C.T. MALE ASSOCIATES
Engineering, Surveying, Architecture & Landscape Architecture, D.P.C.

ENGINEERING ASSESSMENT
of
EDGECOMB POND DAM

NATIONAL DAM ID: NY00245
NY STATE ID: 222-1427

Owned and Operated by:

TOWN OF BOLTON

WARREN COUNTY, NY

Latitude N 43° 34′ 20″
Longitude W 73° 41′ 11″

Document Date: February 17, 2016

Assessment Performed by: Richard C. Wakeman, P.E.
NYS License No. 057208

Engineering Assessment
Edgecomb Pond Dam
Edgecomb Pond Road
Bolton, New York

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1
2.0 INITIAL SAFETY INSPECTION 1
2.1 Document Review 1
2.2 Dam Description 1
2.3 Physical Inspection 3
2.3.1 Earthen Embankments 3
2.3.2 Spillway & Discharge Channel 4
2.3.3 Piping & Valves 5
3.0 HAZARD CLASSIFICATION 5
4.0 HYDROLOGIC & HYDRAULIC ASSESSMENT 6
4.1 Design Rainfall Analysis 6
4.2 Watershed Analysis 6
4.3 Spillway Capacity Analysis 7
4.4 Storage Evacuation Calculations 7
4.4.1 Service Spillway Evacuation 8
4.4.2 Auxiliary/Service Spillway Combination Evacuation 8
4.4.3 Low-Level Drain 8
5.0 SUBSURFACE INVESTIGATION PROGRAM 8
6.0 SUBSURFACE CONDITIONS 9
6.1 Embankment 9
6.2 Foundation Soils 9
6.3 Groundwater 9
7.0 STABILITY ANALYSIS 10
8.0 STABILITY RESULTS 10
9.0 CONCLUSIONS & RECOMMENDATIONS 11
9.1 Spillway 11

9.2 Pipes & Valves 12
9.3 Embankment 12
9.4 Recommended Schedule of Work 13
10.0 CLOSURE 13

TABLE OF CONTENTS (cont).

FIGURES

FIGURE 1: Reservoir Location Map
FIGURE 2: Watershed Delineation Map

APPENDICES

APPENDIX A: Historic Drawings & Documents and Limited Topographic Survey
APPENDIX B: Visual Inspection Checklist
APPENDIX C: Inspection Photographs
APPENDIX D: Rainfall Data
APPENDIX E: Soils Information
APPENDIX F: HydroCAD Modeling Reports
APPENDIX G: Proposed Auxiliary Spillway Plan
APPENDIX H: Evacuation Calculations
APPENDIX I: Subsurface Exploration Log
APPENDIX J: Spillway Structural Stability Calculations

1.0 INTRODUCTION

This Engineering Assessment of Edgecomb Pond Dam in the Town of Bolton, New York has been performed in accordance with the requirements set forth in the New York State Department of Environmental Conservation (DEC) Dam Safety Regulations, Part 673.13. Included in the preparation of this assessment were a review of existing documentation for the dam and the performance of an Initial Safety Inspection, a hydrologic and hydraulic analysis of the reservoir's watershed and spillway structure, a hazard classification assessment of the dam and a structural stability analysis of its concrete gravity section. Existing documentation reviewed included historic drawings and documents, past inspection reports, and the dam's Inspection and Maintenance Plan and Emergency Action Plan. On the basis of the findings of this assessment, conclusions and recommendations for bringing the dam into compliance with DEC regulations are provided. A recommended schedule for this work has also been developed.

2.0 INITIAL SAFETY INSPECTION

2.1 Document Review

Prior to visually inspecting the dam, existing documentation in the possession of DEC was reviewed. Included amongst this information were:

- Historic documents pertaining to the dam, including 1977 reconstruction drawings, Applications for Construction or Reconstruction of a Dam dating from 1929, 1931, 1946, and 1950, and various letters and reports;
- DEC inspection reports;
- A partial topographic survey of the dam and its surroundings, including a portion of Finkle Brook extending downstream to its junction with Edgecomb Pond Road, performed by C.T. Male Associates;
- An Inspection & Maintenance Plan, prepared by C.T. Male Associates, dated June 30, 2010, and revised September 14, 2010; and
- An Emergency Action Plan, prepared by C.T. Male Associates, dated October 15, 2010.

2.2 Dam Description

Edgecomb Pond Dam is located in the Town of Bolton, New York at latitude 43 degrees, 34 minutes, 20 seconds north and longitude 73 degrees, 41 minutes, 11 seconds west (See Figure 1). The dam consists of a concrete spillway and a concrete core wall that extends laterally into natural ground on either side of the spillway. Short earthen embankments are present along the downstream side of the core wall to either side of the spillway. Edgecomb Pond is utilized by the Town of Bolton as its primary water supply reservoir. Outflow from the dam is via Finkle Brook.

Little information is known regarding the original construction of Edgecomb Pond Dam. From historic topographic maps and Applications for the Construction or Reconstruction of a Dam dated 1929 and 1931, Edgecomb Pond is known to have been in existence prior to

1929. Although an Application for the Construction or Reconstruction of a Dam dating from 1929 was recovered during the document review process, this structure does not appear to have been constructed under this application. Rather, another application was submitted for construction of a dam in 1931. The dam's design engineer is unknown. In a letter attached to the 1931 Application, it is noted that "For many years a mill dam of boulders and earth fill has been maintained across the creek and the highway now passes over this dam." The proposed dam was to be constructed upstream of the mill dam, presumably at or near the present location of Edgecomb Pond Dam. It is noted that the dam proposed for construction in 1931 was to be of concrete construction, with a concrete core wall along its length. The top of the concrete core wall and earthen embankments were proposed to be established at elevation 102.5 feet (arbitrary datum). A concrete spillway, approximately 26.5 feet in width, was to be constructed with its spillway crest established at elevation 98.5 feet. So as to provide raw water from the reservoir to the Town of Bolton, a 16-inch diameter pipe was to extend from a "gate well" on the right upstream side of the core wall adjacent to the spillway, through the core wall and embankment, and to the Town of Bolton. It is noted in this letter that the dam's foundations were to be keyed two (2) feet into dense glacial till present at relatively shallow depths below the site. No information was recovered regarding whether this work was actually performed, although the general characteristics and location of the proposed structure appear to be consistent with the current structure.

According to another Application for the Construction or Reconstruction of a dam, dated October 22, 1946, the reservoir storage capacity was to be increased by raising the spillway crest elevation two (2) feet. As shown on the Drawing "Section Thru Dam at Edgecomb Pond", appended to the 1946 Application, the spillway training walls and top of the embankment were not raised in elevation, thereby reducing the amount of freeboard present to only one (1) foot. Similar to the original construction, no information was recovered regarding the design engineer responsible for this reconstruction.

In 1950, the reservoir storage capacity is believed to have again been increased by raising the spillway crest an additional two (2) feet. As noted on the Application for the Construction or Reconstruction of a Dam, dated June 5, 1950, these improvements were prepared by Morrell Vrooman Engineers of Gloversville, New York. Previous reconstruction work on the spillway had reduced the freeboard to one (1) foot, and as noted in the Application, it would be necessary to increase the height of the dam's embankments, core wall and spillway training walls so as to reestablish freeboard consistent with the dam's original design and reduce the potential for "a possibly dangerous condition." As inferred from design calculations presented with the 1950 Application, it appears that the core wall and training walls were to be raised by approximately 3 feet.

So as to further increase the reservoir's storage capacity, improvements were again made to the dam and its spillway in 1977. Drawings showing these improvements, prepared by Robert J. Ganley – Consulting Engineer and entitled "Improvements to Edgecomb Pond Dam", were recovered during the document review process. As indicated on these drawings, the spillway crest was again raised by an additional 2 feet and the spillway reconfigured to an 'ogee' configuration similar to that which currently exists. The dam's concrete core wall was extended to both the left and right sides of the spillway and the top of the core wall was raised by approximately 18 inches. Similarly, the spillway training walls were extended approximately 10 feet further downstream and the top of the walls increased to provide a uniform slope. Two (2) buttress walls were to be constructed on either side of the training walls. A stilling basin was to be constructed at the downstream toe of the new spillway section and was to be infilled with grouted rip rap. Improvements were also made to the downstream channel, which was widened for a distance of approximately 20 feet downstream of the spillway.

In or around 1991, the right core wall was thickened, possibly in an attempt to mitigate cracking of a portion of the concrete core wall constructed in 1977. The extents of this work are unclear. In 1995, a new water treatment plant was constructed on a parcel of land immediately north of Edgecomb Pond Dam. At some point after the new plant's construction, the former raw water supply line which ran through the dam was reportedly severed and the substructure for the former gatehouse infilled with concrete. With the exception of minor repairs and/or resurfacing of the dam's concrete, no significant modifications are known to have been performed to the dam since this time.

According to DEC, the dam is 100 feet long and 16 feet high as measured from its crest to its downstream toe. Based upon the limited topographic survey performed of the dam, it was estimated that the dam is approximately 97 feet long, relatively consistent with that identified by DEC. Similarly, the width of the dam's concrete spillway section is identified by DEC as 26 feet, with that determined from the topographic survey and historical documents being 26.5 feet. Flow over the spillway discharges into Finkle Brook, which ultimately discharges into Lake George at a point approximately two (2) miles east of the dam.

Copies of the historic drawings and documents pertaining to the dam are included in Appendix A. A copy of the limited topographic survey performed of Edgecomb Pond Dam and Finkle Brook is also contained in Appendix A.

2.3 Physical Inspection

The physical condition of Edgecomb Pond Dam was inspected on August 25, 2015, by personnel of our office. Weather conditions on the day of inspection were warm and sunny. At the time of inspection, the reservoir level was approximately two (2) inches below the crest of the stop logs present across the crest of the dam's spillway.

Appendix B contains a Visual Inspection Checklist completed for the inspection and Appendix C contains photographs that are referenced on the Visual Inspection Checklist.

2.3.1 Earthen Embankments

Inspection of Edgecomb Pond Dam began with the dam's earthen embankments. No upstream embankment is present. The dam's concrete core wall is exposed along its length (Photograph No. 2) and extended approximately two (2) feet above the level of the reservoir at the time of inspection. It is believed that in 1991, a portion of the core wall was overlain with additional concrete as shown in Photograph Nos. 2 & 16, so as to mitigate further cracking of a portion of the core wall which had developed since its construction in 1977. No rip rap is present on the upstream side of the core wall (Photograph No. 2).

Due to the presence of the concrete core wall, the dam's crest is relatively narrow and poorly defined. In general, the crest measures less than 5 feet in width as measured from the downstream side of the core wall (Photograph No. 1). Grades along the crest generally decrease progressing right to left across the right embankment and this variation is estimated to be as great as 2 feet. One (1) animal burrow was observed along the downstream side of the core wall at the interface of the core wall and the downstream earthen embankment (Photograph No. 3). A large mass of concrete is present immediately downstream of the core wall near its junction with the right spillway training wall as shown in Photograph No. 4. The purpose for this concrete is unknown although it may represent concrete waste on-site during modification/thickening of the core wall.

The downstream slopes are visually estimated to be inclined at 1:2 (V:H). For both the right and left embankments, the spillway training walls turn back into natural ground a short distance beyond the spillway crest. This results in a very limited length of embankment to both the right and left side of the spillway. A uniform stand of field grass, generally 3 inches in height or less, is present across the right downstream embankment (Photograph No. 1). The left downstream embankment is surfaced with reedy vegetation, approximately 18 inches in height. Several large diameter trees are present just beyond the left downstream embankment-original ground interface. During past DEC inspections of the dam, trees had been observed on the downstream slopes of the embankments. These trees and their root systems appear to have been removed as no evidence of them was visible during the Safety Inspection.

2.3.2 Spillway & Discharge Channel

The dam's service spillway is a concrete weir of an 'ogee' configuration, measuring approximately 26.5 feet in width (Photograph No. 8). As determined from the limited topographic survey performed at the site, the spillway crest is believed to be at or near elevation 1,042.92 feet (NAVD 88 datum). Five (5) wooden timbers (stop logs) are present across the crest of the spillway and raise the reservoir level by approximately 36 inches above the spillway crest to elevation 1,045.92 to 1,046.03 feet. As shown in Photograph No. 9, the timbers were highly deteriorated and, in several locations, had rotted nearly halfway through their width. Slight leakage through the timbers was observed at the time of inspection. A steel channel is present along the top of the timbers and is secured by steel brackets/angles welded to the channel and bolted below to the concrete spillway.

Concrete abutment/training walls are present to each side of the spillway section and extend approximately 12.5 feet downstream of the spillway crest, at which point they turn into natural ground. A 4-inch diameter PVC pipe exits the right downstream training wall as shown in Photograph No. 5. No flow from the pipe was observed at the time of inspection, although there did appear to be flow occurring around the pipe at an estimated quantity of less than one-half (1/2) gallon per minute. Slight undermining of the pipe was observed. During past DEC inspections of the dam, seepage has been observed from this area as well as from around the left training wall. No seepage was observed emanating from around the left training wall at the time of the Safety Inspection.

Flow over the spillway crest enters an energy dissipater present at the base of the spillway (Photograph Nos. 10 & 17). Little to no rip rap was present within the energy dissipater.

The downstream channel is relatively broad and unobstructed for a distance of approximately 110 feet prior to flow being conveyed beneath Edgecomb Pond Road via a corrugated metal pipe measuring 6.5 feet wide by 5 feet high (Photograph No. 7). Downstream of the Edgecomb Pond Road crossing, the downstream channel is heavily wooded along its margins and undeveloped for several thousand feet prior to its crossing beneath Potter Hill Road. Below this point, numerous residences are present along the downstream channel.

Concrete comprising the spillway section was in poor to fair condition, with moderate deterioration noted of all surfaces. The spillway and training walls appear to have experienced moderate amounts of erosion, pitting, and cracking as shown in Photograph Nos. 12, 13, & 15. At several locations near the base of the spillway, the concrete had been severely eroded and, in isolated locations, its thickness was reduced by as much as 3 inches. Numerous cracks were present across the spillway and training walls as shown in Photograph No. 15. Similar deterioration of the dam's concrete surfaces has been observed by DEC during their past inspections of the dam. It appears that attempts have been made to repair the dam's concrete surfaces, as several concrete patches were observed along the spillway training walls. However, the concrete placed in these areas appears to be delaminating and spalling (Photograph No. 14).

2.3.3 Piping & Valves

A wood-framed gatehouse was formerly present immediately upstream of the spillway's right training wall. The gatehouse superstructure was removed sometime prior to 1986. It is believed that a 16-inch diameter raw water supply line was located within the gatehouse substructure and supplied raw water to the Town of Bolton. As noted on the 1950 Application for the Construction or Reconstruction of a Dam, at a point approximately 27 feet downstream from the spillway, a 12-inch diameter blow-off is present on the raw water main. Three (3) valves and/or valve stems are visible within the downstream channel at a point approximately 30 feet from the right training wall (Photograph No. 11), one of which is believed to be the blow-off valve. The functionality of the valves is unknown but it is reported by Town personnel that one (1) of the valves, located on the blow-off line, remains operational but is not frequently exercised.

Although the former gatehouse is no longer present, its concrete substructure is still visible as shown in Photograph No. 18. At some point in the 1990's, the substructure was infilled with concrete (Photograph No. 19). No information is known to exist regarding the arrangement of piping and valves within the former substructure or what work was performed prior to its' infilling with concrete.

3.0 HAZARD CLASSIFICATION

A dam failure analysis of Edgecomb Pond Dam was performed to develop an inundation map for the Emergency Action Plan prepared by C.T. Male in 2010. The inundation map confirmed that the dam failure would result in impacts to several downstream roadways, including Horicon Avenue and NYS Route 9N, and that the flood wave would impact over 30 homes. Approximately 15 of these homes are low lying and would be prone to flooding during heavy storm events, even without the dam's failure. The other 15 homes sitting on higher ground would not normally be impacted by heavy storm events, but would be impacted by the flood wave if the dam failed. For this reason, the High Hazard Classification, Class "C", is appropriate for the dam.

4.0 HYDROLOGIC & HYDRAULIC ASSESSMENT

4.1 Design Rainfall Analysis

With its high hazard classification, the spillway design flood for Edgecomb Pond Dam is one-half of the Probable Maximum Flood (i.e. the ½ PMF). The probable maximum flood is the flow resulting from the probable maximum precipitation (PMP), as defined in "*Hydrometeorological Report No. 51 (HMR-51), Probable Maximum Precipitation Estimates, United States, East of the 105th Meridian*", prepared by the U.S. Army Corps of Engineers (Corps) and the National Oceanic and Atmospheric Administration (last revised August 1980). Figures 18 through 47 from HMR-51 were used to obtain PMP values for different size watersheds and storm durations. These PMP values were input into the Corps' HMR-52 computer program to produce the precipitation hyetograph for the probable maximum precipitation of the dam's watershed. The precipitation hyetograph for the PMP occurs over 72 hours and has a total precipitation during that time period of 33.65 inches. Of this amount, 16.68 inches falls within a 2-hour period at the mid-point of the hyetograph. Appendix D, Rainfall Data, contains the HMR 51/52 probable maximum precipitation input and output data.

4.2 Watershed Analysis

Delineation of the watershed tributary to Edgecomb Pond was performed using topographic information shown on the Bolton Landing USGS 7.5-minute quadrangle maps (Refer to Figure 2, Watershed Delineation Map). The total watershed area draining to the pond was computed to be 1,308 acres (2.04 square miles).

Appendix E, Soils Information, contains detailed soil data obtained from the USDA's Web Soil Survey and Aerial Mapping. From this soil survey, it was determined that there is a wide range of soil conditions within the watershed. Of the total watershed, approximately 40% of the soils belong to Hydrologic Soil Group "B", 40% percent to Group "C" and 20% to Group "D." Land cover within the watershed is heavy forest.

The watershed modeling included four distinct "sub drainage areas" with three sub-areas for land and one sub-area for Edgecomb Pond itself. The land sub-areas were chosen based upon the variation in soil properties and land slope across different portions of the watershed.

Appendix F, HydroCAD Modeling Reports, contains specific information related to each sub-watershed, including land use breakdown, curve number calculations and lag time. Runoff from the watershed was evaluated using HEC-HMS for the ½ PMF storm event and the runoff from HEC-HMS was input into HydroCAD for ease of modeling the spillway. The basic runoff computation methodology employed by HEC-HMS is *Technical Release 20* (*TR-20*) developed by the USDA Soil Conservation Service.

4.3 Spillway Capacity Analysis

In modeling the dam's failure for development of the Emergency Action Plan, it was determined that the dam's existing spillway was incapable of passing the spillway design flood for its high hazard classification. Accordingly, for this Engineering Assessment, the spillway capacity analysis was expanded to include the presence, and assuming the construction of, an auxiliary spillway as well as increasing the height of the existing spillway abutment walls and the dam's core wall.

A limited topographic survey of Edgecomb Pond Dam was performed to establish the configuration, dimensions and crest elevation of the dam's existing spillway (with the stop logs left in place). From this survey, it was determined that the existing spillway, hereinafter referred to as the service spillway, has a stop log crest elevation of 1,046.03 feet and a width of 26.5 feet. The auxiliary spillway proposed for construction and shown on the plan contained in Appendix G, Proposed Auxiliary Spillway Plan, has a crest elevation of 1,047.80 feet, a width of 50 feet and a breadth of 50 feet. The proposed upgrades to the dam include raising the height of the spillway's abutment walls and the dam's core wall to approximately elevation 1,054.00 feet. The total length of the top of the raised core walls was modeled as 71 feet.

The half-PMF flow determined from HEC-HMS model was converted into HydroCAD for the spillway capacity analysis. The full PMP event was simulated over the watershed and, consistent with Section 5.2 of DEC's publication *"Guidelines for Design of Dams,* flow from the watershed was reduced by 50 percent before being routed over the spillway. The peak inflow during the ½ PMF storm event was determined to be 3,657.21 cubic feet per second (cfs), occurring at hour 40.5 of the 72-hour design storm event. The peak outflow over the service spillway/auxiliary spillway combination during the ½ PMF storm event was determined to be the same as the inflow, with a peak water surface elevation of 1,053.08 feet, occurring at hour 40.5. This is reflective of the fact that Edgecomb Pond is small and provides for little to no attenuation during high flow events. The flow breakdown during the SDF is 2,065 cfs over the service spillway and 1,595 cfs over the proposed auxiliary spillway. Under this scenario, there is 0.9 feet of freeboard between the peak water surface elevation and the elevation proposed for the raised core wall. Hence the current requirement of Section 5.3 of DEC's publication *"Guidelines for Design of Dams"* that the spillways of dams have adequate spillway capacity to pass the design flood without overtopping will be met with the construction of the auxiliary spillway and modifications made to the dam.

The proposed auxiliary spillway will be armored with turf reinforcing mat or rip rap to provide for a non-erosive surface during the spillway design flood. Final determination of the type of stabilization will be provided during the final design phase for the auxiliary spillway and dam modifications.

4.4 Storage Evacuation Calculations

DEC's current publication "*Guidelines for Design of Dams*" contains specific time requirements for lowering of the water level behind dams. Each requirement is identified below and the results of the evacuation calculations presented in Appendix H, Evacuation

Calculations. The stage/storage profile for the pond was estimated based upon depth information from the Town of Bolton and the topography of the land immediately adjacent to the pond.

4.4.1 Service Spillway Evacuation

Section 6.5.6 of DEC's "*Guidelines for Design of Dams*" requires that the service spillway have sufficient capacity to evacuate 75 percent of the storage between the auxiliary spillway crest and the service spillway crest within 7 days, assuming no inflow into the pond. At the normal pool (service spillway) elevation of 1,046.03, there is storage of 740 acre-feet. At the proposed auxiliary spillway crest elevation of 1,047.80, there is storage of 805 acre-feet. Evacuation of 75 percent of the storage between the auxiliary and service spillway crests equates to a storage volume of 756 acre-feet, which corresponds to elevation 1,046.52 feet. The calculations presented in Appendix H indicate that after 11.8 hours, the water level drops to elevation 1,046.52 feet, hence the evacuation criterion of Section 6.5.6 of the DEC guidelines will be met with the construction of the auxiliary spillway and modifications made to the dam.

4.4.2 Auxiliary/Service Spillway Combination Evacuation

Section 6.5.5 of DEC's "*Guidelines for Design of Dams*" requires that assuming no inflow, the auxiliary/service spillway combination have sufficient capacity to evacuate the storage between the maximum high water and the auxiliary spillway crest within 12 hours. Calculations presented in Appendix H indicate that it takes 2.75 hours to drop the water level from the maximum high water (elevation 1,053.08) to the auxiliary spillway crest (elevation 1,047.80). Hence, the evacuation criterion of Section 6.5.5 of the DEC guidelines will be met with the construction of the auxiliary spillway and modifications made to the dam.

4.4.3 Low-Level Drain

Section 7.1 of DEC's "*Guidelines for Design of Dams*" requires that the low-level drain(s) have sufficient capacity to discharge 90 percent of the storage below the spillway crest within 14 days, assuming no inflow into the reservoir. The storage at the normal pool is 805 acre-feet, and the bottom of the impounded water is at elevation 1,038 feet, where the storage is 456 acre-feet. Therefore, discharging 90 percent of the impounded storage volume would equal 484 acre-feet, occurring at elevation 1,038.88 feet. As the 12-inch diameter blow-off on the 16-inch diameter main line is reported to serve as the dam's low-level drain, the functionality of the 12-inch drain valve and other valves controlling flow through the 16-inch main line must be determined/verified. If the low-level drain is determined to be functional as reported by Town personnel, calculations presented in Appendix H indicate that it will take 10.2 days to drop the water level from the normal pool to elevation 1,038.88 feet. Hence, the low level drain requirements of Section 7.1 of the DEC guidelines are met provided the valves of the low-level drain and 16-inch main are proven to be operable and flow out of the blowoff pipe is confirmed. If either of these two conditions cannot be satisfied, then pumps with a total capacity of at least 7,825 gallons per minute must be rented to meet this evacuation criterion.

5.0 SUBSURFACE INVESTIGATION PROGRAM

One (1) test boring was advanced through the right embankment of the dam on January 27, 2010. This work was performed by TransTech Drilling Services, Inc. of Schenectady, New York using a Central Mine Equipment Model 550-X all-terrain drill rig. The approximate location of the test boring, as established by tape measurement from existing site features, is shown superimposed on the limited Topographic Survey included in Appendix A.

The test boring was advanced and cased against collapse through rotary drilling of 4¼ inch inside diameter hollow-stem augers. As the augers were advanced, the overburden was sampled and its penetration resistance determined in accordance with the procedures of ASTM Designation D-1586, "*Standard Method for Penetration Testing and Split-Barrel Sampling of Soils*" and using an automatic safety hammer. The sampling and penetration resistance testing was performed in consecutive 2-foot intervals through the height of the embankment and extended several feet into the underlying foundation soils. Below this depth, the sampling and penetration resistance testing was conducted at intervals of five (5) feet or less until termination of the test boring upon encountering refusal at a depth of 18.6 feet below the embankment crest.

A representative of C.T. Male monitored the advancement of the test boring, recorded the standard penetration resistances, and placed representative portions of the samples in jars. The samples were brought to the soils laboratory for classification by a geotechnical engineer. A Subsurface Exploration Log was then prepared for the test boring to present the soil classifications and the records maintained during the test boring's advancement. This log is presented in Appendix I, Subsurface Exploration Log, along with a sheet and key that explains the terms and symbols used in its preparation.

6.0 SUBSURFACE CONDITIONS

6.1 Embankment

In general, the soils found to comprise the dam's embankment consisted of sand which contained little silt and trace amounts of gravel. At the test boring location, the embankment soils were found to extend to a depth of 6 feet below the existing ground surface. Standard penetration resistances (N-values) within the embankment fill ranged from 24 to 44, indicating that the embankment soils are of a firm to compact relative density.

6.2 Foundation Soils

Glacial till was encountered beneath the dam's embankment. In general, the glacial till consisted of sand with little amounts of silt and coarse sand and gravel embedded within its soil matrix. Occasional cobbles were encountered throughout the deposit. The test boring was terminated within this deposit at a depth of 18.6 feet below the embankment crest upon encountering auger and sampler refusal. N-values recorded within the glacial till ranged from 38 to greater than 100, indicating that the deposit is of a compact to very compact relative density.

6.3 Groundwater

The depth to groundwater was measured at the test boring location at a depth of 14.4 feet below the ground surface. It should be noted that this depth was measured shortly after completion of the test boring and, due to the fine-grained nature of the soils present beneath the embankment, may not be representative of the depth to the actual groundwater table.

7.0 STABILITY ANALYSIS

7.1 Spillway Stability

The stability of the dam's service spillway was performed in accordance with the procedures presented in DEC's publication *"Guidelines for Design of Dams*". The following four (4) load cases were analyzed:

- **Load Case I** – Normal Loading Condition – Water Surface at Normal Pool Level, Elevation 1,046.0 feet.
- **Load Case II** – Normal Loading Condition - Water Surface at Normal Pool Level plus Ice Load of 5,000 pounds per lineal foot (plf).
- **Load Case III** - Design Loading Condition - Water Surface at Spillway Design Flood Level, Elevation 1,053.08 feet.
- **Load Case IV -** Seismic Loading Condition - Normal Loading Condition with Water Surface at Normal Pool Level plus Pseudo-Static Earthquake Loading.

Elevations of the spillway section utilized in the analysis were based upon the limited topographic survey performed of the dam. Dimensions shown on the drawings appeared to be relatively consistent with those physically measured during the Safety Inspection and, as such, were utilized in the stability analyses.

8.0 STABILITY RESULTS

Table 1 on the following page presents a summary of the service spillway's computed factors of safety against sliding along a horizontal plane taken along the base of the dam's spillway section, or approximate elevation 1,029.00 feet. As the spillway section is somewhat stout, the entire spillway, training/abutment walls and footings, buttress walls and apron were analyzed as a single unit and included concrete to be placed in raising the height of the spillway and adjacent walls as required to pass the SDF. Passive earth pressure resistance to sliding provided at the end of the spillway apron was ignored but the net passive resistance provided by the core wall and buttress walls to each side of the spillway was included in the analysis. The concrete slab which extends upstream of the spillway was assumed to be cracked at its connection to the spillway and accordingly did not contribute to the spillway's stability analysis.

Appendix J contains the calculations supporting these results. As shown in Table 1, with the exception of Load Case III, all of the load cases have safety factors against sliding in excess of the minimum required values. As the calculated factor of safety for Load Case III was well below the minimum required value, the spillway will need to be anchored to the underlying soil/bedrock to provide an adequate safety factor for this load case.

Table 1
Computed Factors of Safety

Load Case	Load Case Description	Friction Factor of Safety		Location of Resultant Force
		Calculated	Required	
I	Normal Loading Condition - Water Surface at Normal Pool Level	3.09	1.50	Middle Third
II	Normal Loading Condition - Water Surface at Normal Pool Level plus Ice Load of 5,000 plf	1.43	1.25	Middle Half
III	Design Loading Condition - Water Surface at Spillway Design Flood Level	1.01 See Note 1	1.25	Middle Half
IV	Seismic Loading Condition - Normal Loading Condition w/ Water Surface at Normal Pool Level plus Pseudo-Static Earthquake Loading	1.66	1.00	Within Base

NOTE 1: SAFETY FACTOR OF AT LEAST 1.25 WILL BE PROVIDED WITH INSTALLATION OF 3 SOIL/ROCK ANCHORS.

9.0 CONCLUSIONS & RECOMMENDATIONS

Documentation reviewed for the dam included historic documents & drawings, past inspection reports compiled by DEC, the dam's IMP prepared in June 2010 and revised September 2010, and the dam's EAP prepared in October 2010. Based upon our review of this information and a Safety Inspection performed of the dam, it is our opinion that while the dam does not possess a rating of "unsafe" or "unsound", it does not meet all of DEC's dam safety criteria. The following subsections summarize the conditions requiring correction and a schedule recommended for addressing the same.

9.1 Spillway

Construction of the auxiliary spillway and raising of the core wall and training/abutment walls of the existing dam and spillway must be performed to provide the dam with the capacity to pass the spillway design flood without overtopping. Along with this work, the flashboards across the spillway should be removed and the spillway crest raised to elevation 1046.0 feet, the current top of flash boards elevation. Three (3) soil/rock anchors should be installed and their heads embedded within the concrete placed to raise the crest elevation to provide the spillway with an adequate factor of safety against sliding under the spillway design flood.

Deterioration of the dam's concrete was generally moderate, with erosion and spalling noted of nearly all surfaces. At isolated locations, the depth of erosion was observed to be significant, on the order of 3 inches. There is clear evidence of attempts at past repairs to the spillway and training walls, although the repairs have begun to delaminate and spall. It is recommended that a concrete overlay, at least 6-inches in depth, be installed over the existing spillway surface. This overlay should be lightly reinforced and doweled into the existing spillway and adjacent abutment/training walls.

Structural cracking of the spillway abutment/training walls were observed. At the time of inspection the cracks were observed to be tight and no seepage or loss of ground or differential movement was noted to have occurred. It is recommended that the cracks within these walls be sealed through injection of a polyurethane grout.

The right training wall of the spillway should be extended to match the length of the left training wall and the remains of the concrete apron between the walls removed and the apron replaced. Consistent with the stability analyses/assumptions, the new reinforced concrete apron should be at least 18 inches in depth and be doweled into the training walls and the spillway itself.

9.2 Pipes & Valves

No information was recovered regarding the former gatehouse and its operation. It is known that the substructure of the former gatehouse was infilled with concrete, presumably during the 1990's, and no records were recovered regarding what work was done at that time. It is known that a 16-inch diameter raw water distribution main exited this structure and, prior to 1995, provided raw water to the former Town of Bolton Water Treatment Plant. Reportedly the raw water main was severed at some point downstream of the spillway once the new water treatment plant was operational. From historic documents, it is known that approximately 27 feet downstream of the spillway a 12-inch diameter blow-off line was present off of the raw water main. Three (3) valves are visible within the downstream channel and are believed to have controlled flow through these lines. It is reported that the valve on the 12-inch blow-off line is functional, although infrequently used. The other valves are believed to be non-functional and are not used.

The arrangement of piping and valves within the former gatehouse substructure is unclear, and it is unknown if the former raw water main is still functional. It is recommended that the functionality of the raw water main be determined, as the 12-inch diameter blow-off on this line serves as the dam's low-level drain. In the event that the line is found to be functional, it is recommended that the low-level drain valve be regularly exercised. If the drain is not functional, then pumps with a total capacity of at least 7,825 gallons per minute must be rented to achieve this drawdown requirement. The Inspection & Maintenance Plan should be edited to include a description of the functionality of the low-level drain and the size and rental location(s) of the pump(s) which would be required to comply with the 14 day evacuation criterion.

9.3 Embankment

Several large diameter trees present beyond the left downstream embankment-original ground interface should be cut down and their stumps and root systems removed under the direction of a licensed Professional Engineer and receiving a permit to do so. Upon removal of the existing trees, these areas should be seeded and a uniform stand of grass established.

The embankment slopes should be maintained in a groomed condition to facilitate their visual inspection for signs of seepage, erosion, animal burrows, and slope instability. Field grass and weeds may remain in place but should be mowed and/or weed-whacked on a regular basis. Any brush which is present should be removed. This maintenance work should extend from the reservoir level on the upstream slope, across its crest, and down its downstream slope to a point at least 10 feet beyond where the embankment meets original ground along its downstream toe and embankment-original ground interface. The growth should be maintained to a height on the order of three (3) inches or less. Where barren soil is exposed by this work, it should be seeded and mulched to promote establishment of a vegetative cover. All of the work indicated above is not considered urgent but, once initiated, should be performed on a regular basis.

One (1) animal burrow was observed at the time of inspection. Rodents and animal burrows/nests should be dealt with in accordance with the recommendations provided in the dam's Inspection & Maintenance Plan.

9.4 Recommended Schedule of Work

Each of the above described deficiencies should be corrected in order to bring the dam into total compliance with DEC regulations. Rudimentary remedial actions such as the capture and relocation of animals burrowing into the dam and routine maintenance of the dam's concrete surfaces do not require a permit and can be performed by maintenance personnel without incurring significant costs. Other remedial work required to bring the dam into regulatory compliance carries with it the preparation of plans and specifications for addressing the same. Adequate time for the review of the plans and specifications and for permitting of the work by DEC and other regulatory agencies must be provided as well as for the Town of Bolton to be provided a budget estimate for the remedial work and to make funds available for its completion. With these considerations and our opinion that the dam does not have a Condition Rating of "unsafe" or "unsound", the schedule shown in Table 2 is recommended to address the deficiencies.

Table 2
Schedule for Addressing Deficiencies

Work Item	Schedule
Preparation of Plans and Specifications for Improvements	2017
Submit Project Plans & Specifications to DEC – Allow 3 to 6 months for review	2017
Improvements to Existing Spillway	2018
Construction of Auxiliary Spillway	2019

10.0 CLOSURE

This Engineering Assessment of Edgecomb Pond Dam has been based upon the results of an Initial Safety Inspection, a structural stability analysis of the dam's spillway section, and hydraulic and hydrologic analyses of Edgecomb Pond and the dam's spillway, respectively. Several deficiencies have been identified which prevent it from being in compliance with DEC regulations. Recommendations and a proposed schedule for implementing the same have been provided in this Engineering Assessment.

Respectfully Submitted,

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