Geotechnical Engineering Evaluation Report

Waban Hill Reservoir Dam
NID # MA 01111
Newton, Massachusetts

Prepared for:
City of Newton
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Newton, Massachusetts 02459

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Contract # L-5992

March 20, 2014

Sign-off Sheet

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Table of Contents

LIST OF TABLES

LIST OF FIGURES

The appendixes of this report are posted separately on the Newton CPC website.

~~LIST OF APPENDICES~~

1.0 Introduction

Stantec Consulting Services, Inc. (Stantec) performed a subsurface exploration program and slope stability analyses for the Waban Hill Reservoir Dam embankments that is bounded by Ward Street to the south, Manet Road to the east, Reservoir Drive, to the north, and Woodlawn Avenue to the west in Newton, Massachusetts. These services are pursuant to our Agreement (Contract # L-5992) dated October 23, 2013. The purpose of these services was to assess the surficial and internal subsurface conditions of the earth embankments at the reservoir and to provide a geotechnical engineering evaluation of the stability of the embankments under the current normal water levels within the reservoir now that it has been removed from the Massachusetts Water Resources Authority (MWRA) water supply system. The property encompasses approximately 5.06 acres. The City of Newton is considering the acquisition of the property from the Commonwealth of Massachusetts Department of Capital Asset Management and Maintenance (DCAM) for use as public open space.

The internal makeup of the embankment was unknown and the upstream and downstream side slopes of the embankment are very steep. The "Waban Hill Reservoir Dam Phase I Inspection/Evaluation Report," dated in September 7, 2012, identified a number of deficiencies that included heaving/bulging of the slope protection along the upstream slope and areas of potential movement and irregularities in the downstream slope. The purpose of this geotechnical evaluation was to determine the cause of these deficiencies prior to the City taking ownership of the reservoir and associated liability.

The geotechnical investigation consisted of the drilling of four soil borings that varied in depth between 24 feet and 42 feet along the top of the embankment and four borings along the toe of the embankment that varied in depth from 8 feet to 30 feet to determine the makeup of the dam. The information obtained was used to determine engineering properties to the various materials that make up the dam and to evaluate both the upstream and downstream embankment slope stability for several loading conditions (steady state seepage and earthquake) with the reservoir at its maximum storage level around El 255; approximately 15 feet below the top of the dam required by the Commonwealth of Massachusetts Department of Conservation and Recreation Office of Dam Safety per 310 CMR 10.00, "Dam Safety." The bottom of the reservoir is at El 246 resulting in a maximum water depth within the reservoir varying between 9 to 10 feet. Three groundwater observation wells were installed at the southeast corner of the reservoir where the dam height is the greatest to determine the phreatic surface (water flow lines) through the dam.

2.0 SITE AND PROJECT DESCRIPTION

The Waban Hill Reservoir was constructed by the City of Newton in 1877 and was purchased by the Metropolitan Water Board in 1900 as part of the water supply system for the City of Boston and surrounding communities. It is located at coordinates 42.33795N and 71.17663W. A Site Locus Plan is presented as Figure 1.1. The reservoir is an off stream impoundment with a drainage area (3.0 acres) that is essentially the surface area of the reservoir itself (2.9 acres). It was used by the Massachusetts Water Resources Authority (MWRA) to provide high service pressure and was filled via water pumped through a water supply pipeline. It provided an emergency water source to fill a high service pipeline to the Chestnut Hill emergency pump station to provide backpressure to allow a pump start. The reservoir is a redundant feature in the water supply system and has recently been decommissioned from service. The 20-inch diameter pipeline used to fill the reservoir has been disconnected making it impossible to fill the reservoir beyond the current levels other than by rainfall.

The Waban Hill Reservoir is a 1,300 foot long earthen embankment with a maximum height of 22 feet and a maximum storage capacity of 60 acre-feet. The historic normal pool elevation for the reservoir is at El 259. The bottom of the reservoir is at elevation (El) 246 and the top of the embankment is at El 270. The upstream slope is armored with hand placed granite block riprap with a 1.5 horizontal (H) to 1 vertical (V) slope. The crest of the embankment is flat, grass covered, and is about 15 feet wide. The downstream slope is 1.7H to 1V and varies in height from 0 feet at the northeast corner to 22 feet at the southeast corner of the reservoir. The reservoir and embankments encompass approximately 2.9 acres of the 5 acre property.

There is a gatehouse located in the southwest corner that contains a pair of sluicegates. The primary outlet is also located in the southwest corner of the reservoir and is a 20-inch diameter cast iron pipe which contains an air gap and spool piece for reservoir disconnect. The 20-inch pipe then increases to 30-inches and then connects to a 36-inch diameter water supply pipe via a T-connection. A secondary outlet is an 8-inch diameter cast iron drain which can be used to manually control the reservoir elevation. This 8-inch drain discharges to an 8-inch diameter cast iron pipe in Ward Street and is the current means for lowering and controlling the reservoir level with the 20-inch spool piece removed.

There is no overflow spillway at the reservoir. The outlet works serve as the only discharge structures at the dam.

Reservoir levels are maintained as natural fluctuations in the impoundment level occur. The reservoir impoundment is only effected by rainfall and evaporation. If the impoundment level increases, the water surface can be manually lowered by discharging water into the City of Newton storm water system.

The internal makeup of the embankment is unknown (i.e. clay central core). The Phase I Inspection/Evaluation Report dated September 7, 2012 identified a number of deficiencies that included heaving/bulging of the slope protection along the upstream slope and areas of potential movement and irregularities in the downstream slope.

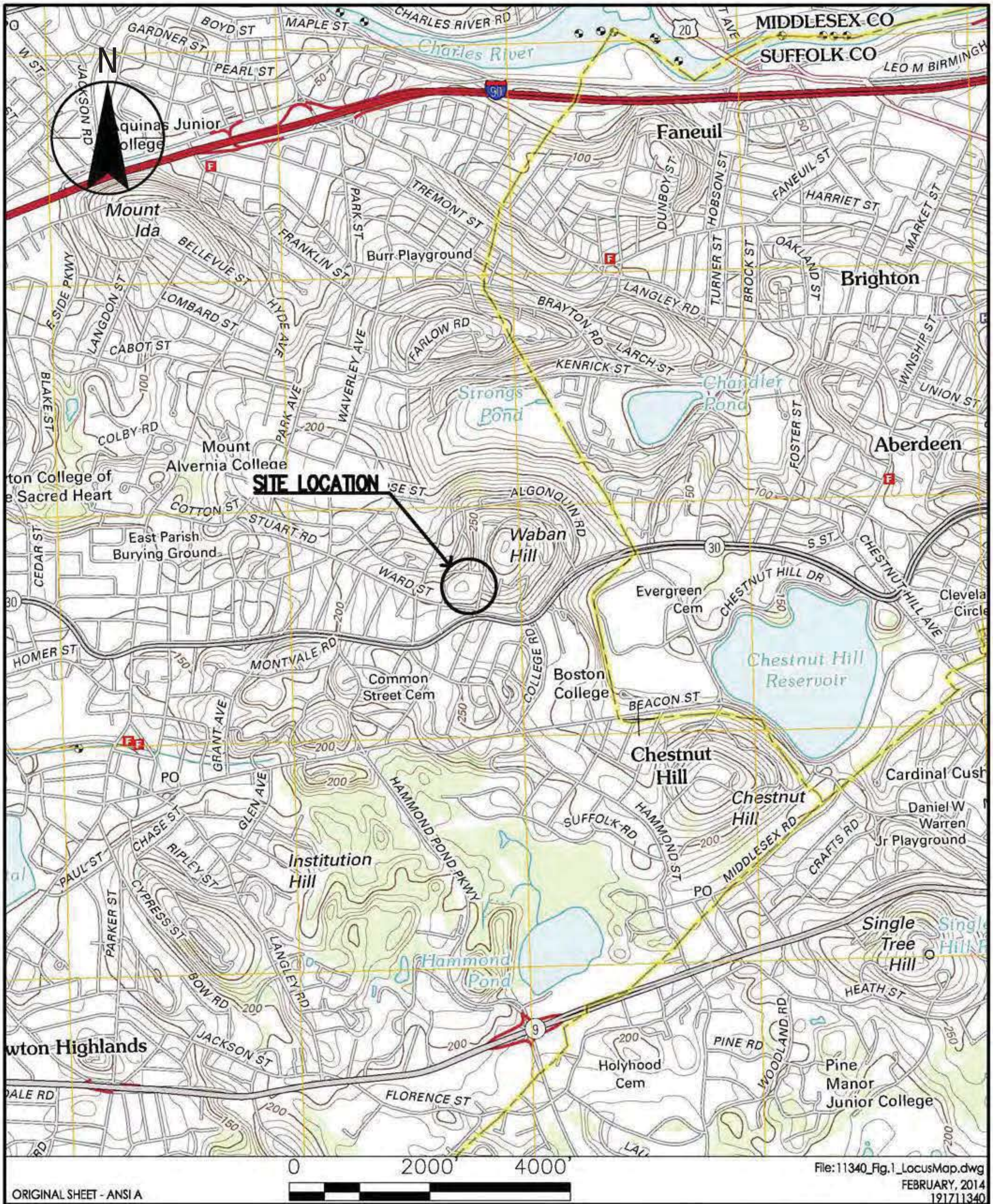

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Figure No.

2.1

Title

SITE LOCUS MAP

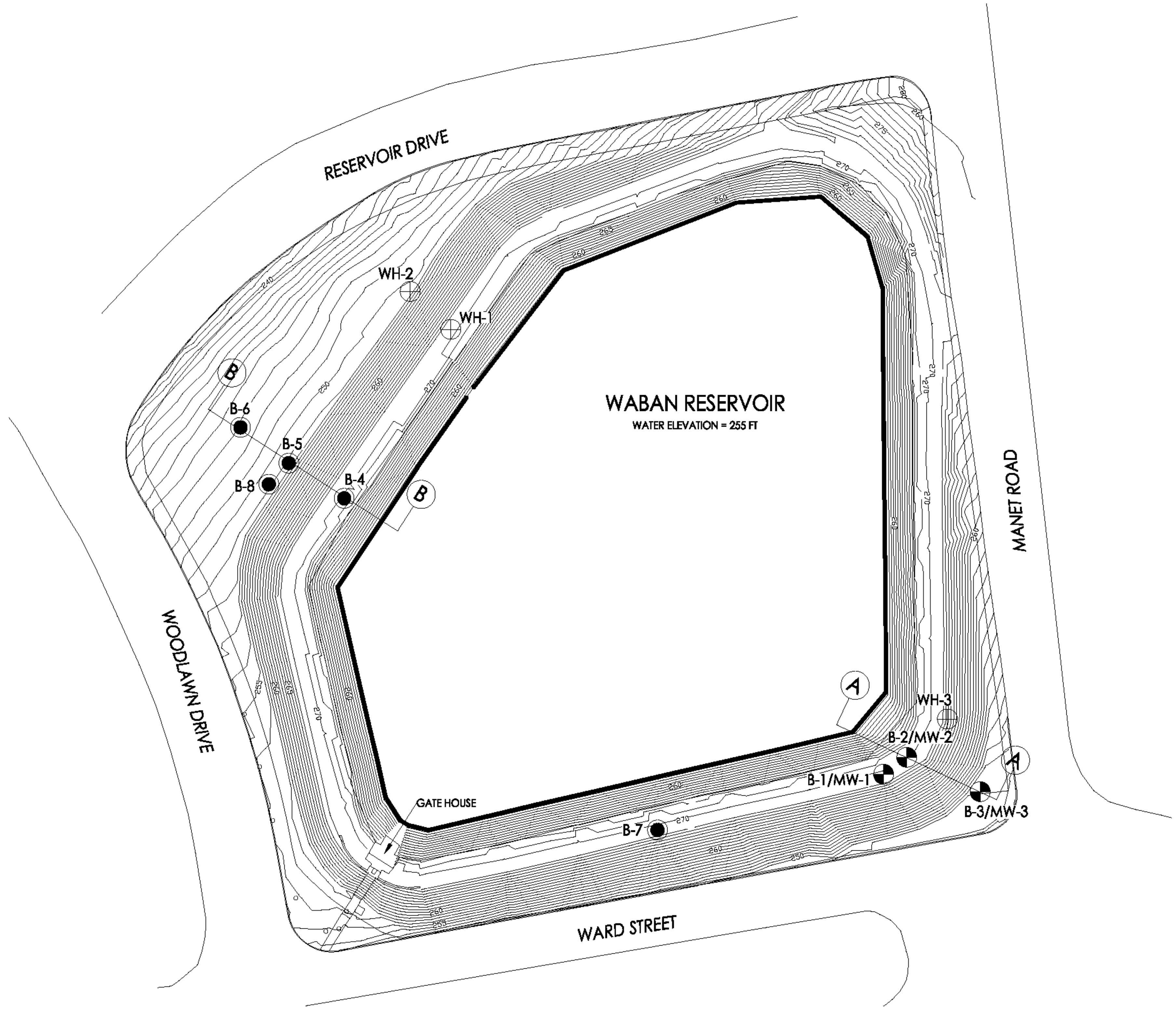
3.0 EXPLORATION AND TESTING PROCEDURES

Eight test borings (B-1 through B-8) were drilled to determine the internal makeup of the embankment by New Hampshire Borings, Inc. of Derry, New Hampshire between January 14 and 17, 2014 under contract with Stantec. An all-terrain vehicle (ATV) track mounted (CME 550) drill rig was used to advance the borings. Hollow stem augers with4-1/4 inch insider diameter were used to advance the borings for the first 8 feet of drilling. Drive and wash drilling methods were then used to advance the borings using 4-inch flush joint casing to the depth of exploration. A roller bit was used to clean out the material within the casing. Three borings (B-1 through B-3) were located along the maximum embankment section at the southeast corner of the reservoir; four borings (B-4 through B-6, and B-8) along the northwest corner adjacent to Woodlawn Drive; and one boring (B-7) was drilled along the top of embankment along the embankment section parallel to Ward Street. The test borings were monitored by a Stantec scientist. The borings were designated B-1 through B-8 and the borehole logs are included in Appendix A. The locations of both borings are shown on Figure 3.1 – Boring Location Plan.

Subsurface information from three borings WH-1 through WH-3 conducted in April 1983 were reviewed and supplemented the findings from the 2014 exploration program. The borings were drilled by Empire Soils Investigation, Inc. of Latham New York and the boring logs are included in Appendix B. Gradation analyses from the 1983 borings are presented in Appendix C.

Standard Penetration Tests (SPTs) were performed in each boring in general accordance with ASTM D1586. The SPT consists of driving a 1 3/8-inch inside diameter split spoon sampler with a 140 pound free hammer falling 30-inches. The blows for each 6-inches of penetration are recorded for a total of 24-inches. The sum of the blows required to drive the sampler from 6-inches to 18-inches penetration is referred to as the Standard Penetration Resistance, or N-value, which is an index of measure of in-situ soil density or consistency. For granular soils, N values less 4 are considered to be very loose; between 4 and 10 loose; between 10 and 30 medium dense; between 30 and 50 dense; and greater than 50 very dense.

Three groundwater observation wells were installed at the southeast corner of the reservoir in borings B-1, B-2, and B-3 to determine the water level through the embankment along the maximum cross section of the embankment.



BORING AND MW ELEVATIONS		
BORING/MW NO.	GROUND ELEVATION (ft above MSL)	PVC ELEVATION (ft above MSL)
B-1/MW-1	270.10	269.58
B-2/MW-2	270.12	269.50
B-3/MW-3	248.25	248.04
B-4	270.56	---
B-5	250.91	---
B-6	246.74	---
B-7	270.21	---
B-8	250.40	---
WH-1	270.0	---
WH-2	251.5	---
WH-3	270.2	---

ORIGINAL SHEET - ANSI B

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Legend

- B-7 TEST BORING (2014)
- B-1/MW-1 TEST BORING WITH MONITORING WELL (2014)
- WH-3 TEST BORING (1983)
- B B SUBSURFACE PROFILE LOCATION

Notes

1. WH- SERIES BORINGS MADE IN APRIL 1983 BY EMPIRE SOILS INVESTIGATIONS, INC. LOCATIONS ARE ESTIMATED.
2. B- SERIES BORINGS MADE IN JANUARY 2014 BY NEW HAMPSHIRE BORING, INC. LOCATIONS ARE AS-BUILT PER CITY OF NEWTON SURVEYORS

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3.1

Title
BORING LOCATION PLAN

4.0 SUMMARIZED SUBSURFACE CONDITIONS

The fill used to construct the embankment was probably from materials excavated from within the reservoir with the exception of the northeast corner which was cut into the side of the hill. Waban Hill is a drumlin, a small hill composed of glacial till. The borings drilled at the dam crest encountered glacial till at a depth of about 20 feet. The material in the embankment was similar to the underlying glacial till except it was less dense. The test borings generally encountered a surficial topsoil layer underlain by embankment fill and glacial till. Subsurface information for the southeast corner of the reservoir is presented in Figure 3.1, Subsurface Profile A-A and for the northwest corner is presented in Figure 3.2, Subsurface Profile B-B. Each unit is described below.

4.1 TOPSOIL

Each boring encountered a six inch layer thick of topsoil.

4.2 EMBANKMENT FILL

The results of the soil samples recovered from the test borings suggest that the embankment is relatively homogeneous. Embankment fill samples primarily consisted of a brown fine Silty Sand with little to trace gravel, trace clay. The fill extended to a depth of about 20 feet or El 250 in each of the four borings drilled from the dam crest. Recorded N-values ranged between 16 to 43 blows per foot (bpf) indicating a medium dense to dense consistency. Superimposing the dam section from the proposed 1944 raising, it appears that the soil sample in Boring B-4 at a depth of 20 feet may have encountered the puddled core (SM-ML).

4.3 GLACIAL TILL

A stratum of glacial till approximately 17.5 feet to 24 feet in thickness was encountered in borings in B-1 and B-2, respectively. This layer of glacial till was generally described as gray/brown, fine to coarse sandy silt, trace clay. Recorded N-values varied between 36 and 100+ bpf, indicating a variable dense to very dense compactness.

4.4 BEDROCK

Bedrock or refusal was not encountered in any of the borings within the depth of exploration.

4.5 GROUNDWATER

A groundwater observation well was installed in borings B-1, B-2 and B-3 along the maximum embankment section at the southeast corner of the reservoir. Two inch diameter 10 feet long well screens were installed at the depths indicated below and solid flush jointed PVC pipe to within six inches of the ground surface. A flush mounted roadway box was installed at the ground surface to allow future access to the well if needed. On January 27, 2014 the water level within the reservoir was at El 255.84. Groundwater was observed in the observation wells as follows:

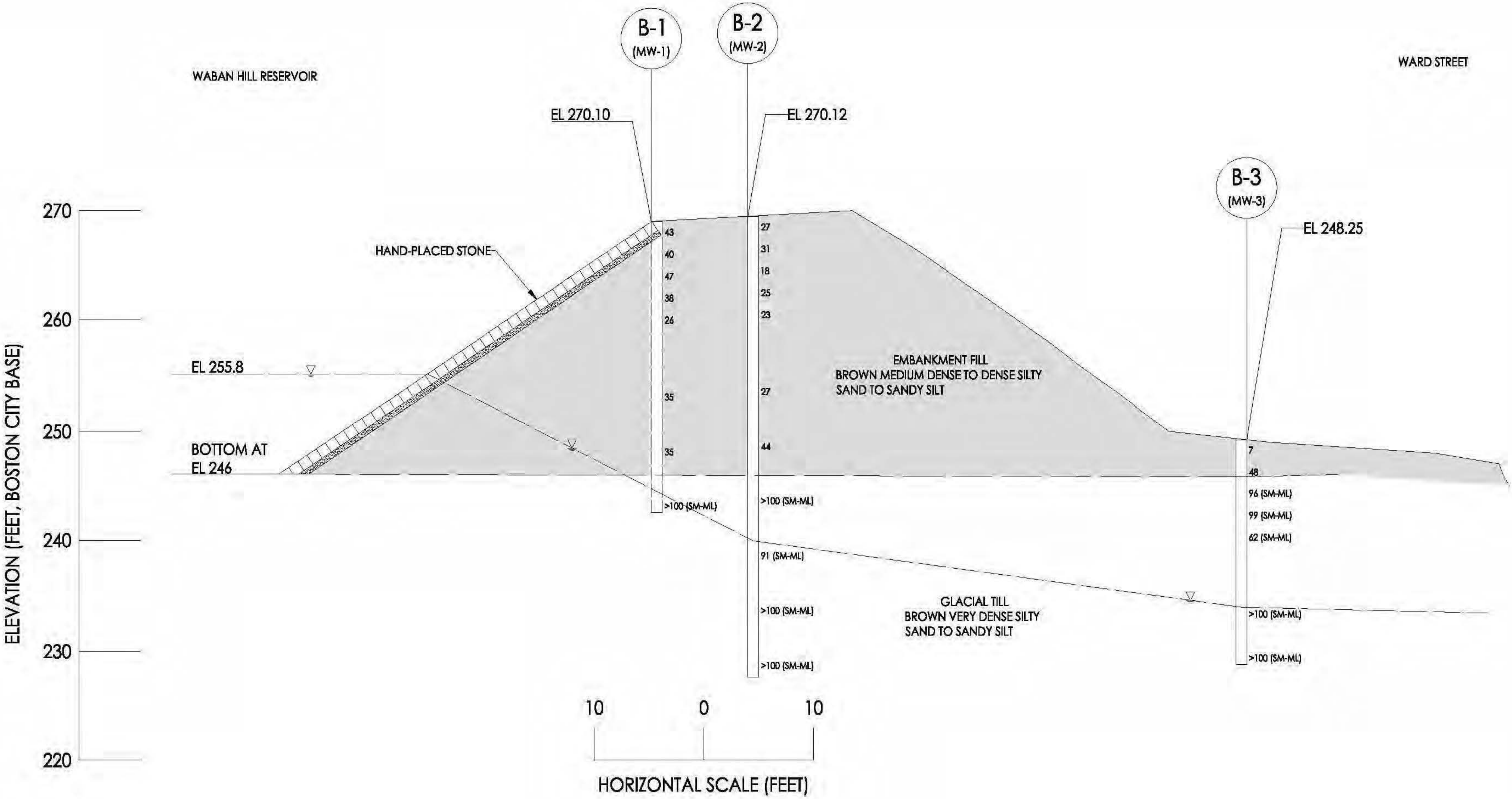
Table 3-1 Groundwater Monitoring Results

Observation Well	Ground Surface Elevation	Depth to Bottom of Well (feet)	Depth to Groundwater (feet)	Groundwater Elevation
B-1	270.12	26	Dry @ 24.79	Below 245.33
B-2	270.10	36	Dry @35.20	Below 234.90
B-3	248.25	18	14.03	234.22

Equilibrated hydrostatic levels may vary dramatically from those recorded during drilling. Actual groundwater levels may vary over time due to seasonal changes in precipitation and temperature, snowmelt, and surrounding and on-site drainage characteristics.

4.6 SOIL LABORATORY TESTING

Laboratory test results, gradation analyses from the 1983 drilling program (WH series borings) are presented in Appendix C.



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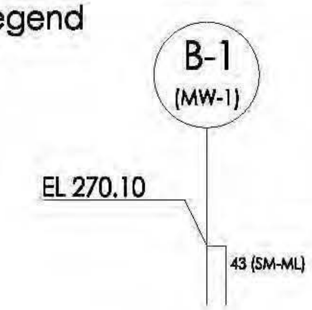

LOCATION AND IDENTIFICATION OF TEST BORING AND MONITORING WELL (IF INSTALLED), MADE JANUARY 2014

GROUND SURFACE ELEVATION OF TEST BORING

NUMBER ALONG BOREHOLE REFERS TO STANDARD PENETRATION TEST BLOW COUNT AND UNIFIED SOIL CLASSIFICATION SOIL TYPE

GROUND AND RESERVOIR WATER LEVEL

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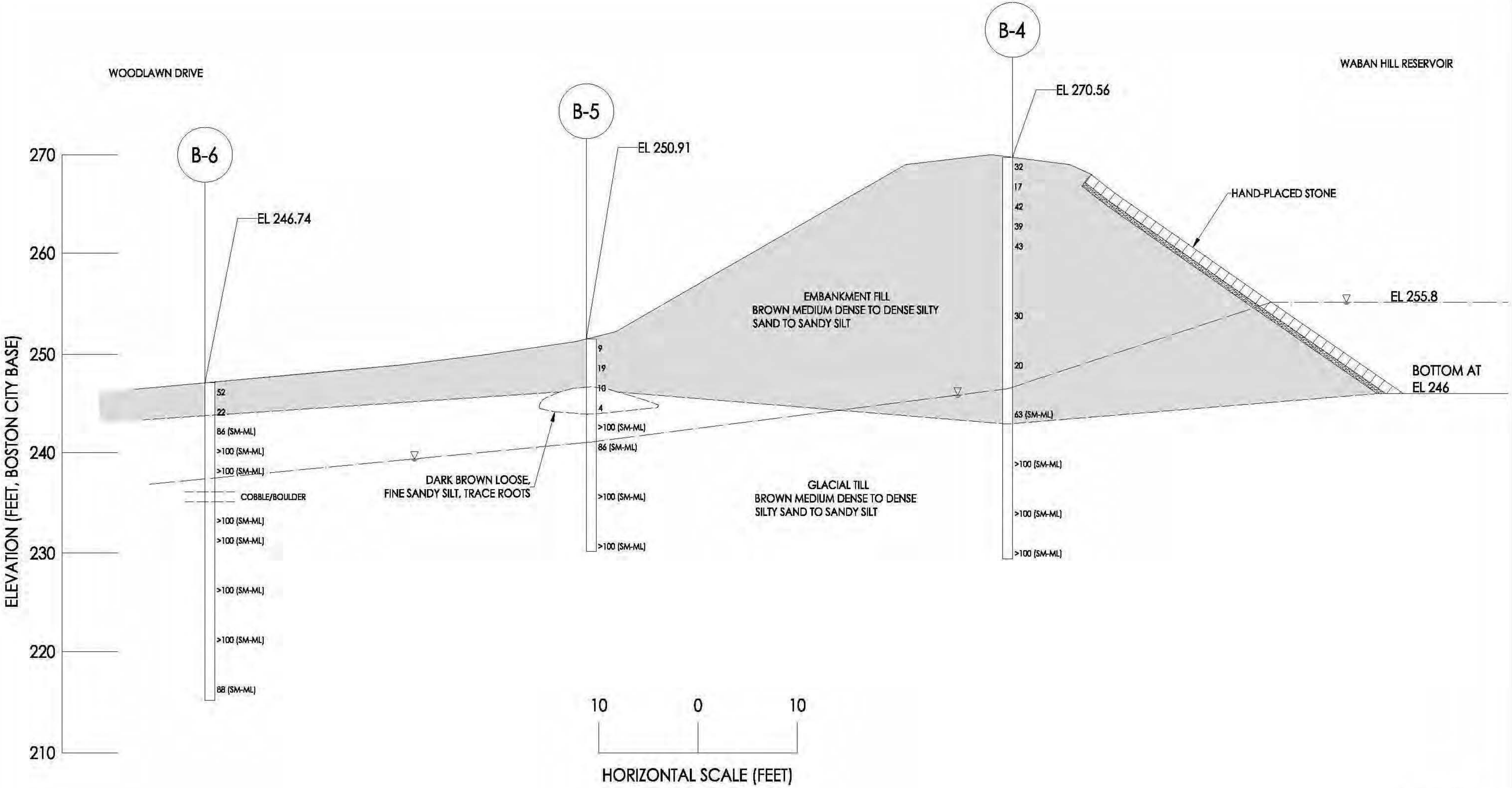
4.1

Title
SUBSURFACE PROFILE A-A



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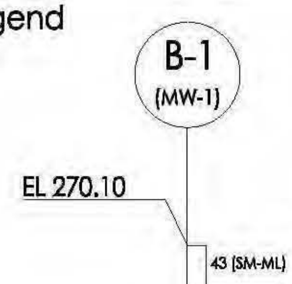

LOCATION AND IDENTIFICATION OF TEST BORING AND MONITORING WELL (IF INSTALLED), MADE JANUARY 2014

GROUND SURFACE ELEVATION OF TEST BORING

NUMBER ALONG BOREHOLE REFERS TO STANDARD PENETRATION TEST BLOW COUNT AND UNIFIED SOIL CLASSIFICATION SOIL TYPE

GROUND AND RESERVOIR WATER LEVEL

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SUBSURFACE PROFILE B-B

assumptions are made regarding potential slip surfaces until the most critical surface is found. When the shear resistance of the soil along the slip surface exceeds that necessary to provide equilibrium then the mass is considered to be stable. If the shear resistance is insufficient, then the mass is unstable. The stability or instability of the mass depends on its weight, the external forces acting on it, soil shear strength, and pore water pressures along the slip surface.

5.1.1 Soil Properties

Soil properties were estimated using the subsurface information obtained from the test borings. Groundwater conditions were based on the groundwater level observed in the observation wells and anticipated increase of one foot due to storm events. A summary of the soil properties used in the analysis is provided in Table 5-1.

Sieve analyses, performed in 1983 on split-spoon samples from the WH series borings boring program were used to estimate the permeability of the soils that make up the embankment. The relationship between conductivity and grain size requires the choice of a representative grain-size diameter (Freeze and Cherry, 1979). A simple, empirical relation is described by the formula:

$K = A\,(d_{10})^2$

where:

K: Hydraulic conductivity in cm/sec

d_{10}: The grain-size diameter, in mm, at which 10% by weight of the soil particles are finer and 90% are coarser. The d_{10} value is taken directly from the gradation curves.

A: A constant; for K in cm/s and d_{10} in mm, the coefficient A is equal to 1.0.

As shown in Appendix C, for the soil samples analyzed, the d_{10} fraction was not able to be calculated directly from the gradation curves due to a relatively high content of fines (35% to 43% passing a No. 200 sieve). The d_{10} fraction was estimated through extrapolation of the gradation curves. The calculated permeability, based on grain-size distribution was estimated to range between 6×10^{-5} cm/sec to 4×10^{-6} cm/sec. Based on this information, the phreatic surface through the embankment would be slow to react to sudden changes in the reservoir level.

Table 5-1 Summary of Soil Properties

Soil Unit	Soil Properties		
	Moist Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Hand Placed Riprap	150	45	0
Embankment Fill	125	38	0
Puddle Fill Core	125	33	100
Glacial Till	130	38	0

5.1.2 Factor of Safety

For stability analyses of embankment dams, the recommended factors of safety will vary with loading conditions. Long-term loading conditions (i.e., steady seepage) require a higher factor of safety while short-term loading conditions (i.e., rapid drawdown, earthquakes) will require a lower factor of safety. The minimum factor of safety for each loading condition per 302 CMR 10.14(b) "Dam Safety" is presented in Table 5-2. The loading conditions that are applicable to Waban Hill Reservoir Dam embankment stability evaluation include steady state seepage under normal pool and earthquake. Because the reservoir has been disconnected from the MWRA system, the rapid drawdown from normal pool condition is not applicable. However, this condition was evaluated to determine whether the observed sloughing along areas of the upstream slope was the result of this loading condition during the 100+ years that the reservoir was used to provide high service pressure and storage. A description of each condition is as follows:

5.1.2.1 Steady-State Seepage: Normal Pool

It is general practice to analyze the stability of the downstream slope of the dam embankment for steady-state seepage (or steady seepage) conditions with the reservoir at its normal operating pool elevation since this is the loading condition the embankment will experience most.

5.1.2.2 Earthquake (Pseudostatic Analysis)

Earthquakes result in an additional loading on the dam embankment materials. The pseudostatic method assumes that the earthquake causes additional horizontal forces in the direction of potential failure surfaces on both the upstream and downstream slopes. These failure surfaces were determined by the long-term static loading conditions (such as steady-state seepage resulting from normal reservoir levels). A seismic coefficient of 0.10g was used in the analyses. The pseudostatic method of analysis is not applied to short-term to temporary static loading conditions (such as end of construction, flood storage pool, or rapid drawdown.

5.1.2.3 Rapid (or Sudden) Drawdown from Normal Pool

This loading condition assumes that steady-state seepage conditions have been established within the embankment as a result of maintaining a reservoir at the normal pool elevation and that the embankment materials beneath the phreatic surface are saturated. The reservoir is then drawn down faster than the pore pressures within the embankment materials can dissipate, resulting in a reduced factor of safety. This loading condition is the normal operating case for pumped-storage reservoirs such as Waban Hill Reservoir where the drawdown of the reservoir (up to 5 to 10 feet per hour) occurs occasionally. This loading condition is analyzed only for the upstream slope of the dam.

Table 5-2 Slope Stability Analysis Minimum Factors of Safety

Loading Condition	Slope	Required Factor of Safety
Steady Seepage from normal pool level	Upstream/Downstream	1.5
Rapid Drawdown from normal pool level	Upstream	1.2
Earthquake for steady seepage conditions with reservoir at normal pool. Seismic loading using seismic coefficient method (Pseudostatic).	Upstream/Downstream	>1.0

5.1.3 Slope Stability Results - Steady Seepage and Earthquake Conditions

The factor of safety includes a margin of safety to guard against ultimate failure, to avoid unacceptable deformations, and to cover uncertainties associated with the measurement of soil properties or the analysis used.

A factor of safety of 1.0 indicates the driving forces are equal to the resisting forces and the slope is in a state of equilibrium. A factor of safety of less than 1.0 indicates the driving forces are greater than the resisting forces and the slope will fail. A summary of the slope stability analyses for the steady seepage and earthquake conditions are presented in Table 5-3 for the two subsurface profiles evaluated. The failure surface for the minimum factor of safety for each condition is presented in Appendix E.

Table 5-3 Slope Stability Results

Slope /Condition Evaluated	Factor of Safety	
	Steady State Seepage	Earthquake
Southeast Corner – Subsurface Profile A-A		
Downstream Slope	1.480	1.181
Upstream Slope	1.327	1.038
Northwest Corner – Subsurface Profile B-B		
Downstream Slope	1.599	1.267
Upstream Slope	1.376	1.070

5.1.4 Slope Stability Results - Sudden Drawdown Condition

Along the upstream slope of the reservoir, a number of surficial sloughs were observed that may be the result of sudden (rapid) drawdown condition that may have occurred when the water in the reservoir was used to fill a high service pipeline back to the Chestnut Hill Reservoir emergency pump station to provide the necessary backpressure to allow a pump to start. This condition was modeled for an upstream slope with the reservoir at El 262 with a sudden drawdown of 10 feet to El 252. A low permeability puddled fill core located along the top of the upstream slope was added to this condition. A factor of safety of less than 1 was calculated (FS= 0.839) and is an indication as to why the displacement/sloughing of the stone paving along the upstream slope may have occurred during past operations.

5.2 CONCLUSIONS

Waban Hill Reservoir Dam Embankments are over 125 years old with no evidence of seepage or wet spots along the toe of the embankments noted during Phase I Dam Safety Inspection/Evaluation Reports dated 1980, 2010, and 2012. The material that makes up the embankment is a dense Sandy Silt to Silty Sand. Based on the lowered elevation of the water level within the reservoir (El 255±) and the removal of the spool piece that prevents the pumping of water into the reservoir from the MWRA system, the water level within the reservoir can vary only by rainfall and evaporation. There is no additional drainage area other than the interior of the reservoir. The upstream (1.5H to 1V) and downstream (1.7H to 1V) embankment slopes are very steep. However, due to the density and makeup of the embankment the dam is considered to be stable under the current conditions. The bottom of the reservoir is at El 246 and the maximum water level in the reservoir will not exceed El 256. The toe of slope at the maximum sections along Reservoir Drive and Ward Street range between El 251 to 248, respectively. Although the slope stability analyses did not satisfy the minimum failure surfaces for the steady state condition, it is believed that the embankment will remain basically stable under the current conditions. The calculated failure surfaces for the shown factors of safety are relatively shallow (see Appendix E for plots) and a substantial embankment would remain even if a slide did occur that would minimize the impact of a release of stored water within the reservoir which amounts to approximately 17 acre-feet.

5.3 HAZARD RE-CLASSIFICATION

The Dam is currently classified by the Massachusetts Department of Conservation and Recreation, Office of Dam Safety as Intermediate Size, High Hazard . Based upon a failure occurring with the water level within Waban Hill Reservoir at El 268.1 (e.g. the elevation of the spillway design flood from the one-half Probable Maximum Flood) the reservoir would have a maximum storage of about 60 acre-feet of water.

Current operations maintain the level of the impoundment between El 255 to El 256, roughly 7 feet above the downstream toe at the southeast corner of the reservoir and 4 feet above the northwest. The storage within the impoundment at El 256 is about 17 acre feet. Consideration should be given to approaching the Office of Dam Safety regarding the reduction the hazard classification of Waban Hill Reservoir Dam.

6.0 Limitations

6.1 USE OF REPORT

This report has been prepared for the exclusive use of City of Newton, Massachusetts and their respective assigns and designees. This report is not intended for the use or reliance of other (third) parties, without the express consent of Stantec and The City of Newton, Massachusetts. Any use, which a third party makes of this report, or any reliance on decisions made based on this report, is the responsibility of such third parties. Further, the findings of this study apply only to the specific Site and project described herein. The findings herein are inapplicable to other Sites, and to developments of different grading, layout, loading, and performance requirements. Stantec accepts no responsibility for damages, real or perceived, suffered by parties as a result of decisions made or actions based on the unintended and/or inappropriate use of this report.

The Geotechnical Report provides recommendations, and is intended for informational use, requiring interpretation by the owner, design team, and contractor for the design and construction of the project, and interpretation of final quantities and construction costs. The Geotechnical Report is not intended, or suitable, by itself, for use as a technical specification or to determine quantities. Anticipated quantities and/or costs may be provided in the Geotechnical Report; such information is an Engineer's interpretation, and may vary dramatically from contractor bids, which are based on potentially differing interpretations, and several other variables not available or considered by the Engineer.

6.2 SUBSEQUENT INVOLVEMENT

The geotechnical process incorporates initial exploration and recommendations as summarized herein, and is followed by continuous involvement during key design and construction benchmarks. The recommendations provided herein are based on preliminary information and assumptions regarding proposed site grading, structural loading and performance requirements. It is recommended that Stantec review final grading, and other applicable plans to assess whether or not these recommendations require modification.

6.3 REPRESENTATION AND INTERPRETATION OF DATA

Surficial and subsurface information presented herein is based on field measurements obtained during the course of the exploration and site reconnaissance. The precision and accuracy of surficial data is a function of the references, benchmarks, methods and instruments employed, as summarized in the report. Subsurface data is based on measurements within the borehole or test pit using the sampling methods described on the exploration logs. The completeness, precision, and accuracy of such data is a function of the frequency and type of exploration and sampling employed, as well as the precision and accuracy of the surface location and elevation of the borehole, and may vary from actual conditions encountered during excavations. Subsurface conditions between, beyond and below explorations, may vary dramatically from the nearest exploration, due to natural geologic action, deposition and weathering, or man-made activities.

6.4 GROUNDWATER

Groundwater levels were recorded during the time periods and frequencies noted on the explorations. It is important to note that groundwater levels are disrupted by the exploration, and require equilibration periods to determine actual hydrostatic levels, which exceed the duration of the measurement period. Multiple hydrostatic groundwater levels may exist, including perched or trapped water, which may not necessarily be accurately represented by one water level reading. Groundwater levels fluctuate due to seasonal variations, adjacent surface water bodies, precipitation, and on-Site and nearby land use.