 7-18	24	7	Drove casing to 14' and drilled to 14' - gravelly soils.	S3: **Sand with Gravel (SW)** - fine to corase sand, 20%-30% subangular gravel to 1/2", brown.	Fill (~14')
15	S4	8-6 3-4	24	8	Drove casing & drilled to 16'.	S4: **Sand with Gravel (SW)** - fine to coarse sand, 15%-25% subangular gravel to 1/2", brown & gray. Appears to be mixture of fill and natural soil.	Sand with Gravel
	S5	9-12 10-7	24	11	Drove casing & drilled to 18'. Added filter sand & pulled casing back to 16' for permeability test. $K = 7.3 \times 10^{-4}$ cm/s	S5: **Sand with Gravel (SW)** - fine to coarse sand, 15%-25% subrounded & subangular gravel to 1/2", gray.	Sand with Gravel
20	S6	5-4 8-8	24	6	Drove casing & drilled to 19'. Drove casing & drilled to 24' - gravelly soils.	S6: **Sand with Gravel (SW)** - similar to S5.	Sand with Gravel (~22.5')
25	S7	11-14 13-12	24	10	Drove casing & drilled to 29' - gravelly soils.	S7: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 10%-20% nonplastic fines, 20%-30% subangular gravel to 3/4" (some weathered), green-gray.	Silty Sand with Gravel

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project:	Dells Pond Dam	Boring Log
	Location:	Littleton, New Hampshire	**B1**
	Client:	DuBois & King, Inc.	
	Project No.:	14830	

Contractor:	New England Boring Contractors	Groundwater Depth:	not measured	Date:	Page 2 of 2
Logged By:	Craig Ward				
Drilling Dates:	6/3/2015	GS Elevation:	785.5 feet	Boring Location:	Right of spillway - See Boring Location Plan
Drill Rig:	Mobile B-47 Truck	Datum:	NGVD		

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
30	S8	15-16 18-14	24	11			S8: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 10%-20% nonplastic fines, 30%-40% subangular gravel to 3/4" (some weathered), gray & olive.	Silty Sand with Gravel
					Drove casing to 34' and drilled to 34' - gravelly soils.			
35	S9	17-39 61/0"	24	12	Drilled ahead of casing.		S9: **Silty Sand with Gravel (SM)** - fine (some medium & coarse) sand, 15%-25% nonplastic fines, 25%-35% subangular gravel to 1" (some weathered) olive & gray.	~35.7'
					Drilled in bedrock or boulder from 35.7' to 37.7'.		**Bedrock or Boulder**	Bedrock or Boulder
40							Bottom of Boring at 37.7' Backfilled with cement grout by tremie. Added soil cuttings through grout column as casing was removed.	
45								
50								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project: Dells Pond Dam Location: Littleton, New Hampshire Client: DuBois & King, Inc. Project No.: 14830	Boring Log **B2**
Contractor: New England Boring Contractors Logged By: Craig Ward Drilling Dates: 6/2/2015 Drill Rig: Mobile B-47 Truck	Groundwater Depth: 14.7' below ground surface (measured in well) Date: 6/3/15 at 7:45 am GS Elevation: 785 feet Datum: NGVD Boring Location: Left of spillway - See Boring Location Plan	Page 1 of 2

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	Stratum
	S1	4-11 7-8	24	15	4" Case & Wash Drove casing to 4'. Casing deflecting on boulders at ~3'. Drilled to 4'.	S1: 0-8": **Sand (SP)** - fine to medium sand, occasional subangular gravel to 1", brown. 8"-13": **Silty Sand (SM)** - fine (some medium) sand, 15%-25% nonplastic fines, olive-brown. 13"-15": **Sand with Silt (SP-SM)** - fine to medium (some coarse) sand, 5%-15% nonplastic fines, occasional subangular gravel to 1/2", brown.	Fill
5	S2	5-5 9-10	24	10	Drove casing and drilled to 9' - gravelly soils.	S2: **Sand with Gravel (SW)** - fine to coarse sand, 10%-20% subangular gravel to 1/2", light brown.	
10	S3	9-10 10-11	24	7	Drove casing and drilled to 14' - gravelly soils.	S3: **Sand with Silt & Gravel (SW-SM)** - fine to coarse sand, 5%-15% nonplastic fines, 20%-30% subangular gravel to 3/4", brown.	
15	S4	12-8 10-12	24	9	Drove casing and drilled to 16' - gravelly soils.	S4: **Sand with Silt & Gravel (SW-SM)** - similar to S3.	
	S5	7-5 9-10	24	6	Drove casing and drilled to 18' - gravelly soils.	S5: 0-4": **Sand with Silt & Gravel (SW-SM)** - similar to S3.	~17'
						4"-6": **Silty Sand with Gravel (SM)** - fine to coarse sand, 10%-20% nonplastic fines (possibly some organic), 20%-30% subangular gravel to 3/4", dark gray.	Sand with Gravel
20	S6	10-6 6-14	24	8	Drove casing & drilled to 20'. Added filter sand & pulled casing back to 18' for permeability test. $K = 1.2 \times 10^{-3}$ cm/s Drove casing and drilled to 24' - gravelly soils.	S6: **Sand with Gravel (SW)** - fine to coarse sand, 20%-30% subangular & subrounded gravel to 1/2", gray.	~22'
25	S7	16-16 16-16	24	10	Drove casing and drilled to 29' - gravelly soils.	S7: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 10%-20% nonplastic fines, 25%-35% subangular gravel to 3/4", 1" lens of silt in middle, dark gray.	Silty Sand with Gravel

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project:	Dells Pond Dam	Boring Log
	Location:	Littleton, New Hampshire	**B2**
	Client:	DuBois & King, Inc.	
	Project No.:	14830	

Contractor:	New England Boring Contractors	Groundwater Depth: 14.7' below ground surface (measured in well)	Date: 6/3/15 at 7:45 am	
Logged By:	Craig Ward			
Drilling Dates:	6/2/2015	GS Elevation: 785 feet	Boring Location: Left of spillway - See Boring Location Plan	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD		

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	STRATUM
					Drilled ahead of casing to 29', then drove casing & and redrilled to 29'.		Silty Sand with Gravel and Sandy Silt with Gravel
30	S8	12-13 17-47	24	9		S8: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 10%-20% nonplastic fines, 25%-35% subangular gravel to 3/4", dark gray.	
					Drilled ahead of casing to 34', then drove casing & and redrilled to 34'.		
35	S9	23-32 27-43	24	11		S9: **Silty Sand with Gravel (SM)** - similar to S8, except olive and dark gray.	
					Drilled ahead of casing to 39', then drove casing & redrilled to 39'.		
	S10	100/6"	6	6		S10: **Sandy Silt with Gravel (ML)** - nonplastic fines, 30%-40% fine sand, 10%-15% subangular gravel to 1/2", green-gray.	
40							
					Drilled ahead of casing with roller bit. Drilled into bedrock or boulder from 43' to 44.5'.		~43'
						Bedrock or Boulder	Bedrock or Boulder
45						Bottom of Boring at 44.5' Installed 2" PVC observation well to 41.9': - 10' Screen - 31.9' riser - filter sand from bottom of borehole to 2' above screen - bentonite chip seal to ~3' above filter sand - soil cuttings above bentonite chip seal. - grouted in roadbox at surface.	
50							

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Dells Pond Dam	Boring Log	B3
Location:	Littleton, New Hampshire		
Client:	DuBois & King, Inc.		
Project No.:	14830		

Contractor:	New England Boring Contractors	Groundwater Depth:	Date: see note
Logged By:	Craig Ward	GS Elevation: 785 feet	Boring Location:
Drilling Dates:	6/1/2015	Datum: NGVD	Left End of Embankment - See Boring Location Plan
Drill Rig:	Mobile B-47 Truck		

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	
	S1	3-3 5-10	24	18	4" Case & Wash Drove casing and drilled to 4'.	S1" 0-3": **Silty Sand (SM)** (topsoil) - fine to medium sand, 15%-25% nonplastic fines (some organic), occasional rounded gravel to 3/8", brown. 3"-18": **Sand with Silt (SP-SM)** - fine to medium sand, 5%-15% nonplastic fines, occasional subrounded gravel to 1/2", occasional silt clumps, brown.	Fill
5	S2	6-5 4-4	24	0	No recovery - probably pushed cobble with spoon. Drove spoon again ~3' to get sample (Rec ~4") Drove casing and drilled to 4' - gravelly soils. Losing water.	S2 (second spoon): **Sand (SW)** - fine to coarse sand, brown.	Fill
10	S3	2-2 3-1	24	2	Poor recovery - probably pushed cobble with spoon. Drove casing and drilled to 11'.	S3: **Sand with Silt & Gravel (SW-SM)** - fine to coarse sand, 5%-15% nonplastic fines, 15%-25% subangular gravel to 3/4", brown. Rock fragment and stick in tip of spoon.	Fill
	S4	4-3 2-3	24	2	Poor recovery - probably pushed cobble with spoon. Drove casing and drilled to 14' - two sticks in wash.	S4: **Sand with Gravel (SW)** - fine to coarse sand, 20%-30% subangular gravel to 1/2", brown.	Fill
15	S5	10-6 4-4	24	0	No recovery - probably pushed cobble with spoon. Drove 3" spoon over same sample interval. Drove casing and drilled to 19' - sand in wash. Losing water.	S5: (3" spoon): **Sand with Gravel (SW)** - similar to S4. Bottom 2" is **Silty Sand (SM)** - fine to medium sand, 30%-40% medium plastic fines (some organic), occasional small sticks, dark brown. Possibly original ground.	~15.5'
20	S6	4-4 5-6	24	11	 Drove casing and drilled to 24'.	S6: **Sand (SP)** - fine, fine to medium, & fine to coarse sand, vague stratification, light gray.	Sand & Silty Sand
25	S7	3-5 7-8	24	9		S7: 0-3": **Sand (SP)** - fine to medium sand, light gray. 3"-6": **Silty Fine Sand (SM)** - fine sand, 10%-20% nonplastic fines, laminated structure, light gray. 6"-9": **Sand (SP)** - fine to medium sand, light gray.	Sand & Silty Sand

Notes: Measured water level in casing (at 44') about 1 hour after rods removed. Water level was 14.7' below the ground surface.

Abbreviations:

- PEN - Penetration length of sampler or core barrel
- REC - Recovery length of sample
- S - Split Spoon Sample
- C - Rock Core Sample
- U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project: Dells Pond Dam Location: Littleton, New Hampshire Client: DuBois & King, Inc. Project No.: 14830		Boring Log **B3**
Contractor: New England Boring Contractors Logged By: Craig Ward Drilling Dates: 6/1/2015 Drill Rig: Mobile B-47 Truck	Groundwater Depth: see note	Date:	Page 2 of 2
	GS Elevation: 785 feet Datum: NGVD	Boring Location: Left End of Embankment - See Boring Location Plan	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	Stratum
					Drove casing and drilled to 29' More casing resistance below 26'.		Sand & Silt ~30'
30	S8	4-10 14-13	24	10		S8: 0-4": **Sand (SW)** - fine to coarse sand, light gray. 4"-6": **Sandy Silt (ML)** - nonplastic fines, 25%-35% fine to medium sand, light gray. 6"-10": **Silty Sand with Gravel (SM)** - fine to coarse sand, 10%-20% nonplastic fines, 30%-40% subangular gravel to 3/4", gray.	
					Drove casing and drilled to 34' - gravelly soils. Casing driving hard.		
35	S9	9-15 22-27	24	9		S9: **Silty Sand with Gravel (SM)** - similar to S8: 6"-10".	
					Drilled ahead of casing to 39', then drove casing to 39' and redrilled. Gravelly soils.		
40	S10	24-31 74-11	24	11		S10: **Silty Sand with Gravel (SM)** - fine to coarse sand, 15%-25% nonplastic fines, 25%-35% subangular gravel to 1', green-gray.	Silty Sand with Gravel
					Drilled ahead of casing to 44', then drove casing and redrilled to 44'. Gravelly.		
	S11	74-26/0.5"	6.5	0		S11: No Recovery	
45					Drilled ahead of casing with roller bit. Drilled into bedrock from 47' to 48' before drilling rock core.		~47'
50	C1		60	56	Cored bedrock at rates ranging from 4.1 to 5.2 minutes per foot.	C1: **Bedrock** - coarse grained igneous or metamorphic rock, hard, mostly green-gray but with white quartz inclusions, joints dipping ~0° and 30° - 45° at spacings ranging from <1" (in fractured upper 1') to 17", fresh to slightly weathered. Probably granodiorite or tonalite. RQD = 45.5/60 = 76%	Bedrock
						Bottom of Boring at 53' Backfilled borehole with cement grout using tremei, then added cuttings.	

Notes: Measured water level in casing (at 44') about 1 hour after rods removed. Water level was 14.7' below the ground surface.

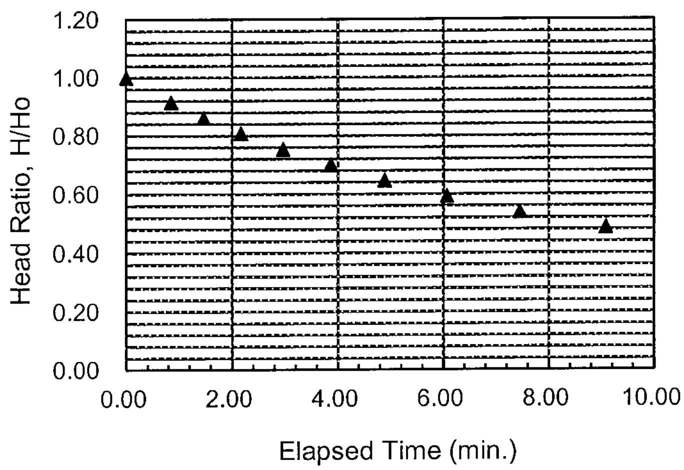
Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

VARIABLE HEAD PERMEABILITY TEST

PROJECT:	Dells Pond Dam			BORING :	B1
LOCATION:	Littleton, New Hampshire			PROJ NO.:	14830
CLIENT:	DuBois & King, Inc.			TEST NO.:	P-1
PERFORMED:	Craig Ward	DATE:	6/3/2015	PAGE NO.:	1
CALCULATED:	Craig Ward	DATE:	6/4/2015	SURFACE	
CHECKED:	Craig Ward	DATE:	06/04/15	ELEV.:	

P-1



STANDPIPE/RISER DETAILS

Casing diameter, d, inches:	4.00
Casing length, feet:	20.00
Casing stick up length, feet (above ground surface) :	4.00
Groundwater depth, Dw, feet (below ground surface):	14.70

TEST INTERVAL DETAILS

Depth to bottom, feet (below ground surface):	18.00
Depth to top, feet (below ground surface):	16.00
Length, L, feet:	2.00
Diameter, D, inches:	4.00

SOIL TYPE:

TIME (hr.min.sec)	ELAPSED TIME (min)	DEPTH-RIM TO WATER (feet)	HEAD H (feet)	HEAD RATIO H/Ho	K (cm/sec)	K (feet/min)
10:00:00 AM	0.00	0.00	18.70	1.00		
10:00:51 AM	0.85	1.60	17.10	0.91	9.2E-04	1.8E-03
10:01:28 AM	1.47	2.60	16.10	0.86	8.6E-04	1.7E-03
10:02:10 AM	2.17	3.60	15.10	0.81	8.0E-04	1.6E-03
10:02:58 AM	2.97	4.60	14.10	0.75	7.5E-04	1.5E-03
10:03:52 AM	3.87	5.60	13.10	0.70	7.2E-04	1.4E-03
10:04:53 AM	4.88	6.60	12.10	0.65	6.8E-04	1.3E-03
10:06:04 AM	6.07	7.60	11.10	0.59	6.4E-04	1.3E-03
10:07:27 AM	7.45	8.60	10.10	0.54	6.0E-04	1.2E-03
10:09:05 AM	9.08	9.60	9.10	0.49	5.6E-04	1.1E-03
	Ave. permeability:				7.3E-04	1.4E-03

NOTES:

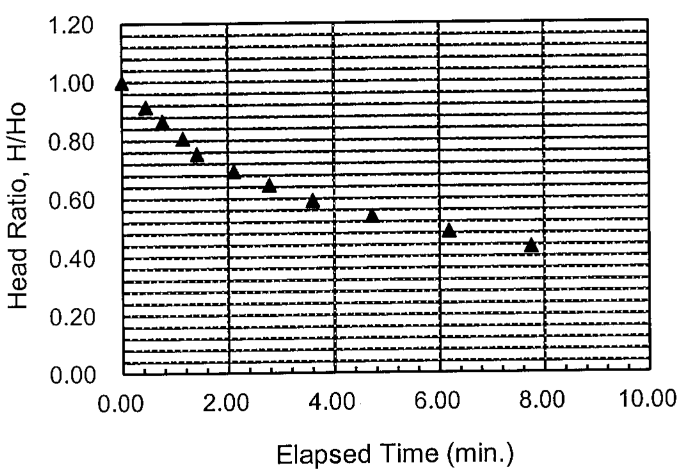
1. Test procedure utilized corresponds to a Falling Head test in an open borehole.
2. Hydraulic Conductivity, K = r²/(2*L)*ln(L/R)/(t2-t1)*ln(H2/H1), (Lambe & Whitman), Soil Mechanics, 1969, pp 285, case G; isotropic conditions, L/D not less than 4.
3. H represents the difference between static groundwater level and water level in casing.

VARIABLE HEAD PERMEABILITY TEST

PROJECT:	Dells Pond Dam		
LOCATION:	Littleton, New Hampshire		
CLIENT:	DuBois & King, Inc.		
PERFORMED:	Craig Ward	DATE:	6/2/2015
CALCULATED:	Craig Ward	DATE:	6/4/2015
CHECKED:	Craig Ward	DATE:	06/04/15

P-1

BORING :	B2
PROJ NO.:	14830
TEST NO.:	P-1
PAGE NO.:	1
SURFACE ELEV.:	



STANDPIPE/RISER DETAILS

Casing diameter, d, inches:	4.00
Casing length, feet:	22.00
Casing stick up length, feet (above ground surface) :	4.00
Groundwater depth, Dw, feet (below ground surface):	14.70

TEST INTERVAL DETAILS

Depth to bottom, feet (below ground surface):	20.00
Depth to top, feet (below ground surface):	18.00
Length, L, feet:	2.00
Diameter, D, inches:	4.00

SOIL TYPE:

TIME (hr.min.sec)	ELAPSED TIME (min)	DEPTH-RIM TO WATER (feet)	HEAD H (feet)	HEAD RATIO H/Ho	K (cm/sec)	K (feet/min)
10:00:00	0.00	0.00	18.70	1.00		
10:00:27	0.45	1.60	17.10	0.91	1.7E-03	3.4E-03
10:00:46	0.77	2.50	16.20	0.87	1.5E-03	2.9E-03
10:01:09	1.15	3.60	15.10	0.81	1.6E-03	3.2E-03
10:01:25	1.42	4.60	14.10	0.75	2.3E-03	4.4E-03
10:02:07	2.12	5.70	13.00	0.70	1.0E-03	2.0E-03
10:02:47	2.78	6.60	12.10	0.65	9.4E-04	1.9E-03
10:03:36	3.60	7.60	11.10	0.59	9.3E-04	1.8E-03
10:04:43	4.72	8.60	10.10	0.54	7.4E-04	1.5E-03
10:06:11	6.18	9.60	9.10	0.49	6.2E-04	1.2E-03
10:07:45	7.75	10.60	8.10	0.43	6.5E-04	1.3E-03
	Ave. permeability:				1.2E-03	2.4E-03

NOTES:
1. Test procedure utilized corresponds to a Falling Head test in an open borehole.
2. Hydraulic Conductivity, K = r²/(2*L)*ln(L/R)/(t2-t1)*ln(H2/H1), (Lambe & Whitman), Soil Mechanics, 1969, pp 285, case G; isotropic conditions, L/D not less than 4.
3. H represents the difference between static groundwater level and water level in casing.

Ward Geotechnical
Consulting, PLLC

Appendix B

DuBois & King inc.

TEST PIT LOG

Test Pit No.: 1

Project: Dells Pond Dam

Project No.: 622821L

Date: 9/17/2015

By: R. Durfee

Location: Littleton, NH

Surface Elevation: 786.8

Groundwater Elevation: None

Depth From	Depth To	Description of Material	Elevation
0"	1' - 0"	Loam	
1' - 0"	1' - 8"	Brown Sandy Gravel	
1' - 8"	4' - 0"	Grey Sand	

TEST PIT LOG

Test Pit No.: 2

Project: Dells Pond Dam

Project No.: 622821L

Date: 9/17/2015

By: R. Durfee

Location: Littleton, NH

Surface Elevation: 787.8

Groundwater Elevation: None

Depth From	Depth To	Description of Material	Elevation
0"	0' - 10"	Loam	
0' - 10"	4' - 0"	Fine to medium sand	

TEST PIT LOG

Test Pit No.: 3

Project: Dells Pond Dam

Project No.: 622821L

Date: 9/17/2015

By: R. Durfee

Location: Littleton, NH

Surface Elevation: 784.6

Groundwater Elevation: None

Depth From	Depth To	Description of Material	Elevation
0"	0' - 8"	Loam	
0' - 8"	3' - 8"	Sand, silty sand	

DuBois & King inc.

TEST PIT LOG

Test Pit No.: 4

Project: Dells Pond Dam

Project No.: 622821L

Date: 9/17/2015

By: R. Durfee

Location: Littleton, NH

Surface Elevation: 783.0

Groundwater Elevation: None

Depth From	Depth To	Description of Material	Elevation
0"	0' - 8"	Loam	
0' - 8"	4' - 0"	Sand and gravel	

DuBois & King inc.

TEST PIT LOG

Test Pit No.: 5

Project: Dells Pond Dam

Project No.: 622821L

Date: 9/17/2015

By: R. Durfee

Location: Littleton, NH

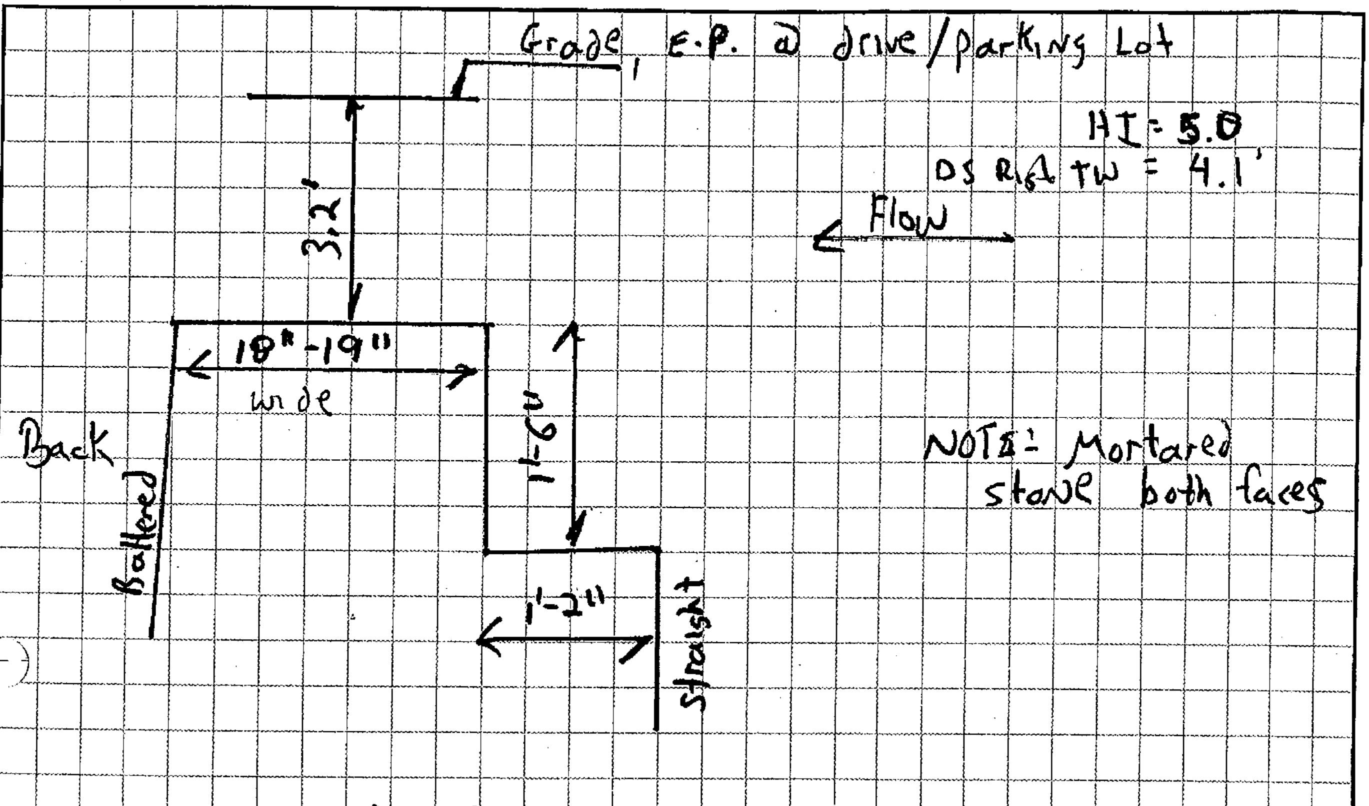
Surface Elevation: 785.0

Groundwater Elevation: None

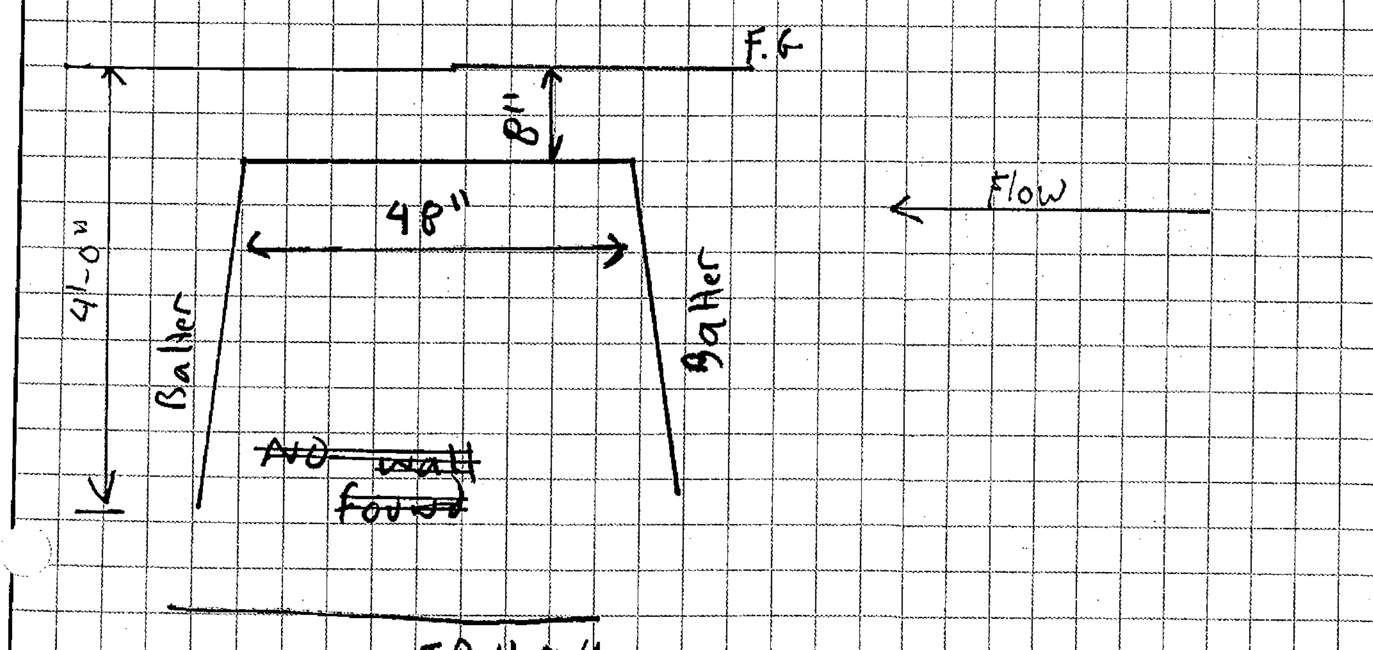
Depth From	Depth To	Description of Material	Elevation
0"	0' - 8"	Loam	
0' - 8"	2' - 6"	Sand and gravel	

DUBOIS & KING, INC.
www.dubois-king.com
Engineering * Planning
Development * Management

JOB: Littleton, NH Dells Pond 622821L
SHEET NO. 1 OF 3
CALCULATED BY: RNO DATE: 9/17/15
CHECKED BY: DATE:
SCALE:



Wall @ TP #1



TP #4

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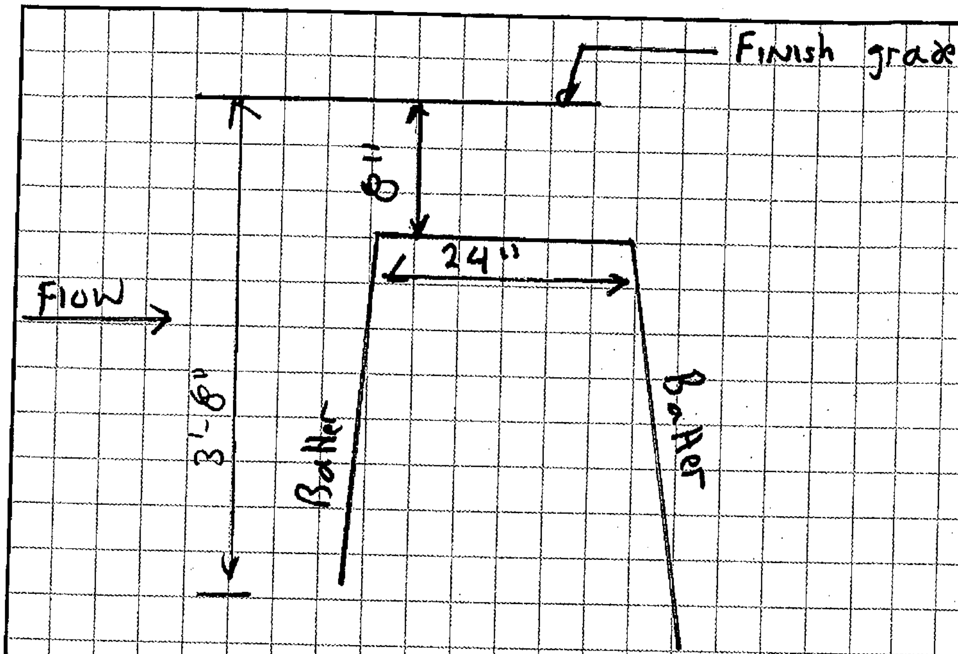
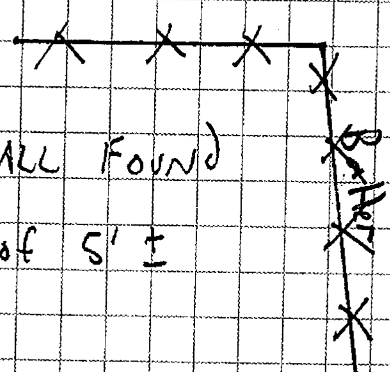
JOB Littleton, NH - Bells Pond 622821L
SHEET NO. 2 OF 3
CALCULATED BY RHO DATE 9/17/15
CHECKED BY ______ DATE ______
SCALE ______



wall @ T.P # 3



NO WALL FOUND

AT Depth of 5' ±

NO wall @ T.P # 2

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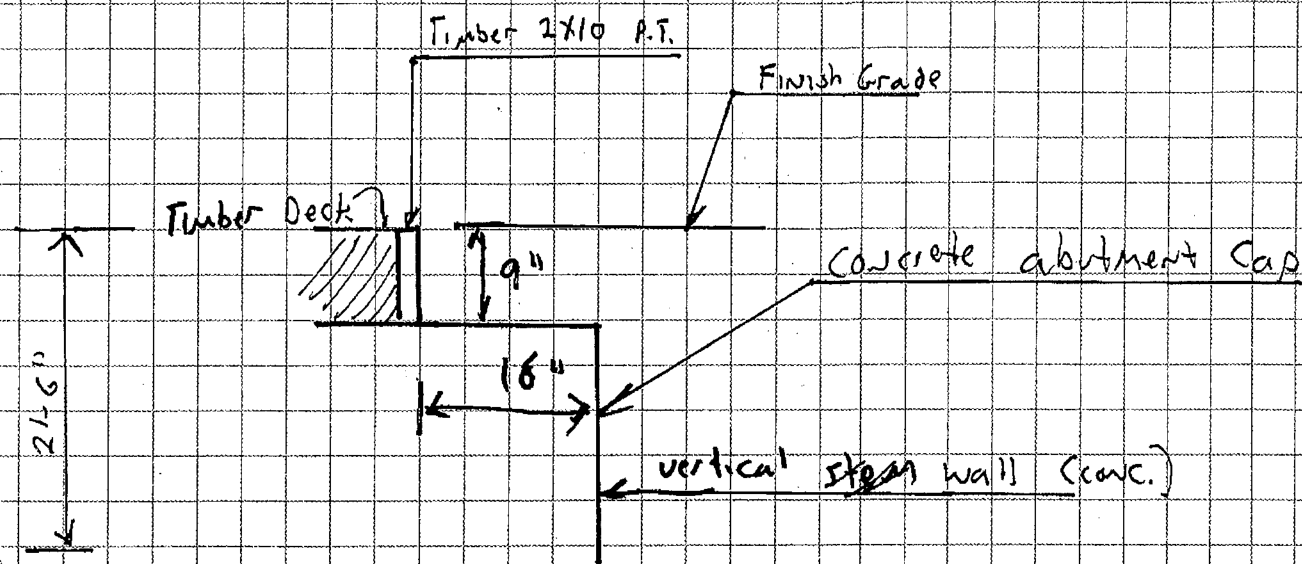
Development * Management

JOB: Dells Pond Dam - Littleton, NH 622821C
SHEET NO. 3 OF 3
CALCULATED BY: RND DATE: 9/17/15
CHECKED BY: DATE:
SCALE:



[redacted]

Test Pit #5

Behind N.W. Bridge Abutment

Appendix C

Ward Geotechnical
Consulting, PLLC

Figure C.1 - Max Embankment Retaining Wall Section
Existing Conditions
Normal Pool

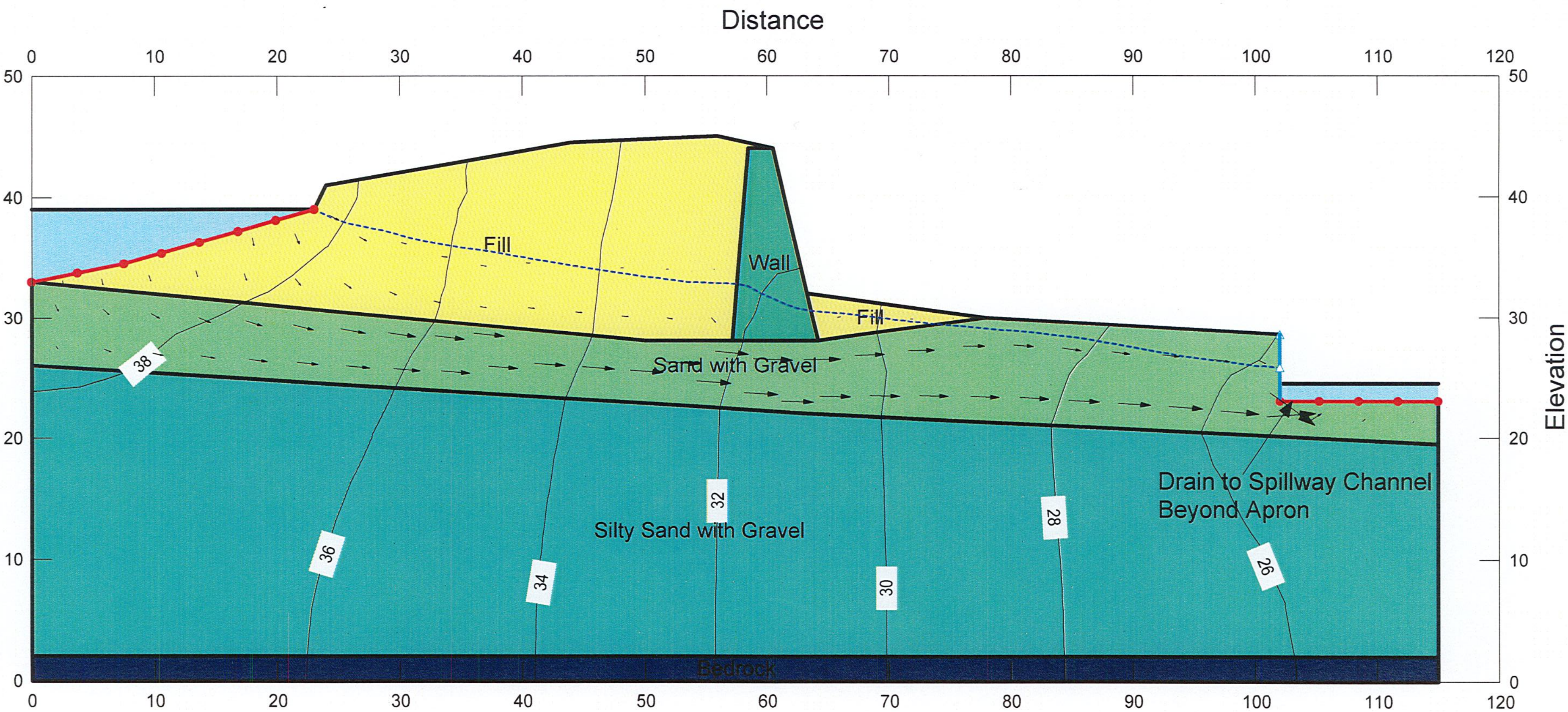


Figure C.2 - Max Embankment Retaining Wall Section
Existing Conditions
Normal Pool
Drain to Apron in Spillway Channel

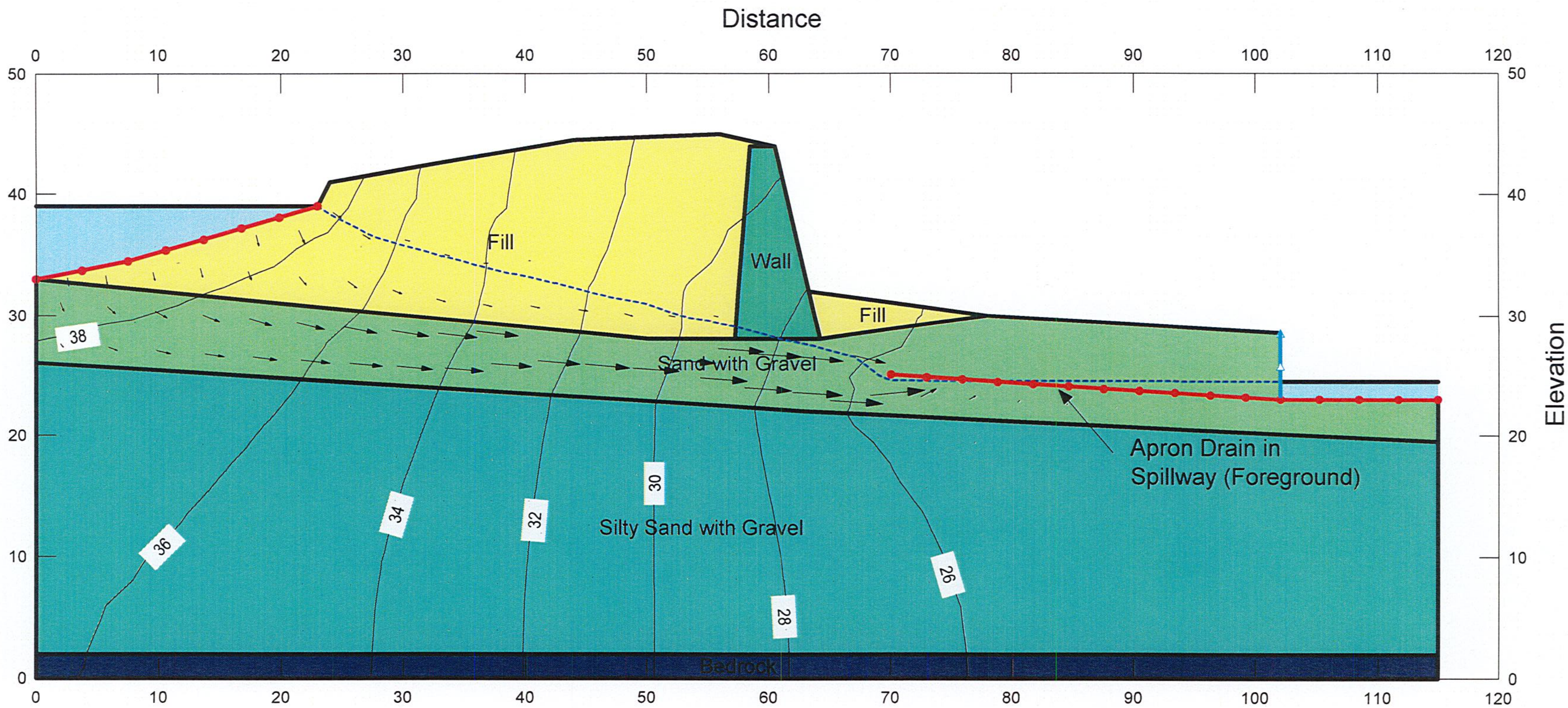


Figure C.3 - Max Embankment Retaining Wall Section
Proposed Conditions with Berm
Design Flood

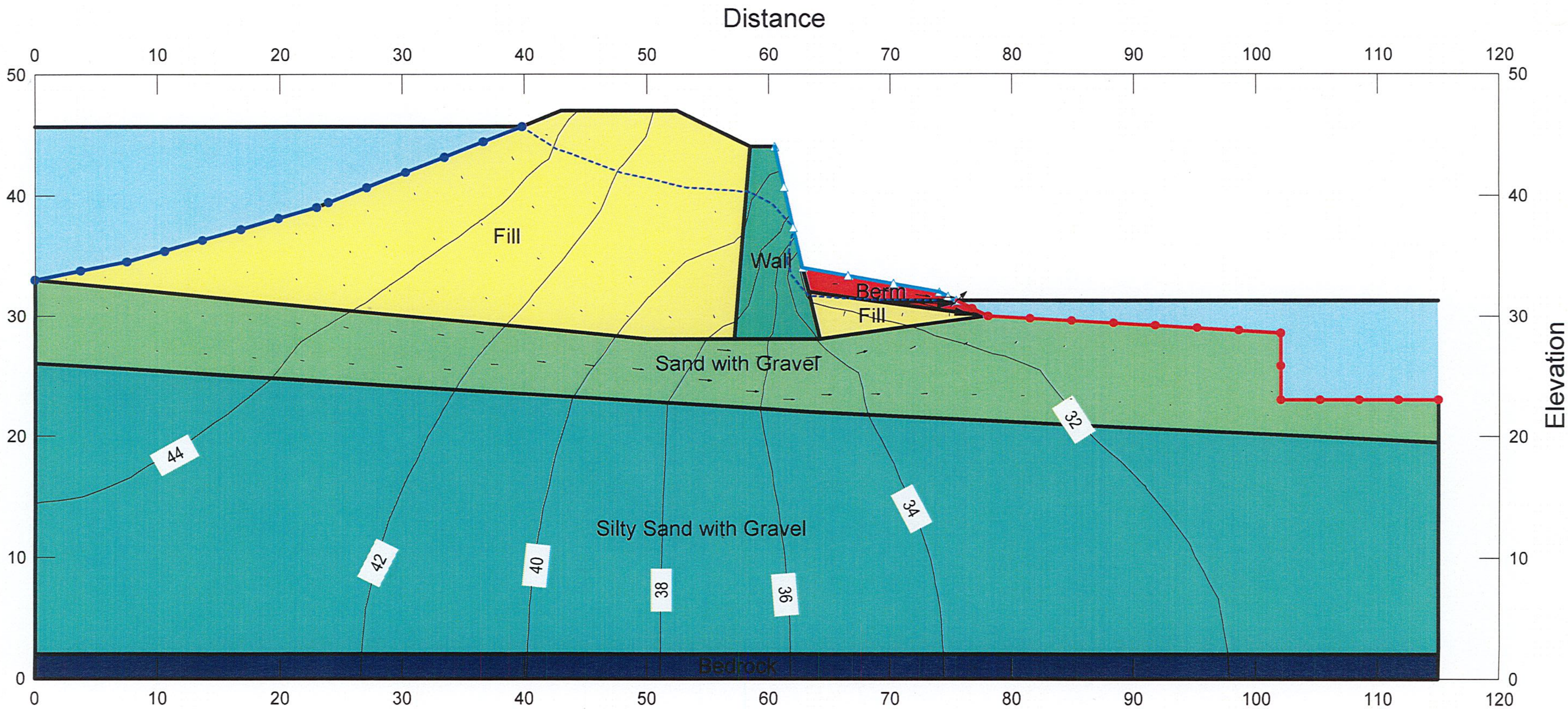


Figure C.4 - Spillway Section
Normal Pool

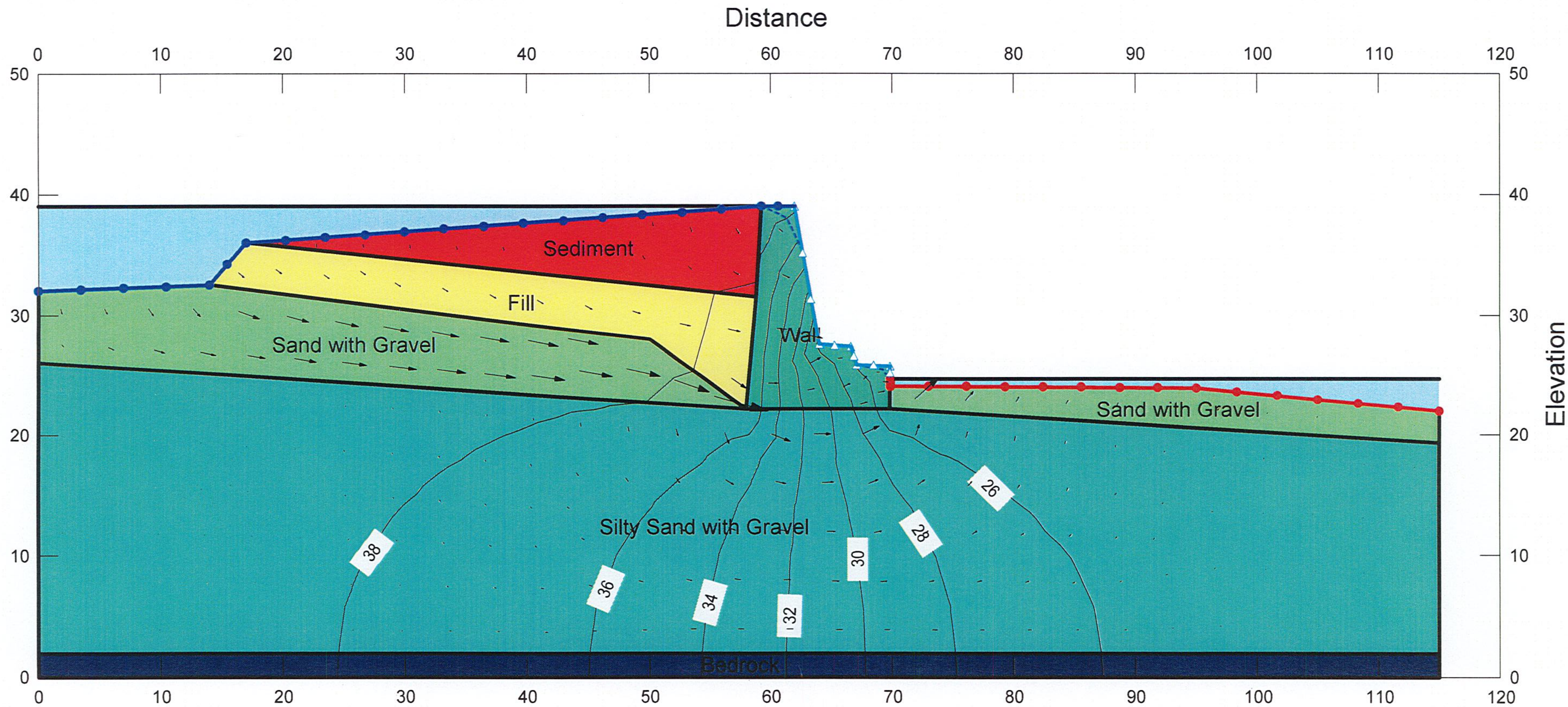
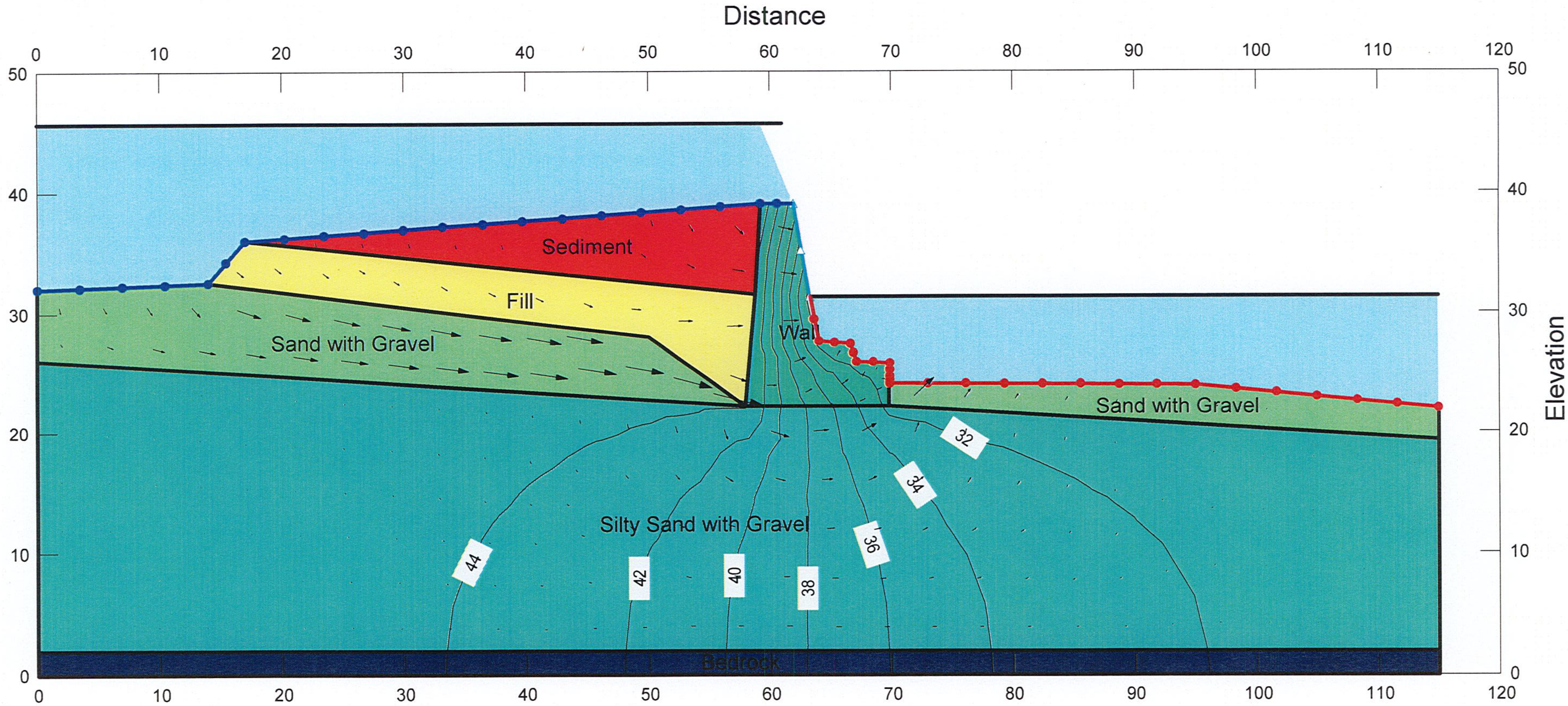


Figure C.5 - Spillway Section
Design Flood



Appendix D

Ward Geotechnical
Consulting, PLLC

Figure D.1 - Max Embankment Section
Existing Conditions
Normal Pool

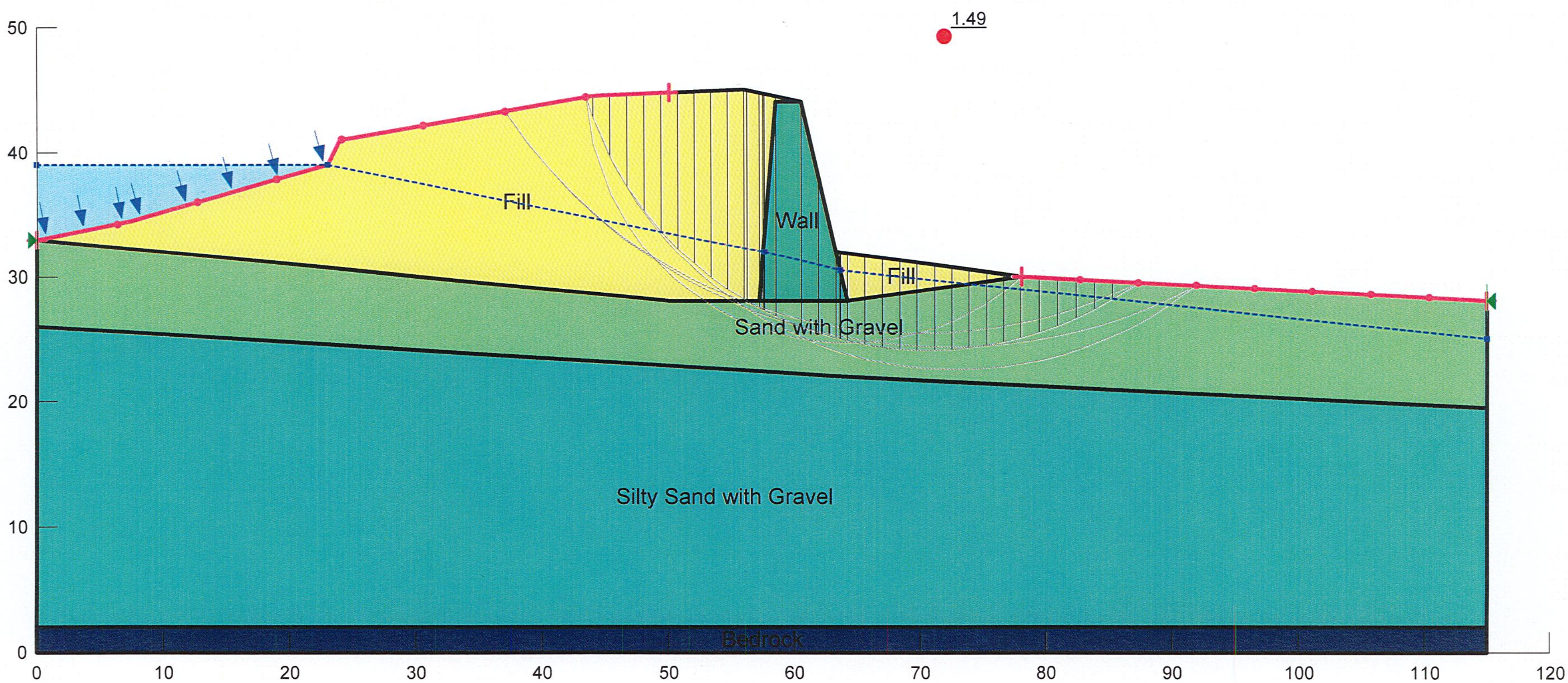


Figure D.2 - Max Embankment Retaining Wall Section
Proposed Conditions
Normal Pool

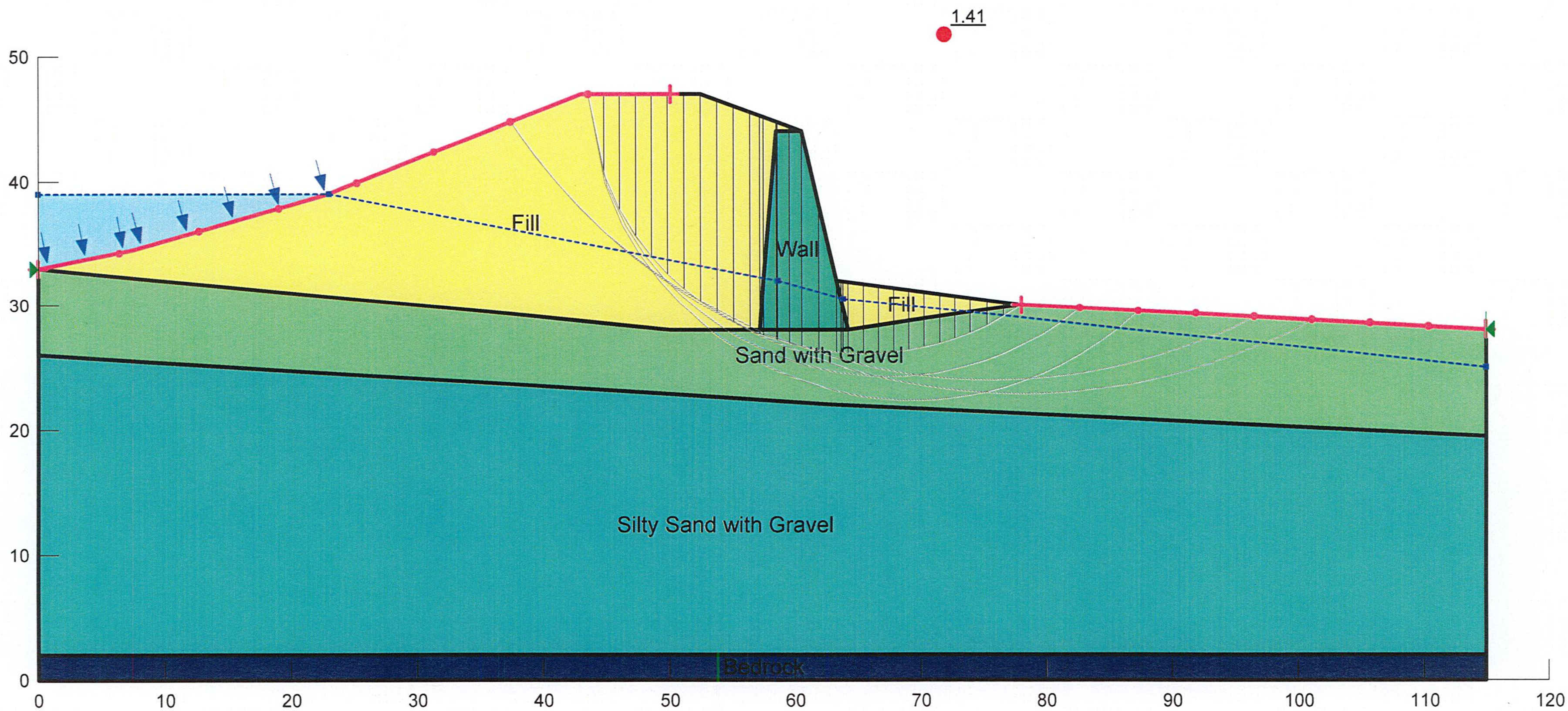


Figure D.3 - Max Embankment Retaining Wall Section
Proposed Conditions with 2.5' x 20' Berm
Normal Pool

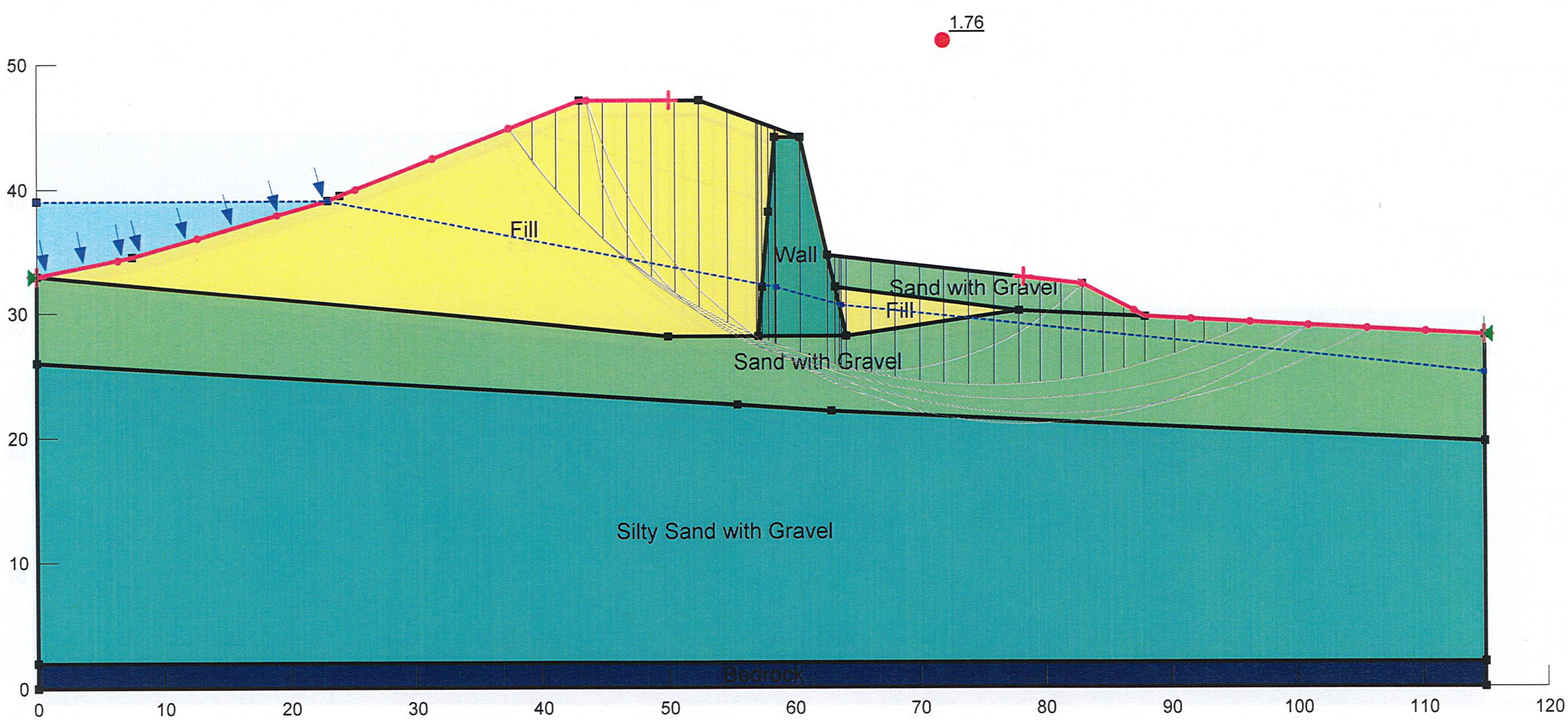


Figure D.6 - Max Embankment Retaining Wall Section
Proposed Conditions
Seismic: PGA = 0.12g

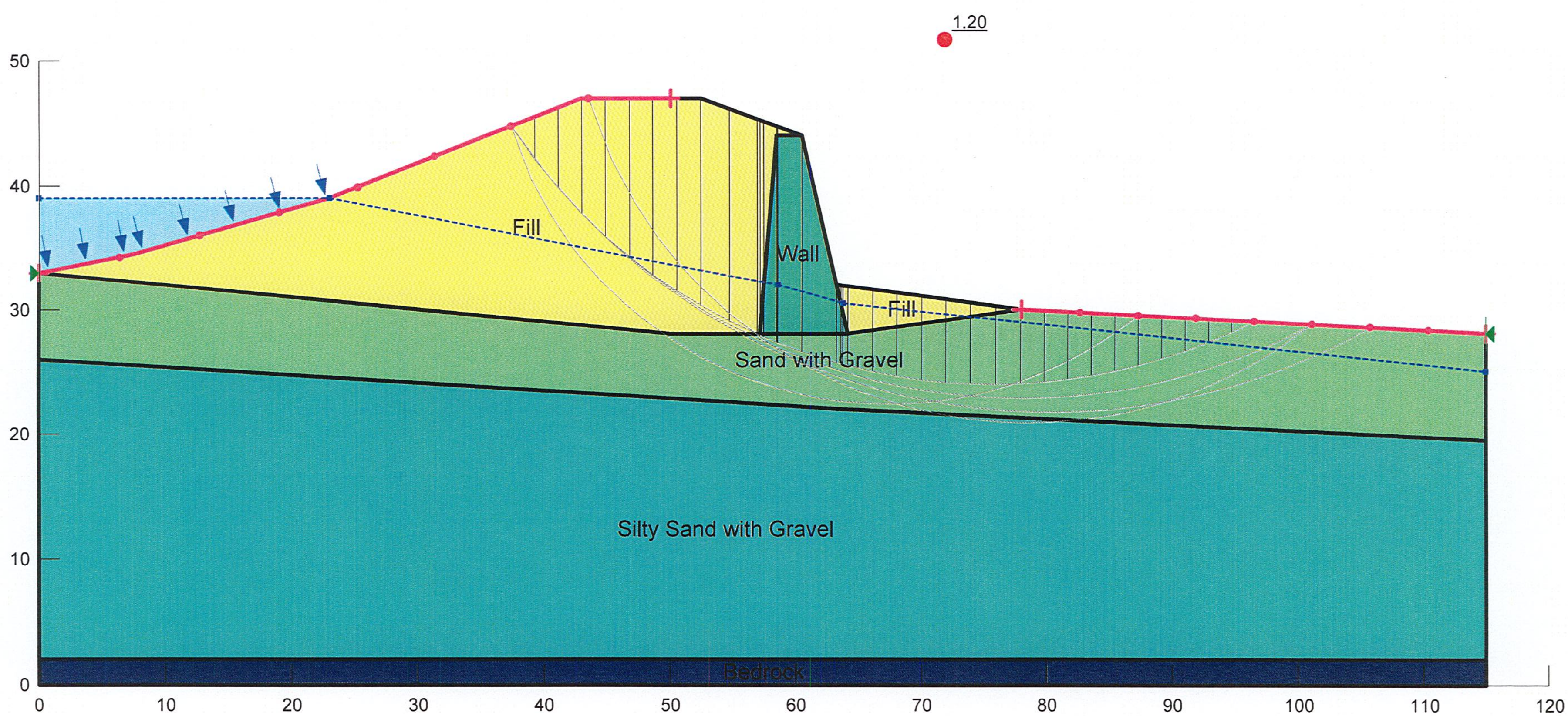
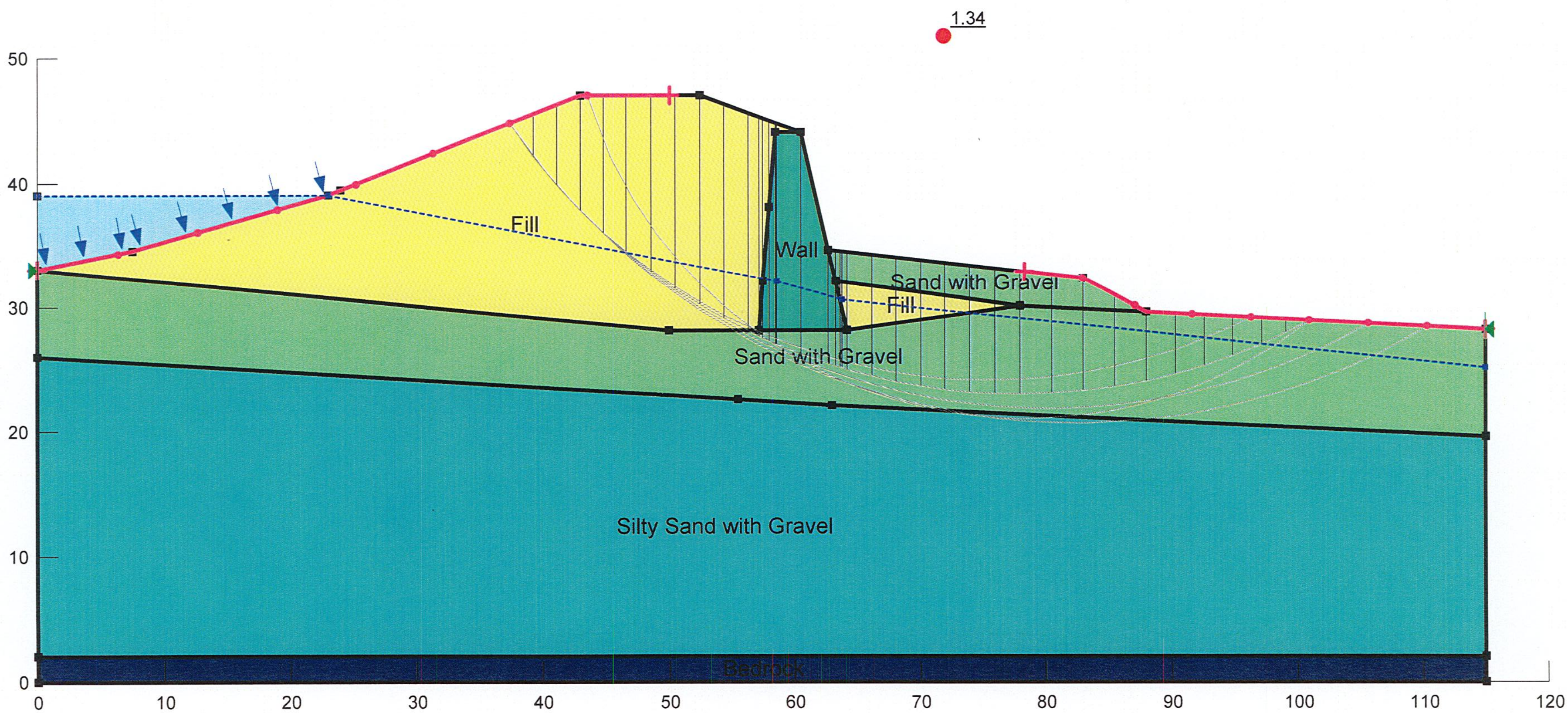
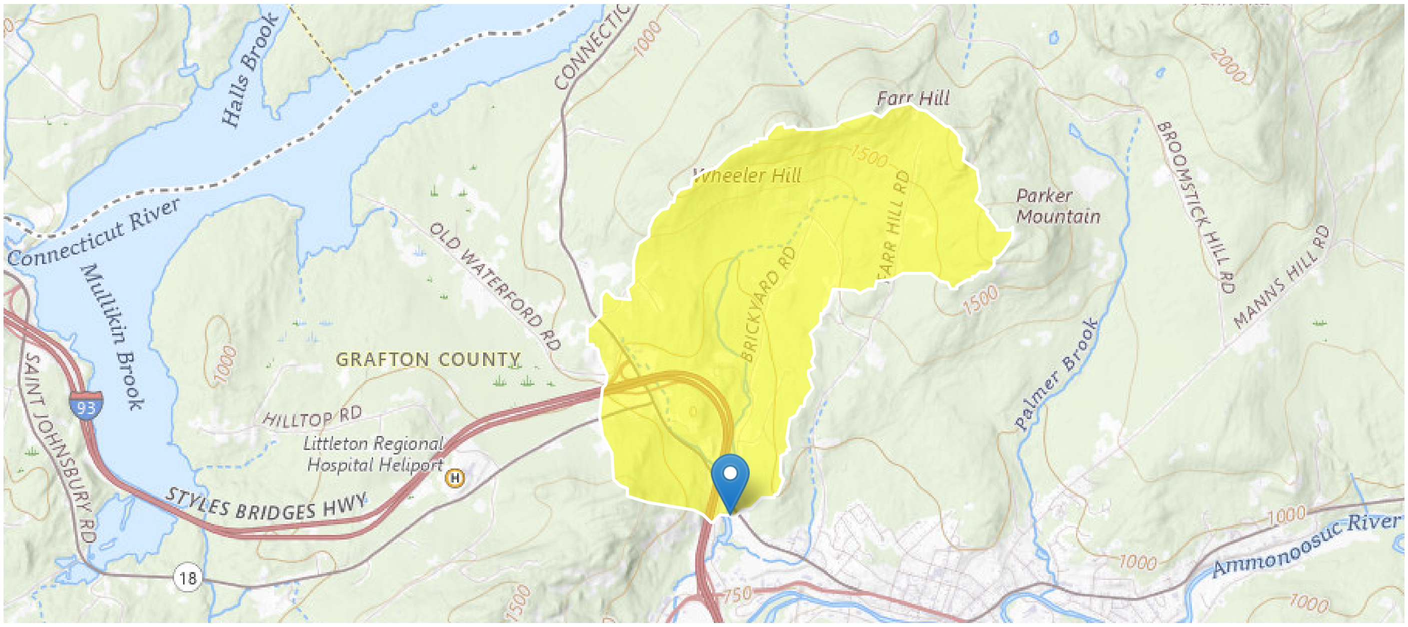


Figure D.7 - Max Embankment Retaining Wall Section
Proposed Conditions with 2.5' x 20' Berm
Seismic: PGA = 0.12g



StreamStats Report #1

Region ID: NH
Workspace ID: NH20250822135315628000
Clicked Point (Latitude, Longitude): 44.31450, -71.79645
Time: 2025-08-22 09:54:03 -0400



Collapse All

Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
APRAVPRE	Mean April Precipitation	2.855	inches
BSLDEM30M	Mean basin slope computed from 30 m DEM	17.107	percent
CENTROIDX	Basin centroid horizontal (x) location in state plane coordinates	951503.5	meters
CENTROIDY	Basin centroid vertical (y) location in state plane units	667799.1	meters
CONIF	Percentaqe of land surface covered by coniferous forest	17.2783	percent
CSL10_85	Change in elevation divided by length between points 10 and 85 percent of distance along main channel to basin divide - main channel method not known	249	feet per mi
DRNAREA	Area that drains to a point on a stream	2.78	square miles
ELEVMAX	Maximum basin elevation	1958.888	feet
LC11DEV	Percentage of developed (urban) land from NLCD 2011 classes 21-24	12	percent
LC11IMP	Average percentage of impervious area determined from NLCD 2011 impervious dataset	2.96	percent
MINTEMP_W	Mean winter minimum air temperature over basin surface area	9.683	degrees F
MIXFOR	Percentage of land area covered by mixed deciduous and coniferous forest	46.3492	percent
OUTLETX	Basin outlet horizontal (x) location in state plane coordinates	950295	feet

Parameter Code	Parameter Description	Value	Unit
OUTLETY	Basin outlet vertical (y) location in state plane coordinates	661395	feet
PREBC0103	Mean annual precipitation of basin centroid for January 1 to March 15 winter period	5.63	inches
PREBC_1112	Mean annual precipitation of basin centroid for November 1 to December 31 period	6.69	inches
PRECIPCENT	Mean Annual Precip at Basin Centroid	38.1	inches
PRECIPOUT	Mean annual precip at the stream outlet (based on annual PRISM precip data in inches from 1971-2000)	37.6	inches
PREG_03_05	Mean precipitation at gaging station location for March 16 to May 31 spring period	7.1	inches
PREG_06_10	Mean precipitation at gaging station location for June to October summer period	18.3	inches
SNOFALL	Mean Annual Snowfall	85.207	inches
TEMP	Mean Annual Temperature	42.439	degrees F
TEMP_06_10	Basinwide average temperature for June to October summer period	59.438	degrees F
WETLAND	Percentage of Wetlands	0.1254	percent

Peak-Flow Statistics

Peak-Flow Statistics Parameters [Peak Flow Statewide SIR2008 5206]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
APRAVPRE	Mean April Precipitation	2.855	inches	2.79	6.23
CSL10_85	Stream Slope 10 and 85 Method	249	feet per mi	5.43	543
DRNAREA	Drainage Area	2.78	square miles	0.7	1290
WETLAND	Percent Wetlands	0.1254	percent	0	21.8

Peak-Flow Statistics Flow Report [Peak Flow Statewide SIR2008 5206]

PIL: Lower 90% Prediction Interval, PIU: Upper 90% Prediction Interval, ASEp: Average Standard Error of Prediction, SE: Standard Error, PC: Percent Correct, RMSE: Root Mean Squared Error, PseudoR^2: Pseudo R Squared (other -- see report)

Statistic	Value	Unit	PIL	PIU	ASEp	Equiv. Yrs.
50-percent AEP flood	103	ft^3/s	62.5	170	30.1	3.2
20-percent AEP flood	163	ft^3/s	97.3	273	31.1	4.7
10-percent AEP flood	214	ft^3/s	125	367	32.3	6.2
4-percent AEP flood	284	ft^3/s	160	505	34.3	8
2-percent AEP flood	341	ft^3/s	186	626	36.4	9
1-percent AEP flood	409	ft^3/s	215	778	38.6	9.8
0.2-percent AEP flood	574	ft^3/s	276	1190	44.1	11

Peak-Flow Statistics Citations

Olson, S.A.,2009, Estimation of flood discharges at selected recurrence intervals for streams in New Hampshire: U.S.Geological Survey Scientific Investigations Report 2008-5206, 57 p. (http://pubs.usgs.gov/sir/2008/5206/)

Flow-Duration Statistics

Flow-Duration Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	3.26	689
PREG_06_10	Jun to Oct Gage Precipitation	18.3	inches	16.5	23.1
TEMP	Mean Annual Temperature	42.439	degrees F	36	48.7

Flow-Duration Statistics Disclaimers [Low Flow Statewide]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Flow-Duration Statistics Flow Report [Low Flow Statewide]

Statistic	Value	Unit
60 Percent Duration	1.47	ft^3/s
70 Percent Duration	1.02	ft^3/s
80 Percent Duration	0.608	ft^3/s
90 Percent Duration	0.32	ft^3/s
95 Percent Duration	0.202	ft^3/s
98 Percent Duration	0.127	ft^3/s

Flow-Duration Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Low-Flow Statistics

Low-Flow Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	3.26	689
PREG_06_10	Jun to Oct Gage Precipitation	18.3	inches	16.5	23.1
TEMP	Mean Annual Temperature	42.439	degrees F	36	48.7

Low-Flow Statistics Disclaimers [Low Flow Statewide]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Low-Flow Statistics Flow Report [Low Flow Statewide]

Statistic	Value	Unit
7 Day 2 Year Low Flow	0.189	ft^3/s
7 Day 10 Year Low Flow	0.0677	ft^3/s

Low-Flow Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Seasonal Flow Statistics

Seasonal Flow Statistics Parameters [Low Flow Statewide]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
BSLDEM30M	Mean Basin Slope from 30m DEM	17.107	percent	3.19	38.1
CONIF	Percent Coniferous Forest	17.2783	percent	3.07	56.2
DRNAREA	Drainage Area	2.78	square miles	3.26	689
ELEVMAX	Maximum Basin Elevation	1958.888	feet	260	6290
MIXFOR	Percent Mixed Forest	46.3492	percent	6.21	46.1
PREBC0103	Jan to Mar Basin Centroid Precip	5.63	inches	5.79	15.1
PREG_03_05	Mar to May Gage Precipitation	7.1	inches	6.83	11.5
PREG_06_10	Jun to Oct Gage Precipitation	18.3	inches	16.5	23.1
TEMP	Mean Annual Temperature	42.439	degrees F	36	48.7
TEMP_06_10	Jun to Oct Mean Basinwide Temp	59.438	degrees F	52.9	64.4

Seasonal Flow Statistics Disclaimers [Low Flow Statewide]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Seasonal Flow Statistics Flow Report [Low Flow Statewide]

Statistic	Value	Unit
Jan to Mar15 60 Percent Flow	1.33	ft^3/s
Jan to Mar15 70 Percent Flow	1.1	ft^3/s
Jan to Mar15 80 Percent Flow	0.968	ft^3/s
Jan to Mar15 90 Percent Flow	0.73	ft^3/s
Jan to Mar15 95 Percent Flow	0.578	ft^3/s
Jan to Mar15 98 Percent Flow	0.488	ft^3/s
Jan to Mar15 7 Day 2 Year Low Flow	0.994	ft^3/s
Jan to Mar15 7 Day 10 Year Low Flow	0.526	ft^3/s
Mar16 to May 60 Percent Flow	6.87	ft^3/s
Mar16 to May 70 Percent Flow	5.33	ft^3/s
Mar16 to May 80 Percent Flow	3.79	ft^3/s
Mar16 to May 90 Percent Flow	2.55	ft^3/s
Mar16 to May 95 Percent Flow	1.81	ft^3/s
Mar16 to May 98 Percent Flow	1.24	ft^3/s
Mar16 to May 7 Day 2 Year Low Flow	1.49	ft^3/s

Statistic	Value	Unit
Mar16 to May 7 Day 10 Year Low Flow	0.778	ft^3/s
Jun to Oct 60 Percent Flow	0.483	ft^3/s
Jun to Oct 70 Percent Flow	0.354	ft^3/s
Jun to Oct 80 Percent Flow	0.258	ft^3/s
Jun to Oct 90 Percent Flow	0.164	ft^3/s
Jun to Oct 95 Percent Flow	0.112	ft^3/s
Jun to Oct 98 Percent Flow	0.0947	ft^3/s
Jun to Oct 7 Day 2 Year Low Flow	0.181	ft^3/s
Jun to Oct 7 Day 10 Year Low Flow	0.0646	ft^3/s
Nov to Dec 60 Percent Flow	2.82	ft^3/s
Nov to Dec 70 Percent Flow	2.21	ft^3/s
Nov to Dec 80 Percent Flow	1.73	ft^3/s
Nov to Dec 90 Percent Flow	1.16	ft^3/s
Nov to Dec 95 Percent Flow	0.779	ft^3/s
Nov to Dec 98 Percent Flow	0.501	ft^3/s
Oct to Nov 7 Day 2 Year Low Flow	1.65	ft^3/s
Oct to Nov 7 Day 10 Year Low Flow	0.741	ft^3/s

Seasonal Flow Statistics Citations

Flynn, R.H. and Tasker, G.D.,2002, Development of Regression Equations to Estimate Flow Durations and Low-Flow-Frequency Statistics in New Hampshire Streams: U.S.Geological Survey Scientific Investigations Report 02-4298, 66 p. (http://pubs.water.usgs.gov/wrir02-4298)

Bankfull Statistics

Bankfull Statistics Parameters [Appalachian Highlands D Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	0.07722	940.1535

Bankfull Statistics Parameters [New England P Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	3.799224	138.999861

Bankfull Statistics Parameters [USA Bieger 2015]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	0.07722	59927.7393

Bankfull Statistics Flow Report [Appalachian Highlands D Bieger 2015]

Statistic	Value	Unit
Bieger_D_channel_width	23.2	ft

Statistic	Value	Unit
Bieger_D_channel_depth	1.5	ft
Bieger_D_channel_cross_sectional_area	35.4	ft^2

Bankfull Statistics Disclaimers [New England P Bieger 2015]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Bankfull Statistics Flow Report [New England P Bieger 2015]

Statistic	Value	Unit
Bieger_P_channel_width	33.6	ft
Bieger_P_channel_depth	1.72	ft
Bieger_P_channel_cross_sectional_area	58.3	ft^2

Bankfull Statistics Flow Report [USA Bieger 2015]

Statistic	Value	Unit
Bieger_USA_channel_width	17.7	ft
Bieger_USA_channel_depth	1.5	ft
Bieger_USA_channel_cross_sectional_area	29.7	ft^2

Bankfull Statistics Flow Report [Area-Averaged]

Statistic	Value	Unit
Bieger_D_channel_width	23.2	ft
Bieger_D_channel_depth	1.5	ft
Bieger_D_channel_cross_sectional_area	35.4	ft^2
Bieger_P_channel_width	33.6	ft
Bieger_P_channel_depth	1.72	ft
Bieger_P_channel_cross_sectional_area	58.3	ft^2
Bieger_USA_channel_width	17.7	ft
Bieger_USA_channel_depth	1.5	ft
Bieger_USA_channel_cross_sectional_area	29.7	ft^2

Bankfull Statistics Citations

Bieger, Katrin; Rathjens, Hendrik; Allen, Peter M.; and Arnold, Jeffrey G.,2015, Development and Evaluation of Bankfull Hydraulic Geometry Relationships for the Physiographic Regions of the United States, Publications from USDA-ARS / UNL Faculty, 17p. (https://digitalcommons.unl.edu/usdaarsfacpub/1515?utm_source=digitalcommons.unl.edu%2Fusdaarsfacpub%2F1515&utm_medium=PDF&utm_campaign=PDFCoverPages)

Maximum Probable Flood Statistics

Maximum Probable Flood Statistics Parameters [Crippen Bue Region 1]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	2.78	square miles	0.1	10000

Maximum Probable Flood Statistics Flow Report [Crippen Bue Region 1]

Statistic	Value	Unit
Maximum Flood Crippen Bue Regional	7440	ft^3/s

Maximum Probable Flood Statistics Citations

Crippen, J.R. and Bue, Conrad D.1977, Maximum Floodflows in the Conterminous United States, Geological Survey Water-Supply Paper 1887, 52p. (https://pubs.usgs.gov/wsp/1887/report.pdf)

Recharge Statistics

Recharge Statistics Parameters [Groundwater Recharge Statewide 2004 5019]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
CONIF	Percent Coniferous Forest	17.2783	percent	3.07	56.18
MINTEMP_W	Mean Winter Min Temperature	9.683	degrees F	0.8	19.88
MIXFOR	Percent Mixed Forest	46.3492	percent	6.21	46.13
PREBC_1112	Nov to Dec Basin Centroid Precip	6.69	inches	6.57	15.2
PRECIPCENT	Mean Annual Precip at Basin Centroid	38.1	inches	37.44	75.91
PRECIPOUT	Mean Annual Precip at Gage	37.6	inches	35.83	53.11
PREG_03_05	Mar to May Gage Precipitation	7.1	inches	6.83	11.54
PREG_06_10	Jun to Oct Gage Precipitation	18.3	inches	16.46	23.11
SNOFALL	Mean Annual Snowfall	85.207	inches	54.46	219.07
TEMP	Mean Annual Temperature	42.439	degrees F	36.05	48.69

Recharge Statistics Disclaimers [Groundwater Recharge Statewide 2004 5019]

One or more of the parameters is outside the suggested range. Estimates were extrapolated with unknown errors.

Recharge Statistics Flow Report [Groundwater Recharge Statewide 2004 5019]

Statistic	Value	Unit
GW_Recharge_Jan_to_Mar15	3.45	in
GW_Recharge_Mar16_to_May	6.86	in
GW_Recharge_Jun_to_Oct	2.94	in
GW_Recharge_Nov_to_Dec	2.88	in
GW_Recharge_Ann	17.5	in

Recharge Statistics Citations

Flynn, R.H. and Tasker, G.D.,2004, Generalized Estimates from Streamflow Data of Annual and Seasonal Ground-Water-Recharge Rates for Drainage Basins in New Hampshire, U.S. Geological Survey Scientific Investigations Report 2004-5019, 67 p. (http://pubs.usgs.gov/sir/2004/5019/)

Application Version: 4.29.2
StreamStats Services Version: 1.2.22
NSS Services Version: 2.2.1