Shabomeka Lake Dam Rehabilitation Design Report

Mississippi Valley Conservation Authority (MVCA)
June 2021
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Stantec

Shabomeka Lake Dam Rehabilitation Design Report

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Prepared by:
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Sign-off Sheet

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Executive Summary

The structure at Shabomeka Lake Dam has been experiencing significant deterioration due to settlement of the old timber structure below the current concrete structure. The existing structure was to be removed in 1989. We have proposed in this report that the existing concrete and timber structure be removed and replaced with a structure based on the embankment and hydraulic/operational requirements. The new structure is a reinforced concrete structure of similar style as the existing one but with a bascule gate, and it will be installed at the same location.

In preparation of the Class Environmental Assessment (EA) and to ensure that all environmental effects are considered in the dam rehabilitation/reconstruction design, the following studies were prepared:

- Environmental and biological inventory of existing conditions based on the biological assessment and inventory. Biological impact and species at risk were identified in the EA study.
- Hydrologic and hydraulic studies to ensure the proposed design is in compliance with the Ontario Ministry of Natural Resources and Forestry (MNRF) regulations and with the Canadian Dam Association (CDA) 2013 Guidelines including a design concept report with recommendation of the preferred option. Biological impact and species at risk were identified in the EA study.

Following the analysis and review of the previous studies, including the available dam surveys, dam inspections and geotechnical investigation, and based on hydrological and hydraulic studies and environmental inventory performed by Stantec, we recommend the rehabilitation of the dam instead of the complete reconstruction. This recommended rehabilitation leads to the following:

- Reduction of the environmental impact of works;
- Limitation of the area of construction works within the existing dam location;
- Avoidance of the implantation of a significant cofferdam by working between September and December; and
- Reduction in cost in comparison with the dam reconstruction option.

The development of the design concept was completed in conjunction with the evaluation of the environmental and biological impact and with respect to the hydraulic, geotechnical, and structural conditions.

The impacts of the construction activities were performed, and mitigation measures were included in the design.

The Class B cost estimates for the rehabilitation works of the Shabomeka Lake Dam is about **$1,533,990** including 10% contingency, material testing, provisional allowance for additional geotechnical investigation, sediment removal and grouting works but excluding contract administration, resident supervision and HST.

Abbreviations

AEP	Annual Exceedance Probability
CDA	Canadian Dam Association
DSA	Dam Safety Assessment
EA	Environmental Assessment
FOS, FS	Factor of Safety
HPC	Hazard Potential Classification
IDF	Inflow Design Flood
LOL	Loss of Life
LOWL	Low Operating Water Level
LRIA	Ontario Lakes and Rivers Improvement Act
MNRF	Ontario Ministry of Natural Resources and Forestry
MOWL	Maximum Operating Water Level
MVCA	Mississippi Valley Conservation Authority
NBCC	National Building Code of Canada
OMS	Operation Maintenance and Surveillance
PAR	Population at Risk
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
RQD	Rock Quality Designation
SDFS	Technical Bulletin Structural Design and Factors of Safety Ontario Ministry of Natural Resources, August 2011
SEBJ	Energy Society of James Bay
SPT	Standard Penetration Test
Stantec	Stantec Consulting Ltd.
USBR	United States Bureau of Reclamation
VST	Vane Shear Test
WL	Water Level

1.0 INTRODUCTION

The Mississippi Valley Conservation Authority (MVCA) mandated Stantec Consulting Ltd. (Stantec) to complete a Class Environmental Assessment (EA) and Preliminary Design for the reconstruction of the Shabomeka Lake Dam near the village of Cloyne in Ontario.

The dam was built in the 1950s with earth embankments and a wooded sluice gate that was later changed to concrete. In 1988, rehabilitation works were carried out on the concrete control structure and a clay backfill was added to reduce seepage. However, after the rehabilitation, Ontario Hydro determined that this work was temporary since the new structure did not meet the overturning/sliding condition. The water level was lowered in the winter season for safety matters during the spring freshet. Since 1988, no work has been performed on the dam. A comprehensive dam safety assessment was completed in 2005, and the following **Table 1-1** lists a statement of recommended work.

Table 1-1: 2005 DSA Recommendations

Project Component	2005 DSA Recommendations	Follow-up
Embankment	• Perform work to meet rotational failure requirement within 5 meters of the control structure; • Additional erosion protection on the downstream side of the embankments adjacent to the control structure; and • Repair the settlement on the embankment at the junction of the embankment and the control structure.	Part of this design report.
Emergency spillway	• Construction of an overflow weir for events higher than IDF.	Not required but requested by the MNRF during the LRIA review.
Control structure (concrete)	• Restoration of the concrete at areas of erosion, chipping, and cracking.	Reconstruction of this structure is part of this design report.
Winch system	• Should be replaced.	Already done.
Maintenance	• Monitor cracks and areas of abrasion due to freeze-thaw, areas of rust along the steel beam supports at the upstream side, settlement on the upstream gabion.	
Safety	• Improve signage; • Modify the handrails and install a gate to limit public access to the control structure; and • Re-work dam benchmark protection to eliminate trip hazard by trimming the height of the protective pipe and installing a cap.	Already done. Not completed. Already done.
Reconnaissance surveys	• Aerial reconnaissance surveys should be undertaken during seasonal periods and flood	Not required.

Project Component	2005 DSA Recommendations	Follow-up
	events to obtain a visual record for future reference and assessment.	
OMS Manual	- Prepare a separately bound Operation, - Maintenance, and Surveillance plan.	Already done (to be revised after dam rehabilitation).

In 2016, an assessment of the Shabomeka Lake Dam was performed. **Table 1-2** below lists the proposed works to be performed.

Table 1-2: 2016 Assessment Report

Project Component	2016 Assessment Recommendations	Follow-up
Embankment	- Raise the embankment and road elevation; - Add gravel or drainage blanket on the downstream slope; and - Re-grade the downstream slopes so that there is an adequate factor of safety against rotational failure of the embankments.	Part of this design report.
Emergency discharge	- Add emergency discharge capacity.	Part of this design report.
Control structure (concrete)	- Re-build the control structure situated slightly to the north of the existing structure to avoid the known bedrock fault.	Part of this design report.
Studies	- Update the hydrologic modelling; and - Evaluate the embankment freeboard requirements.	Part of this design report.

Following the 2016 Assessment, a design of the dam reconstruction was recommended with rough cost estimates, but no engineering study was performed. This design is presented in **Table** 1-3 below, and it was the basis of the Stantec engineering study. However, it appears that the embankment rehabilitation could be optimized.

The removal and reconstruction of the embankment proposed in the 2016 cost estimate was reviewed, but it showed that rehabilitation was a better option. The cost of replacement of the embankment would be higher than rehabilitation and would have had higher impact on the environment. The proposed concept by Stantec was oriented towards lowering the impacts on the environment using the results of Stantec's Environmental Assessment -Environmental Inventory performed as part of this project.

Following the site visit by Stantec, it appears that the embankment movement is partially caused by the bad condition of the wooden sheet pile. Therefore, Stantec chose to investigate a solution for the replacement of this sheet pile. This report proposes a solution of cement-bentonite core.

The design presented in this report is the subject of an Environmental Assessment that was completed in 2020, following the requirements of Conservation Ontario's Class Environmental Assessment (Class EA) for Remedial Flood and Erosion Control Projects (2002, as amended in 2013). The Class EA study has examined the proposed work described herein as one of multiple alternatives to address the structural deficiencies of the existing dam and recommends the implementation of the work described herein. Additional consultation will be undertaken as a part of the EA study to confirm or modify the recommended alternative.

Table 1-3: 2016 Design for Cost Estimation

	2016 Assessment Recommendations	**Follow-up**
Cofferdam / diversion	• Construction of a cofferdam and temporary diversion channel (i.e., earthen cofferdams of less than 1.5 metre in height);	Part of this design report
Control structure	• Demolition and removal of the existing concrete control structure • Construction of a new concrete control structure complete with concrete retaining walls. – The new structure generally matches the existing structure; however, longer retaining walls are required to accommodate the flatter embankment side slopes; – The new structure is founded at elevation 268.5 m with a deck elevation of 272.2 m; and – The existing gate hoist is re-used as part of the new control structure.	Part of this design report
Embankment	• Removal of the existing embankments to about elevation 268.0 metres over a total length of about 60 metres; • Construction of new embankments up to elevation 272.2 metres over a total length of 60 metres – 3H to 1V slopes; – Full riprap protection along both the upstream and downstream faces; – A width of 5 m across the top of the embankment; – A 40-m long riprap-lined emergency spillway having a width of about 3 m; and • The crest of the existing embankment is raised between about 20 and 60 m south of the existing control structure (i.e., the dip in the access roadway) to elevation 272.2 m.	Optimization performed in this design report

2.0 HYDROLOGIC STUDY

2.1 REVIEW OF THE 2005 DSA HYDROLOGY

A hydrologic study was performed for the 2005 Dam Safety Assessment. As part of the design of the new dam, it was required to update the inflows, since the assessment was done more than 10 years ago. The 2005 DSA used three methods to determine the inflows.

- **Method 1 – Mass balance from historical records at Shabomeka Lake**: The mass balance of Shabomeka Lake was performed using the 1990-2003 available data. Note that this method used the lake evaporation that "was estimated from a water budget analysis completed for the study area in 2003." Since this 2003 water budget analysis is not named (and no document in the list of reference is dated 2003), it is impossible for Stantec to update these results. The spring (annual) peak flow (15.8 m^3/s) of this method was chosen, and the shape of the hydrograph was derived from the April 1998 hydrograph computed with this mass balance hydrograph. The results of this mass balance analysis were provided to Stantec and it made it possible for Stantec to obtain the spring IDF shape.
- **Method 2 – Correlation to adjacent Gauge Site:** A regression was made between the Clyde River station (02KF013) and Shabomeka Lake, using the 11 years of records (1990-2003). This station is still functioning, and the results can be updated.
- **Method 3 – OTTHYMO simulation**: A simulation was performed using various return periods. The model is calibrated using the June 2002 exceptional rain event. The summer (seasonal) peak flow (8.55 m^3/s) of this method was chosen, and the shape of the hydrograph was derived from the exceptional rain that occurred in June 2002. The OTTHYMO simulation is not available, and it is impossible for Stantec to update these results. The hydrograph is plotted and Stantec used this shape to update the hydrology.

2.2 UPDATE OF THE CORRELATION TO ADJACENT GAUGE SITE

The Clyde River Station (02KF013) is still functioning, and data was retrieved (1972-2016). Using the updated maximum yearly(spring) and seasonal (summer) flows, the same regression analysis as described in the 2005 DSA was applied ($Q_{Shabomeka} = 0.125Q_{Clyde} + 0.54$). Note that this linear regression equation cannot be updated, since it depends on the mass balance methods that were performed on the years 1990-2003.

Stantec identified a closer hydrometric station, Mississippi River below Marble Lake (02KF016), which is located downstream from Shabomeka Dam (therefore it includes the watershed of Shabomeka Lake). It has recorded data from1988-2016.

All the data at Clyde River was initially retrieved for the update. The points shown in Figure 4.2 of the 2005 DSA represent the corresponding flow at Clyde River and Shabomeka Lake between 1990 and 2003. Therefore, it is possible to read the Clyde River flow on the figure and associate this flow with the date it occurred. Then, the flow for the same date in the Mississippi River (02KF016) was retrieved. Therefore, **Figure** 2-1 below displays Shabomeka Lake vs. Mississippi River. A regression analysis between Mississippi River and Shabomeka Lake was then performed.

It was chosen a second order polynomial regression as the best fit. The following regression equation was extracted from **Figure** 2-1.

$$Q_{Shabomeka} = 0.0043Q_{Mississipi}{}^{2} + 0.0737Q_{Mississipi}$$

This regression analysis is valid for flows in Shabomeka Lake between 0 m3/s and 12 m3/s. An extrapolation of this regression analysis is performed for flows over 12 m3/s.

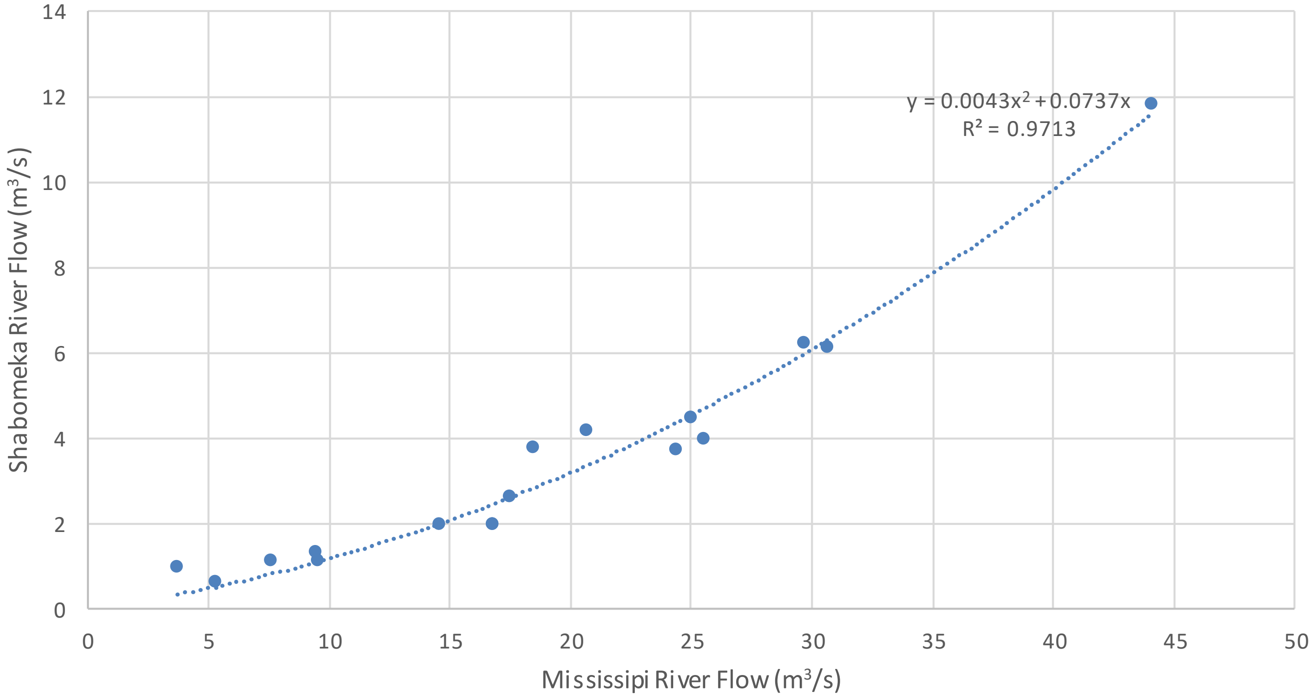

Figure 2-1: Shabomeka Lake Inflow vs Mississippi River

2.3 PEAK FLOW RESULTS

A frequency analysis was performed on the Clyde River Station (02KF013), and the Log-Pearson III frequency distribution was used. Then, the results of the frequency analysis were used to update the results using the regression analysis as described in the 2005 DSA.

A frequency analysis was also performed on the Mississippi Station (02KF016), and again the Log-Pearson III frequency distribution was used. Then, the results of the frequency analysis were used to update the results using the regression analysis as described in Section 2.2 of this report.

Table 2-1: Peak Flow Update

	Correlation using Clyde River Station (02KF013)		Correlation using Mississippi River Station (02KF016)	
Recurrence	Spring (annual)	Summer (seasonal)	Spring (annual)	Summer (seasonal)
2	3.93	1.37	4.01	1.15
5	5.31	2.01	6.49	2.70
10	6.26	2.53	8.59	4.32
20	7.18	3.10	11.01	6.43
25	7.48	3.30	11.87	7.23
50	8.41	3.96	14.82	10.15
75	8.96	4.38	16.76	12.20
100	9.35	4.70	18.25	13.83
1,000	12.67	7.78	34.36	33.19
5,000	15.21	10.63	51.57	55.76
10,000	16.37	12.07	61.05	68.54

2.4 PEAK FLOW COMPARISON WITH 2005 DSA

To simplify the comparison, only the 100-year recurrence is compared, since it is the IDF of the dam and is the value that will be used for the design. The three methods of the 2005 DSA are presented, along with the 2016 updates using the two available flow stations (Clyde and Mississippi Rivers). **Table** 2-2 shows that the highest peak flow (spring and summer) is observed with the 2016 correlation using the Mississippi River Station.

Since the results of the Mississippi River Station showed higher flows, the results from this station was used. Moreover, Shabomeka Lake is included in the Mississippi River Station watershed.

Peak flow discussion: The data available at both hydrometric stations do not give instantaneous peak inflow information and consist of exactly the same data that were used in the 2005 DSA method 2 (Correlation to adjacent gauge site). The instantaneous peak flow is used for design of a bridge/culvert design since the inflow cannot be stocked upstream, and this instantaneous peak flow needs to be conveyed by the bridge/culvert. However, for a dam, the inflow is routed in the reservoir and the instantaneous peak flow is not seen at the dam outlet, but the outflow consists of a routed average flow.

Regulation effect discussion: The Mississippi River gauge (02KF016) is located 5 km downstream from Lower Mazinaw Lake and could be partially influenced by the regulation effect of Mazinaw Lake (Environment Canada indicates that the regulation type is "natural"). The effect of regulation caused by a dam is usually that the routing of the dam leads to lower outflows. However, from all the analyzed methods, this method led to the highest peak flow; therefore, the conservative design should consider the highest flow (Mississippi River gauge correlation).

Table 2-2: 100-year Peak Flow Comparison

Method	**Spring (Annual)**	**Summer (seasonal)**
2005 DSA Method 1 Mass balance from historical records at Shabomeka Lake:	15.8	4.23
2005 DSA Method 2 Method 2-Correlation to adjacent Gauge Site	9.86	5.50
2005 DSA Method 3 Method 3-OTTHYMO simulation	N/A	8.55
2016 Update Correlation using Clyde River Station	8.96	4.38
2016 Update Correlation using Mississippi River Station	**18.25**	**13.83**

2.5 CLIMATE CHANGE

To account for climate change that causes an increase in rainfall, and eventually inflow, it was chosen to base the increase on the MTO method. The IDF curve at the location of the dam was extracted from the MTO IDF Curve Lookup application[1]. The IDF curves for the year 2017 and 2117 were retrieved.

It was chosen to use the longest rainfall event shown on the curves (24h), since the 2005 DSA showed that the most critical event occurs over three days (the IDF curves do not display three days rainfall event). The total rainfall depth for 2017 at the dam location is 122.4 mm, and it is forecasted to be 136.8 mm in 2117. This corresponds to an increase of rainfall of 11.7%. Therefore, the flow was majorated with this value of 11.7%.

The majorated peak flow values are presented in **Table** 2-3.

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[1] http://www.mto.gov.on.ca/IDF_Curves

Table 2-3: Climate Change Increase

Recurrence (Years)	Spring (Annual)	Summer (Seasonal)
2	4.48	1.28
5	7.25	3.02
10	9.61	4.83
20	12.31	7.19
25	13.26	8.08
50	16.56	11.35
75	18.73	13.64
100	**20.40**	**15.45**
1,000	38.40	37.09
5,000	57.64	62.33
10,000	68.23	76.61

2.6 HYDROGRAPH SHAPE

2.6.1 Summer Hydrograph

A calibrated hydrograph shape was developed in the 2005 DSA using the 2002 exceptional rain event in the OTTHYMO simulation. Stantec used the shape of this hydrograph (3-day rainfall) plotted on page 4-9 of the 2005 DSA report.

Figure 2-2 below shows the hydrograph with the 2016 updated peak flow of 15.45 m³/s.

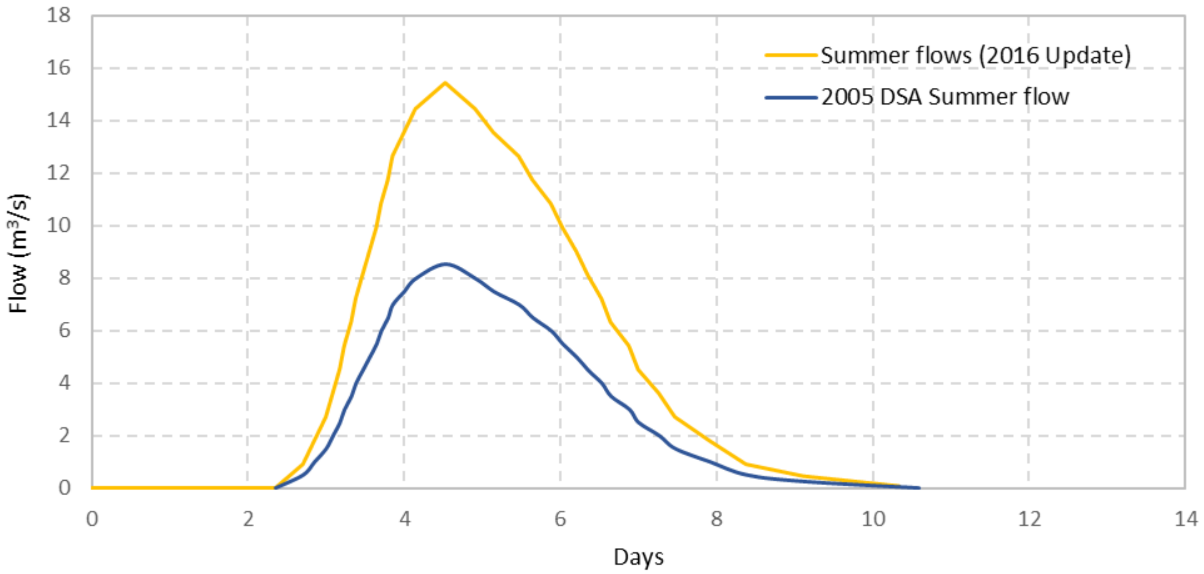


Figure 2-2: Shabomeka Lake Inflow Summer Hydrograph

2.6.2 Spring Hydrograph

The spring hydrograph of the 2005 DSA used the largest snowmelt event occurring over the 11 years of inflow generated by the Method 1-Mass Balance Analysis. This event occurred in April 1998. The 2005 DSA report does not plot the winter hydrograph, so the results of the mass balance analysis were used to extract the shape of the hydrograph. **Figure** 2-3 shows the hydrograph with the 2016 updated peak flow of 20.4 m³/s.

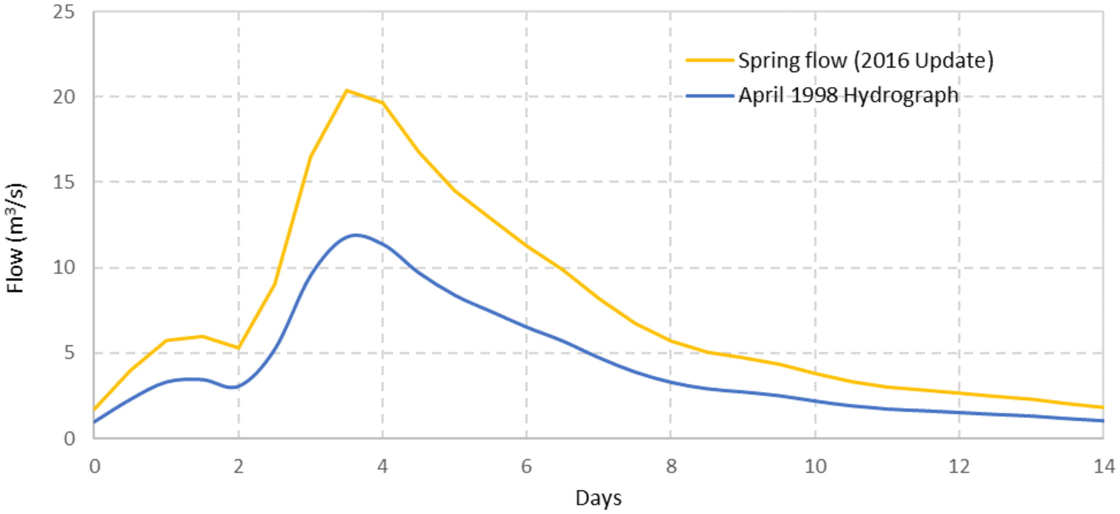


Figure 2-3: Shabomeka Lake Inflow Spring Hydrograph

2.6.3 Comparison of summer and spring Hydrograph

The spring and the summer hydrographs previously computed in this 2016 update are shown on **Figure** 2-4 below for comparison purposes.

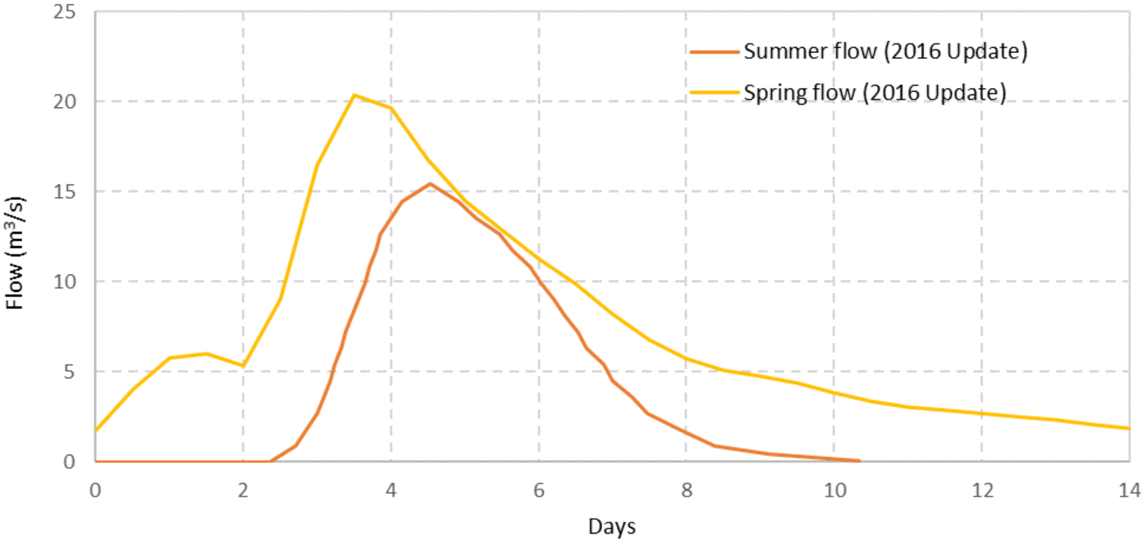


Figure 2-4: Summer and Spring Hydrograph Comparison

3.0 HYDRAULIC ANALYSIS AND DESIGN

3.1 BACKWATER CURVE

The 2005 DSA performed a backwater analysis (presented on page 4-13 of the 2005 DSA Report). The backwater curve on the downstream side of the dam was taken from the 2005 DSA report. It is plotted in **Figure** 3-1 below.

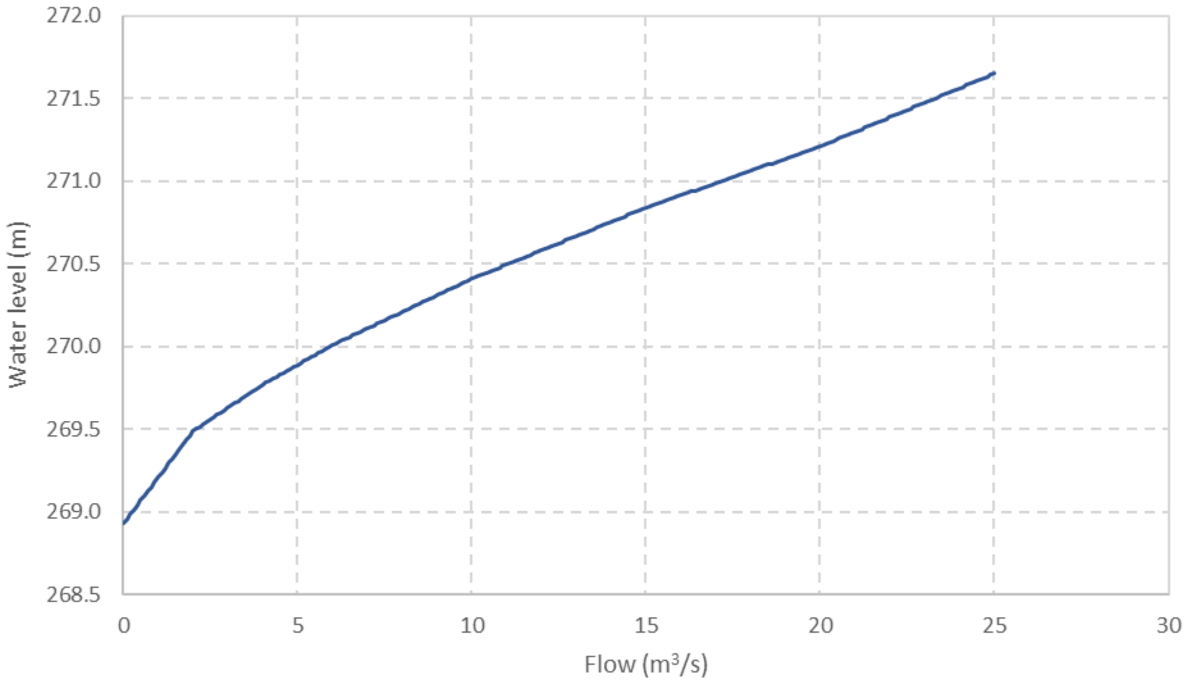


Figure 3-1: Backwater Curve Downstream the Dam

3.2 DISCHARGE RULE CURVE

The 2005 DSA used the rating curves that were previously developed in 1989. The curves are shown in the 2005 DSA, but the hypothesis for these discharge curves are not explained (the 1989 study is not available). It was chosen to develop new rating curves for theoretical formulas. The following paragraphs explain the development of these curves.

Flow $$Q = C_{sl} * L * H^{3/2} * (2g)^{1/2}$$

Where C_{sl} is the discharge coefficient (see below), *L* is the effective length (see below), *H* is the water head, and *g* is the gravitational constant.

Effective Length $$L = Ln - 2\ Ka * H$$

Where Ln is the real length of the spillway (1.93 m), Ka is the abutments contraction coefficients (0,1).

Discharge coefficient

When no stoplogs are in place, the discharge coefficient is constant with a value of 0.3502. As explained in the *Handbook of Hydraulics* (Brater et al.,1996), when stoplogs are in place, the discharge coefficient for a broad-crested weir (breadth of 0,3 m) varies from 0,3386 (when the upstream water level is at the stoplogs elevation) to 0.4131 (when the water elevation is 1,7 m or more over the stoplogs elevation).

Downstream Influence

The apron effect relationship described in the Design of Small Dams (USBR, 1987) is used to reduce the discharge coefficient when a downstream influence is observed. The backwater curve shown in Section 3.1 is used to identify the downstream water level for each discharge. The reduction factor of the discharge coefficient varies from 1 to 0.77 depending on the downstream water level.

Figure 3-2 shows the discharge curves for all the possible cases (0 to 8 stoplogs).

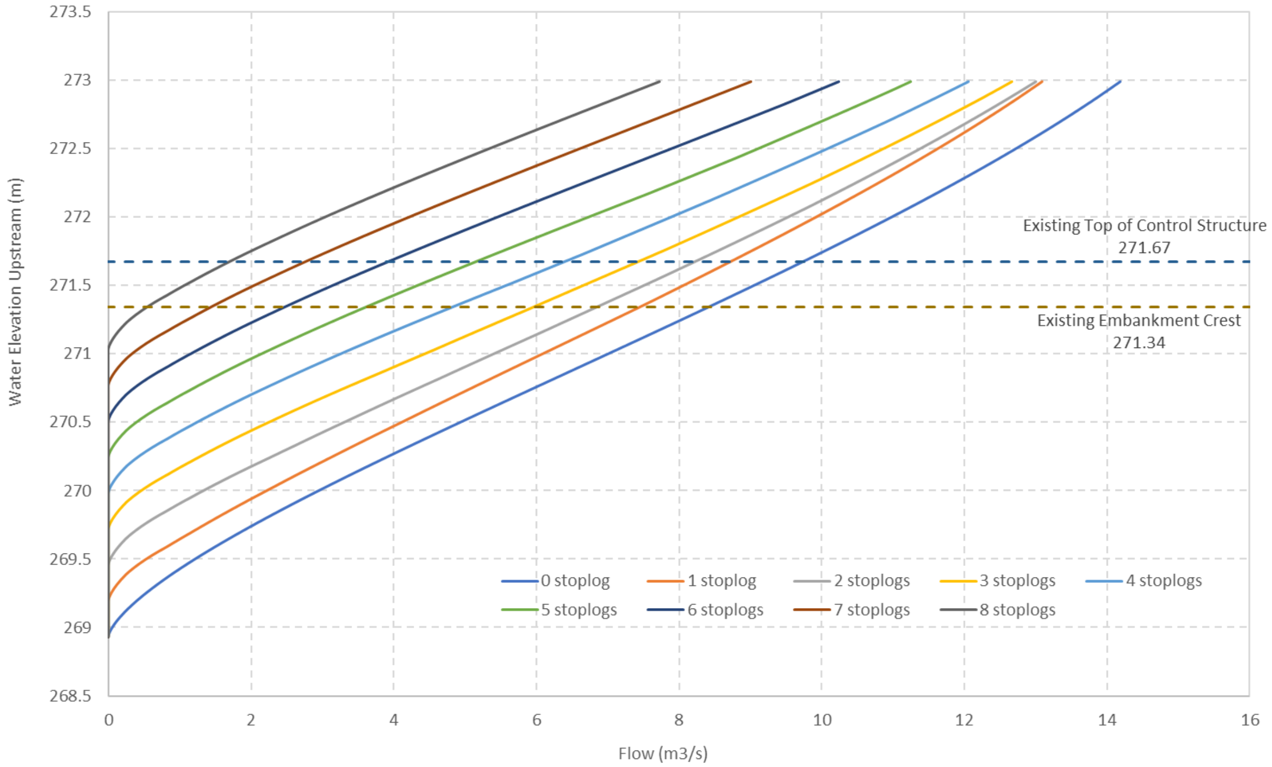


Figure 3-2: Discharge Rule Curve

3.3 FLOOD ROUTING

The flood routing was performed in Microsoft Excel. A time step of 15 minutes was chosen for computation. A starting water level and number of stoplogs were imposed at the beginning of the computation. For each time step, an inflow (from the flow hydrograph in Section 2.6) is available. For this time step, the outflow discharge was evaluated from the water level using the discharge curves, depending on the number of stoplogs (Section 3.2). The difference between the inflow and outflow allows to compute for a variation of volume during the time step. From this volume, the variation of water elevation can be computed, considering a constant lake area of 2.7*10^6 m^2. Then, for this new water level, the stoplog operating rules are verified and the number of stoplogs is modified according to the rules (the rules are not clearly defined, and discussion is presented further in this report). For the next time step, the loop was repeated using the new water level and number of stoplogs.

Note that a bascule gate has been included in the design, so its operation will be similar to stoplogs. The bascule gate discharges water by overflow above the gate crest, which is similar to overflow over stoplogs. The opening of the bascule gate by incremental step is the same as removing a stoplog.

3.3.1 Summer Operation

3.3.1.1 Assumptions

The peak summer inflow is 15.45 m^3/s (Section 2.4), and the summer hydrograph shape is presented in Section 2.6.1. In summer, the OMS states that target level in the lake is 271.00 m and is set as the initial water level. At the beginning of the event, all stoplogs are in place.

3.3.1.2 Stoplog Operating Rule Sensitivity Analysis

The OMS manual states that one stoplog is removed if water level goes over 271.1 m. In case of severe rain, all six stoplogs can be removed at a rate to prevent overtopping. The removal rate of the stoplogs influence the water level, and a sensitivity analysis of the removal rate is performed in this study.

First, the worst possible scenario was performed, which consists of leaving all six (6) stoplogs in place during the passage of the IDF. The water elevation in the reservoir attains 272.17 m. The results of this modeling are presented in **Table** 3-1. This scenario is not considered for the design as stoplogs are removed.

Second, the best management scenario was performed. The first stoplogs is removed at elevation 271,1 m (OMS requirement). The maximum operating water level is 271.28 m; however, in case of a major flood event (IDF), the water level will rise higher. The best management practice would be to remove the stoplogs at a rate where all six stoplogs would be removed when elevation 271.28 m is attained. This corresponds to a rate of removing one stoplog when an increment of water is 2.5 cm. The results of this modeling are presented in **Table** 3-2.

Third, intermediate scenarios were performed, using slower removal rates, and are presented in the upper part of **Figure** 3-3. The first stoplog is always removed at elevation 271,1 m (OMS requirement).

Note: With a rate of removing one stoplog when an increment of water is 10 cm, only seven stoplogs would be removed because the maximum water level in the reservoir (271.72 m) is lower than the last stoplog removal elevation (271.9 m). Therefore, for an increment higher than 10 cm, not all stoplogs are removed and this is shown on the lower part of **Figure** 3-3.

Table 3-1: Results of the Summer IDF Worst Management Scenario (no stoplog removed)

Initial Reservoir Elevation (m)	271.00
Maximum Reservoir Elevation during IDF (m)	272.17
Time of Max. Reservoir Elevation (days)	7.23
Initial Number of Stoplogs	8
Minimum Number of stoplogs	8
Maximum Water Elevation Downstream (m)	269.73
Peak Inflow (m^3/s)	15.45
Time of Peak Inflow (days)	4.53
Peak Outflow (m^3/s)	3.75
Time of peak outflow (days)	6.85

Table 3-2: Results of the Summer IDF Best Management Scenario (removal rate of 0.025 m)

Initial Reservoir Elevation (m)	271.00
Maximum Reservoir Elevation during IDF (m)	271.56
Time of Max. Reservoir Elevation (days)	6.17
Initial Number of Stoplogs	8
Minimum Number of stoplogs	0
Maximum Water Elevation Downstream (m)	270.33
Peak Inflow (m^3/s)	15.45
Time of Peak Inflow (days)	4.53
Peak Outflow (m^3/s)	9.25
Time of peak outflow (days)	5.93

3.3.1.3 Discussion of Summer Operation Results

This sensitivity analysis shows that the water elevation with the best management scenario is 271.56 m, and the water elevation with worst management scenario is 272.17 m. Therefore, in case of IDF, the water elevation will vary between these limits depending on the real stoplog operating. The worst management scenario is not considered for the design of the dam, and elevation 271.56 m is considered for the summer IDF.

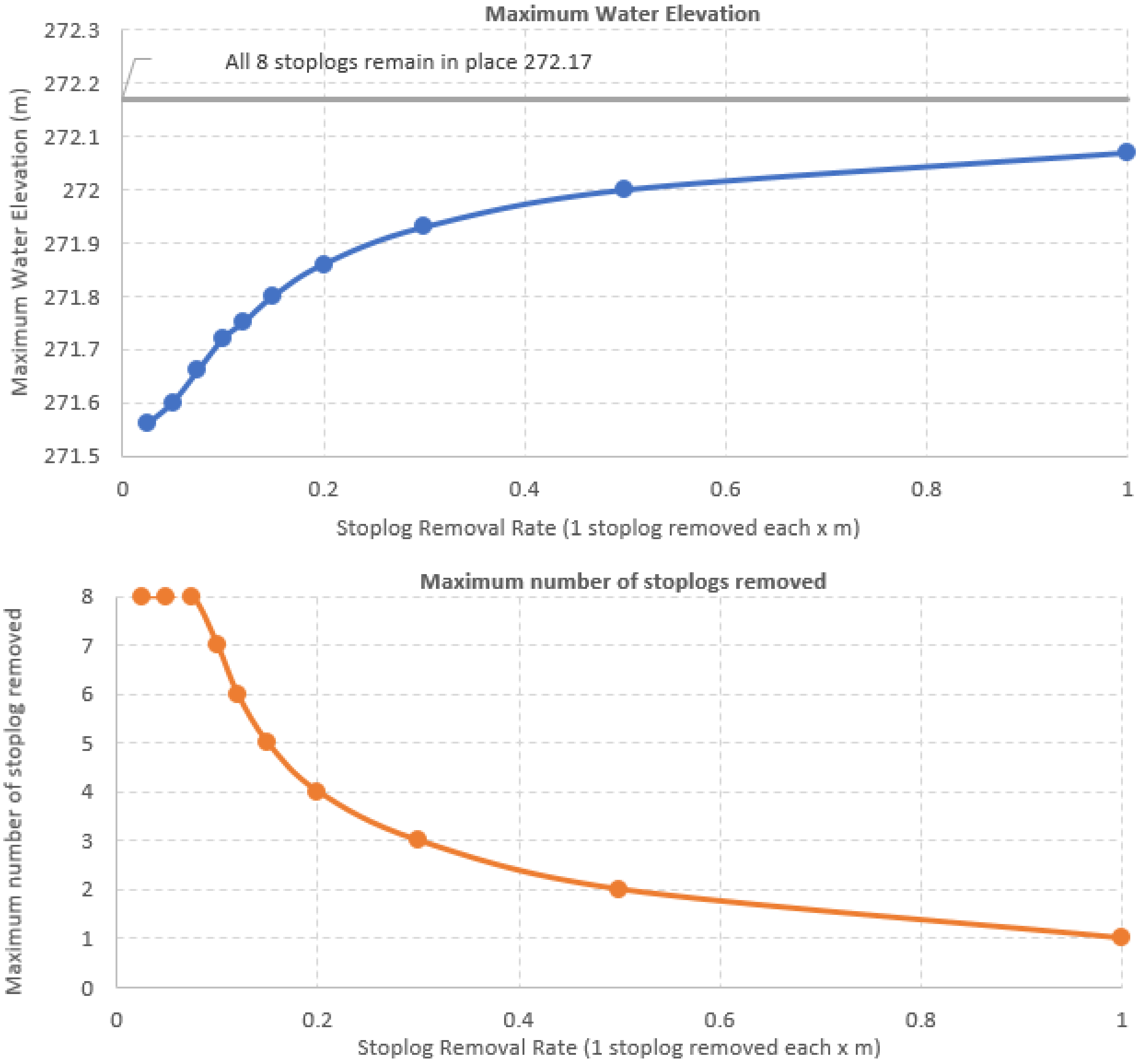


Figure 3-3: Stoplog Removal Rate Comparison

3.3.2 Spring Operation

The peak spring inflow is 20.40 m^3/s (Section 2.4), and the spring hydrograph shape is presented in Section 2.6.2. In the beginning of spring, two stoplogs are in place, as described in the OMS manual. The two stoplogs remain in place during the event. In winter, the regulated winter water level is between 269.28 m and 269.50 m. Therefore, the initial water level is considered to be 269.5 m.

The results of this simulation are summarized in **Table** 3-3 below.

Table 3-3: Results of Spring IDF (2 stoplogs in place)

Initial Reservoir Elevation (m)	269.50
Maximum Reservoir Elevation during IDF (m)	271.35
Time of Max. Reservoir Elevation (days)	7.47
Initial Number of Stoplogs	2
Minimum Number of stoplogs	2
Maximum Water Elevation Downstream (m)	270.09
Peak Inflow (m^3/s)	20.40
Time of Peak Inflow (days)	3.50
Peak Outflow (m^3/s)	6.88
Time of peak outflow (days)	7.19

3.3.3 Summary of Flood Routing Results

The purpose of this flood routing is to establish the water level in the lake during the passage of the IDF. The maximum operating water level is 271.28 m; however, in the case of a major flood event (IDF), the water level will rise higher to these elevations:

- Summer IDF (best management scenario): 271.56 m
- Spring IDF (2 stoplogs in place):271.35 m

3.3.4 Comparison with the 2005 DSA

The following lists the differences between the 2005 DSA and the 2016 update, and it explains the differences between the analyses:

- The 2016 update includes the data 2003-2016 that was not available in 2005.
- The flow station used for the 2016 update was the Mississippi River Station, which includes Shabomeka Lake in its watershed. The 2005 DSA used another station located on the Clyde River.

- The discharge rating curves are updated to take into account the variation of the discharge coefficient depending on the upstream water level. Also, the downstream submergence of the stoplog sill is considered, and reduce the discharge capacity of the spillway.
- A climate change increase for peak flow is included in the 2016 update.
- The removal rate of the 2005 DSA is not explained. Stantec presents a sensitivity analysis of the removal rate in this study.

Table 3-4: Comparison of Flood Routing for the 1/100-year Return Period

Season	Flow/Elevation Description	2005 DSA	2016 update (best management scenario)	2016 update (worst management scenario)
Spring (Annual)	Peak Inflow (m^3/s)	15.8	20.4	-
	Peak Outflow (m^3/s)	5.17	6.88	-
	Peak Reservoir Elevation (m)	271.01	271.35	-
Summer (seasonal)	Peak Inflow (m^3/s)	8.55	15.45	15.45
	Peak Outflow (m^3/s)	7.81	9.25	3.75
	Peak Reservoir Elevation (m)	271.16	271.56	272.17

3.4 TAILWATER LEVELS

The water elevations downstream of the dam for the flood conditions will be as follows:

Table 3-5: Tailwater Level

	Maximum Reached Elevation (m)	Initial Water Level (m)
100-year Summer Flood	270.49	269.59
100-year Spring Flood	270.68	269.64

4.0 FREEBOARD AND CREST HEIGHTENING DESIGN

The freeboard is the minimum vertical distance between the still pool reservoir level and the crest of the dam. This safety margin is maintained to restrict overtopping of the earth embankments by large waves, including consideration of wind and wave setup, and wave runup.

In 2005, a wave height and minimum freeboard analysis was performed. In 2016, a wave height and minimum freeboard memo was performed by Gemtec; the result of this memo was a freeboard requirement of 1.21 m, which seems to be a high value. To optimize the design, a more precise analysis is performed in this report. Both analyses showed the embankments would be overtopped.

The Canadian Dam Association (CDA) considers two freeboards: the normal freeboard, and the minimum freeboard.

- The normal freeboard is dictated by no overtopping by 95% of the waves caused by the most critical wind with a frequency of 1/1000 year when the reservoir is at maximum normal elevation.
- The minimum freeboard, for a low consequence dam, by no overtopping by 95% of the waves caused by the most critical wind with a frequency of 1/100 year when the reservoir is at maximum level during the passage of the IDF.

To compute the wind and wave setup, and wave runup, Stantec used the methodology explained by the Energy Society of James Bay (SEBJ,1997) presented hereafter.

4.1 WIND DATA ANALYSIS

The two previous wind analyses used the available wind speeds recorded in Kilaloe (#6104125) between 1953 and 1972. This station is located 78 km north of the dam. However, there is a wind station in Bancroft-Auto (#616I001) that has records from 1995 to 2015 (21 years), and is located closer to the dam, 60 km northwest of the dam. Therefore, the wind analysis from Bancroft-Auto was used for this analysis since it has more recent records, and it is closer to the dam.

Figure 4-1: Wind Recording Stations

The Bancroft-Auto station recorded wind data from 1995 to 2015 (21 years), with 1 measure per hour (sustained wind on the hour of observation). A frequency analysis was performed on the maximum annual wind in each of the four cardinal directions. The analysis was also performed using the maximum summer wind (May 15th to October 15th). Four statistical distribution laws were compared (Gumbel, Normal, Log-Normal, Log-Pearson 3). It was chosen to use the same law for all directions, and the Log-Pearson 3 was chosen to be the best fit. The computed wind results are presented in **Table** 4-1 and **Table** 4-2.

Table 4-1: Computed Wind Speed from Annual Winds (km/h)

Return period (year)	North	East	South	West
2	28.82	21.98	28.02	29.99
10	39.22	30.34	37.27	39.99
100	56.36	43.13	46.06	52.98
1000	79.06	59.07	53.17	66.95

Table 4-2: Computed Wind Speed from Summer Winds (km/h)

Return period (year)	North	East	South	West
2	23.88	18.58	23.12	23.26
10	33.85	21.91	30.09	30.93
100	51.41	25.19	37.82	40.62
1000	76.27	27.97	45.07	50.81

4.2 WAVES CALCULATION

A focal point on the dam was chosen and the direct fetch was drawn on the uninterrupted water surface. The water surface was extracted from Google Earth imagery. The length and angle of each fetch were used for the wave height computation. The "*Guide Pratique de Dimensionnement du RipRap*" of SEBJ, 1997 is used. The following **Figure** 4-2 shows the fetch of the focal point.

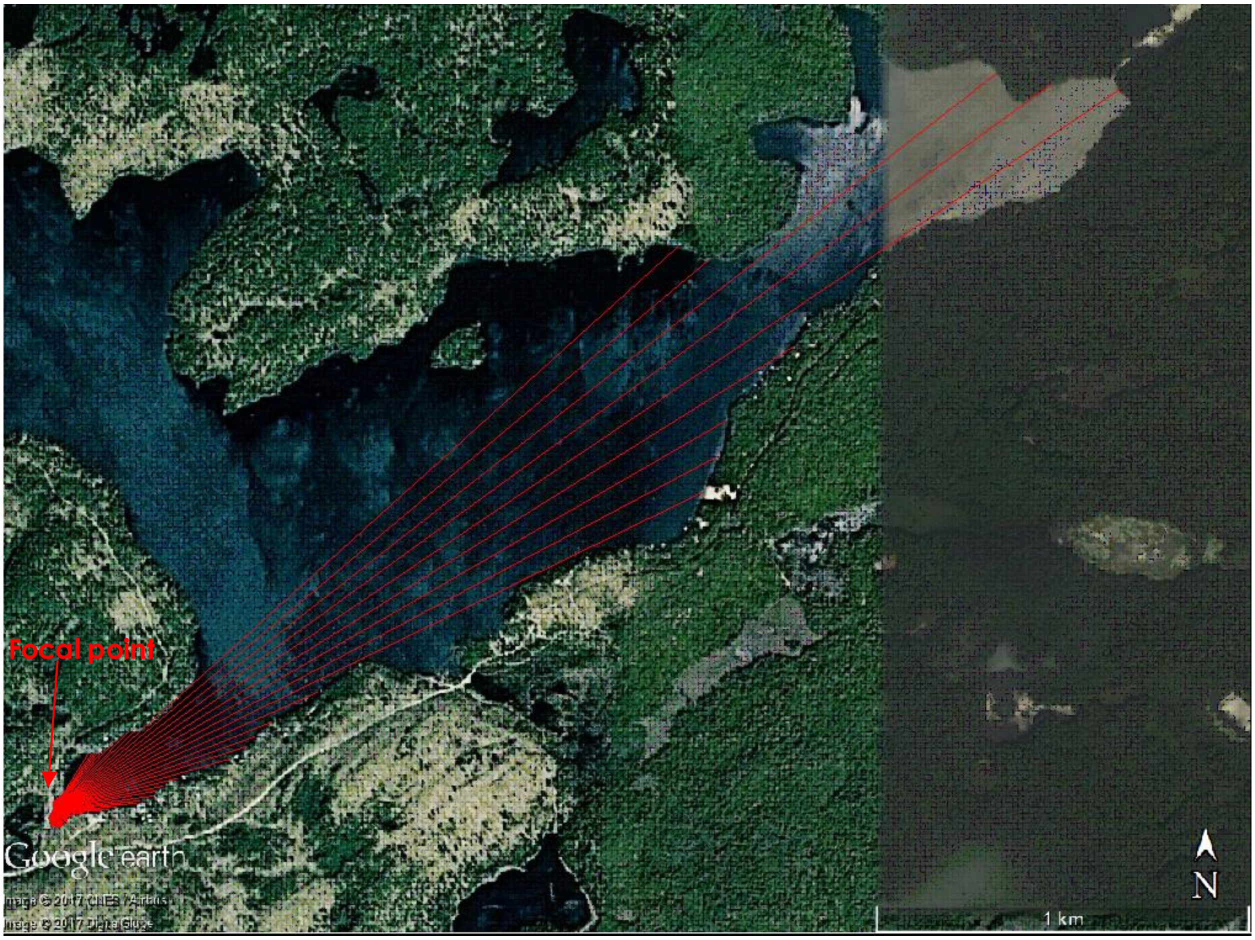


Figure 4-2: Radials and Fetch Determination

4.3 FREEBOARD

The average depth was evaluated in the area where the waves develop (it excluded the north branch of the lake). From the bathymetric maps, the average reservoir depth is 15.9 m. The wind blowing continuously with the same direction on the water body (average depth of 15.9 m) causes a rise of the water level (Wind Setup) while the slopes of the embankment (2H:1V) causes some waves to rise (Wave Runup). These two elevations are combined with the wave height which corresponds to the average of the 5% of the highest waves (95% probability) to form the maximum rise in water.

Table 4-3: Required Freeboard from Annual Winds

Return period (years)	Height of 95% of waves (m)	Required Freeboard (m)
2	0.13	0.23
10	0.19	0.34
100	0.29	0.52
1 000	0.43	0.77

Table 4-4: Required Freeboard from Summer Winds

Return period (years)	Height of 95% of waves (m)	Required Freeboard (m)
2	0.10	0.19
10	0.14	0.26
100	0.19	0.30
1 000	0.30	0.53

The dam is classified as "Significant" according to CDA. The normal freeboard corresponds to no overtopping by 95% of the waves cause by the 1/1000 year when reservoir is at maximum normal elevation. The minimum freeboard, for a "Significant" classification dam, correspond to no overtopping by 95% of the waves cause by the 1/10 year when reservoir is at IDF elevation. A significant classification dam has an IDF from 1/100 year to 1/1000 year. Since the 1/100 year recurrence (lowest of the range) was chosen, Stantec chose to maintain a safe approach and to match it with the waves of 1/100 year. It should be noted that a different management of the water level is observed in summer and winter; therefore, both seasons are computed and presented in **Table** 4-5 below. In the existing conditions, the embankment crest is at elevation **271.34 m;** as a result, overtopping would occur.

The following **Table** 4-5 shows the required freeboards for the dam.

Table 4-5: Required Elevation

	Water level in lake (m)	Required freeboard (m)	Crest Required elevation (m)
Normal Freeboard	271.1 (max. summer target)	0.77 (1000 year) (annual winds)	271.87
Minimum Freeboard			
• Spring/winter operation	271.35 (spring IDF level)	0.52 (100 year) (annual winds)	271.87
• Summer operation	271.56 (summer IDF level)	0.30 (100 year) (summer winds)	271.86

4.4 UPSTREAM RIPRAP PROTECTION

The upstream slope of the embankment will remain in its existing condition with a maximum slope of 2H:1V. The protection of the upstream face of the embankment will correspond to the 100-year return period, which is 140-230 mm (D_{50} = 185 mm) with a thickness of 370 mm.

Note that the 1000-year return period protection corresponds to 200-350 mm (D_{50} = 275 mm) with 550 mm thickness, but this return period is not required as dam classification is "Significant".

4.5 STILLING BASIN

4.5.1 HEC-RAS Model

The energy dissipation will be done using a stilling basin with some baffles. The gate and the stilling basin have been included in HEC-RAS model to determine the required information for energy dissipator design, i.e. the water depth (d_1) and Froude number (Fr) of the proposed spillway. By imposing the downstream water level of the backwater curve, the analysis revealed the hydraulic jump is relatively small, and the Froude number is too low for design of a stilling basin. A more stringent scenario was chosen comprising of a lower downstream water level imposed by a normal depth. The normal depth varies between 0,01 and 0,0001, and there were no changes before the hydraulic jump where the values are taken for design. The following **Figure** 4-3 shows the result of the HEC RAS model.

Two locations were assessed for the design of the stilling basin. The first location was downstream from the gate, where the d_1 is greatest. The d_1 and Froude number at this location are 0.5 m and 2.25, respectively. The second location was where the Froude number is the highest, which occurs right after the downstream concrete slab where the full river width is observed. The d_1 and Froude were 0.1 m and 5.05, respectively.

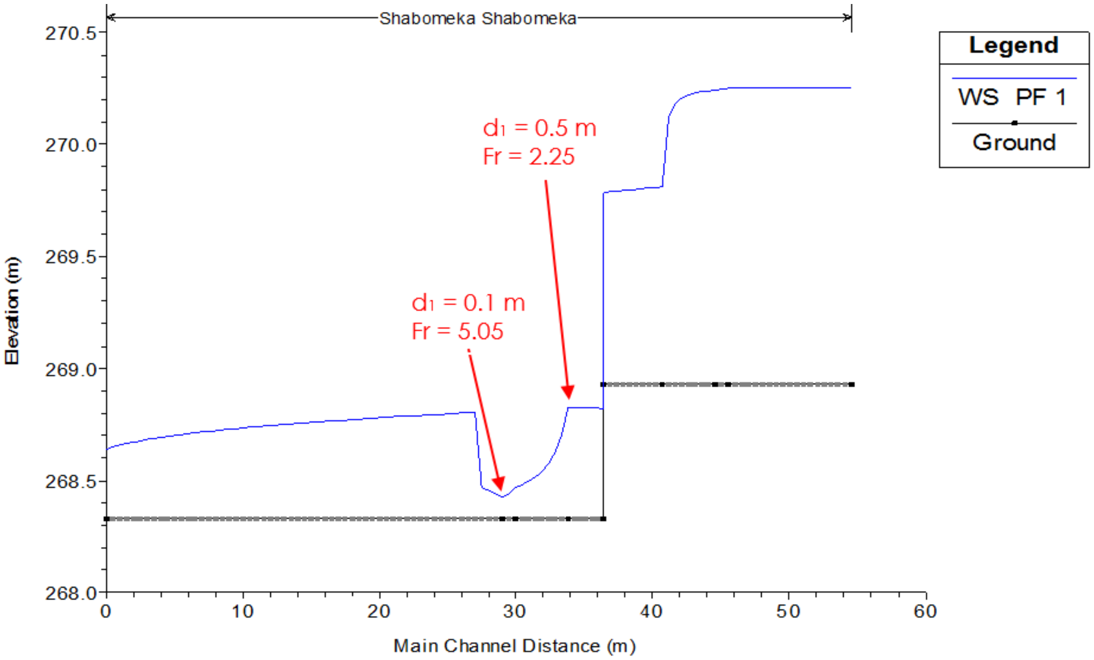


Figure 4-3: Profile View of HEC-RAS 1D

4.5.2 Types of stilling basins considered

Four different types of stilling basins were analysed including the USBR's Type I and Type IV, the *Design of Small Dams Alternative Low Froude Number* stilling basin and *USACE*'s stilling basin. The design controlling characteristic is the length of the stilling basin as the available length of the proposed structure downstream of the new gate location is 6.41 m. Therefore, to minimize cost and environmental impact, it was beneficial to consider a design of the stilling basin that could meet the desired conditions.

4.5.3 Calculations

The calculations were done using values of d_1 and Froude determined by HEC-RAS model. The theoretical tailwater depth, d_2, that is required for design of each basin was calculated using the following equation:

$$\frac{d_2}{d_1} = \frac{1}{2}(\sqrt{1-8F^2}-1)$$

The theoretical tailwater at the two considered location is 0.67 m (biggest Froude number) or 1.36 m (biggest d_1). It is required that the real tailwater depth be equal or bigger than the theoretical tailwater depth. The backwater curve presented in previous section of the report show that the tailwater depth would be 2.16 m (270.49 m (DS water level) – 268.33 (slab elevation)), which is bigger than the theoretical tailwater depth, so this criterion is compliant.

4.5.4 Results

The following table shows the design values of the four different stilling basins using the available charts in the reference documents. All of the results provided a basin length inferior to the maximum available length of 6.41 m. In all of the results, the scenario that used the largest Froude number demanded a shorter basin length than those that used the largest d_1. Therefore, the more conservative results to use for the stilling basin were those that used the $d_1 = 0.5$ m.

The results are reassuring because the USBR Type I basin (flat bottom with no baffle blocks or sill) do not exceed the maximum basin length. This means the proposed slab could rely only on the flat slab to dissipate the energy. Choosing another basin type will only give an additional safety margin.

The table below shows the results of each basin type with all of the dimensions taken from the charts.

Table 4-6: Potential Stilling Basin Dimensions

Method		**USBR Stilling Basin: Type I**		**Design of Small Dams: Type IV Basin**		**Design of Small Dams: Alternate Low Froude Number Basin**		**USCE**	
Scenario		**Biggest Froude**	**Biggest d1**	**Biggest Froude**	**Biggest d1**	**Biggest Froude**	**Biggest d1**	**Biggest Froude**	**Biggest d1**
Given data (HEC-RAS)	d_1	0.10	0.50	0.10	0.50	0.10	0.50	0.10	0.50
	Fr	5.05	2.25	5.05	2.25	5.05	2.25	5.05	2.25
Calculated	d_2	0.67	1.36	0.67	1.36	0.67	1.36	0.67	1.36
Results from charts	L	4.00	6.26	N/A	6.29	0.87	2.21	2.66	5.44
	L_1	-	-	-	-	0.60	1.90	1.29	2.61
	x	-	-	-	-	0.50	-	1.03	2.04
	h_3	-	-	-	-	0.02	0.10	-	-
	S	-	-	-	-	-	-	0.11	0.23
	S_e	-	-	-	-	-	-	0.05	0.11

4.5.5 Recommendations

The scenarios that used the largest d_1 were used to select the basin as a means of being more conservative. The *USACE* basin is the recommended solution as it uses baffle blocks, does not use chute blocks and has a basin length of 5.44 m, which is less than the maximum 6.41 m. The absence of chute blocks is beneficial as the proposed dam modifications might not necessarily allow for chute blocks as there is no rollway. The *USCE*'s basin would allow for the end sill to be located at 5.44 m and allow for another 0.97 m of concrete downstream before reconnecting to the natural ground.

4.6 EMERGENCY CANAL WITH FUSE PLUG

An additional emergency canal is proposed in the embankment. The hydraulic analysis shows the proposed spillway will ensure the passage of the IDF. The client asked to include an emergency canal to have additional discharge in case of larger flood events or delay in operation of the stoplogs/gate. The invert of emergency canal consists of a non-erodible stone protected canal with a fuse plug. The canal invert will be set 7 cm above the maximum operating level (271.35 m) and the fuse plug crest will be set at the IDF water level (271.56 m). If water level is above 271.56 m, the fuse plug will be eroded, and the canal will start discharging flow. We remind that the crest of the embankment will be at elevation 271.87 m. An HEC-RAS model was prepared to estimate the discharge of this canal, and with an energy level of 271.87 m in the lake, the canal can discharge an additional flow of 3 m^3/s without exceeding the IDF water level.

The velocities in this canal were modeled at 3.6 m/s, so a D50 of 450mm is required to protect against erosion.

5.0 DAM BREACH ANALYSIS AND HPC CLASSIFICATION

5.1 DAM BREACH MODELING

This dam classification showed that the dam is "Significant" with an Annual Exceedance Probability (AEP) of 1/100. Therefore, the other scenarios are not required to be computed (1000 year, 1/3 between 1000 year and PMF, 2/3 between 1000 year and PMF, PMF).

5.1.1 Dam Breach Parameters

Bottom of breach: The bottom elevation of the approach channel (between the gabion walls) is at elevation 268.86 m. This elevation is supposed for the bottom of the breach.

Water level:

Sunny Day Water Level: The maximum summer target range (OMS) is 271.1 m.

Summer 100-year Water Level: To obtain the most stringent scenario, it was considered that eight stoplogs remain in place. It does not correspond to the existing operation, but it consists of the highest possible water level that could be attained in the lake. The maximum water level is 272.17 m, as presented in Section 3.3.3.

Spring 100-year Water Level: As presented in Section 3.3.2, the maximum water level attained in the reservoir is 271.35 m.

Width of breach and development time

Concrete Structure: For concrete structures, the entire width is breached (4 m). Usually, vertical slopes are considered. However, since the structure is directly abutted on the embankment, if the concrete structure collapses, the embankment on the side of the structure will not remain vertical; hence, a slope of 1H:1V is considered.

Embankment: For erodible structures, the base width of the breach is equal to four times its height (12.84 m). The lateral slopes are 1H:1V.

Flow

Sunny Day Flow: The average yearly flow was computed on the Mississippi River Station and was transposed to Shabomeka Lake using the regression presented in Section 2. The average flow is 0.69 m^3/s.

IDF Flows: The summer hydrograph presented in Section 2.6.1 is used (peak flow of 15.45 m^3/s). The spring hydrograph presented in Section 2.6.2 is used (peak flow of 20.40 m^3/s)

Breach trigger time

Sunny Day: A constant inflow of 0.69 m^3/s is the input; therefore, the trigger time can be anytime.

IDF Flow: To obtain the highest peak flow, the trigger is set when the water level is maximum in the reservoir. To obtain this time, a simulation is performed without failure. Note that no overtopping over the embankment is modeled, since the new dam will not allow overtopping.

Table 5-1: Summary of Dam Breach Parameters

		Q Total (m^3/s)	Water Level (m)	Water Height (m)	Width at Base (m)	Time of breach development (h)
Concrete Structure	**Sunny day (Average flow)**	0.51	271.1	2.24	4	0
	100 yr. - Summer	15.45	272.17	3.21	4	0
Embankment	**Sunny day (Average flow)**	0.51	271.1	2.24	12.84	0.5
	100-year, Summer	15.45	272.17	3.21	12.84	0.5
	100-year, Spring	20.40	271.35	2.44	12.84	0.5

5.1.2 Dam Breach Discharge

The dam breach was modeled in HEC-HMS (version 4.2), developed and distributed by the US Army Corps of Engineers. The dam breach parameters have been included in the model. The outflow curves from the dam breach are presented on **Figure** 5-1 and **Figure** 5-2; and in **Table** 5-2. It shows that the embankment breach leads to higher flow than concrete structure and only the embankment breach scenarios will be modeled in this analysis.

Table 5-2: Dam Breach HEC-HMS Results

	Total Peak Discharge (m^3/s)
Sunny day condition - Embankment	82.51
Under 100 yr. summer condition - Embankment	157.33
Under 100 yr. spring condition - Embankment	103.56
Sunny day condition - Concrete	33.07
Under 100 yr. summer condition - Concrete	70.56

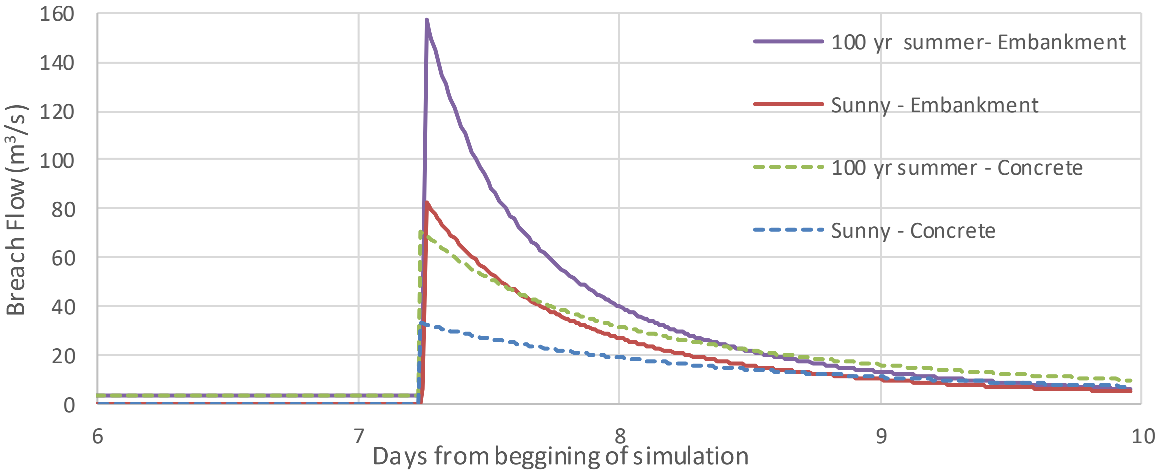


Figure 5-1: Summer Dam Breach Hydrographs

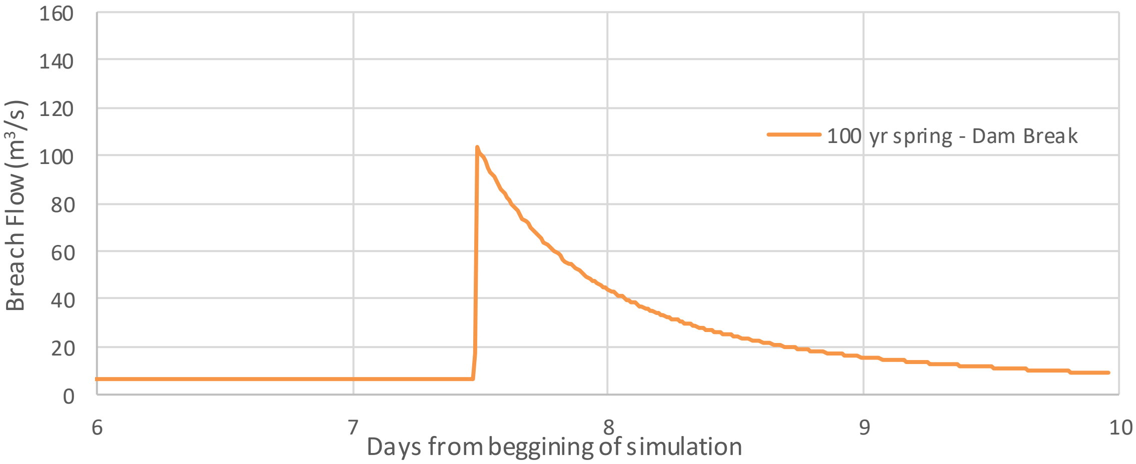


Figure 5-2: Winter Dam Breach Hydrograph

5.1.3 Hydraulic Model

The one-dimensional HEC-RAS model (version 5.0.3) developed and distributed by the US Army Corps of Engineers was used to compute the flood routing and prepare inundation mapping.

The DRAPE DSM dataset was obtained from the MNRF website. The HEC-GeoRAS application (version 10.1) was used to create the HEC-RAS geometry. The upstream extent was chosen as the dam. The downstream extends consist of the dam located at the exit of Lower Mazinaw Lake. Eighteen (18) cross sections have been generated. An average Manning coefficient of 0.035 has been considered for streambed and river banks.

The Upper and Lower Mazinaw Lakes were modeled as one storage area, considering they are linked together by a channel of 40m width. The area of the lakes was computed in ARC-GIS (16.06 km^2). Two (2) cross sections are located in the lake, where Semicircle creek enlarges to become the lake. Therefore, the area of the storage area (15.84 km^2) excludes the small area already included in the cross sections.

5.1.4 Boundary Condition

In this unsteady flow simulation, the upstream condition consists of the inflow of the river (with and without dam breach), computed in Section 5.1.2.

The downstream condition was set as the dam of the Lower Mazinaw Lake. The drawings of the dam were obtained, and the geometry of the spillway was put as the outflow of Lower Mazinaw Lake. Note that the operating level of Lower Mazinaw Lake (267.80 m) shown on the drawing was set as the water elevation in the lake. Also, the Lower Mazinaw Lake dam stoplog elevation was set as the top of the stoplogs (all in place) (267.80 m). No backwater effect at Lower Mazinaw Lake Dam was modeled, but the Lower Mazinaw Lake stoplogs cause a drop of 3.2 m behind the stoplogs. This height prevents the water elevation downstream Lower Mazinaw Lake to submerge the top of Lower Mazinaw Lake dam stoplogs, so no backwater effect should occur. Note that if stoplogs are removed, lower water level would be observed in Lower Mazinaw lake and would lead to lower consequences around the lake.

5.1.5 Mazinaw Lake Inflows

In case of IDF, other inflows come into Lower Mazinaw Lake from the other streams. The watershed of Lower Mazinaw Lake was computed with ARC-GIS. The area of the watershed (excluding Shabomeka Dam watershed) is 297.99 km^2. The Mississippi River Station was used to compute the inflow in the lake. The computed inflow of Shabomeka watershed were subtracted from the Mississippi River Station flows. Then, the transposition method was used to transpose the Mississippi River Station flows without Shabomeka (318.69 km^2) to Lower Mazinaw watershed without Shabomeka (297.99 km^2). Considering climate change effect, the summer peak inflow is 36.53 m^3/s, and the spring inflow is 40.64 m^3/s. The same hydrographs' shape as presented in Section 2.6 were used as inflow in Lower Mazinaw Lake.

5.2 SIMULATIONS RESULTS AND INUNDATION MAPPING

5.2.1 Dam Break Simulations Results

The sunny day failure, 100-year summer and 100-year spring scenarios were performed. The results of these simulations are presented in **Table** 5-3 through **Table** 5-5.

Table 5-3: Sunny Day HEC-RAS Results

			Sunny day					
	HEC-RAS Cross Section	**Distance from Dam (m)**	**Peak Flow (m^3/s)**	**Flood Wave arrival Time (hour)**	**Peak Flow arrival Time (hour)**	**Water Elevation without Dam Break (m)**	**Water Elevation with Dam Break (m)**	**Incre-mental Water Level (m)**
Dam Location	4963	0	67.51	0:00	0:00	269.62	271.38	**1.76**
Semicircle Lake outlet	3585	1378	67.51	0:22	2:41	269.62	271.36	**1.74**
Between Semicircle and Lower Mazinaw Lakes	3192	1771	67.51	0:26	2:44	267.76	269.91	**2.15**
Lower Mazinaw Lake	Storage area	4963	66.65	1:08	3:22	267.8	268.05	**0.25**

Table 5-4: 100-Year Summer HEC-RAS Results

			100-year summer					
	HEC-RAS Cross Section	**Distance from Dam (m)**	**Peak Flow (m^3/s)**	**Flood Wave arrival Time (hour)**	**Peak Flow arrival Time (hour)**	**Water Elevation without Dam Break (m)**	**Water Elevation with Dam Break (m)**	**Incre-mental Water Level (m)**
Dam Location	4963	0	156.2	0:00	0:30	270.68	271.99	**1.31**
Semicircle Lake outlet	3585	1378	132.3	0:05	2:07	270.67	271.96	**1.29**
Between Semicircle and Lower Mazinaw Lakes	3192	1771	132.26	0:11	2:10	268.83	270.23	**1.40**
Lower Mazinaw Lake	Storage area	4963	130.15	0:17	2:15	268.38	268.75	**0.37**

Table 5-5: 100-Year Spring HEC-RAS Results

			100-year spring					
	HEC-RAS Cross Section	Distance from Dam (m)	Peak Flow (m^3/s)	Flood Wave arrival Time (hour)	Peak Flow arrival Time (hour)	Water Elevation without Dam Break (m)	Water Elevation with Dam Break (m)	Incremental Water Level (m)
Dam Location	4963	0	103.6	0:00	0:30	270.68	271.68	**1.00**
Semicircle Lake outlet	3585	1378	88.16	0:05	2:13	270.67	271.66	**0.99**
Between Semicircle and Lower Mazinaw Lakes	3192	1771	88.15	0:07	2:30	268.83	269.84	**1.01**
Lower Mazinaw Lake	Storage area	4963	86.7	0:15	2:35	268.74	268.99	**0.25**

5.2.2 Dam Break Inundation Mapping

The inundation maps are shown in **Appendix C.** Inundation mapping has been completed using ArcGIS. As shown in the previous Tables (5-3 through 5-5), the 100-year summer scenario has slightly higher water levels than the 100-year spring scenario. To simplify the map, only sunny day and 100-year summer scenarios are drawn. The 100-year spring flood scenario is not drawn since it has a lower water level than the 100-year summer flood and has almost a comparatively similar flooding area extension (considering the drawing scale) as the 100-year summer scenario.

5.3 CONSEQUENCE IDENTIFICATION AND DAM CLASSIFICATION

The dam classification is performed according to the technical bulletin "Classification and Inflow Design Flood Criteria" published in 2011 by the Ontario MNRF. It is also performed following CDA Guidelines.

5.3.1 Consequence Classification

The last column of **Table** 5-6 and **Table** 5-7 present the incremental water levels caused by the dam breach assessment. The dam classification is required to consider the incremental consequences, dictated by the incremental change in water level.

5.3.1.1 Population at risk

Only temporary population is at risk. The description of the population at risk is presented in next section "Loss of Life", but it is mainly caused at Snyder Bay Lane. The dam classification considering this criterion is "significant".

5.3.1.2 Loss of life

Shabomeka Lake Road, located in the first 300 m downstream the dam, it has an elevation of 271.1 m (from DRAPE dataset). Therefore, in case of dam break, there would be a depth of water over the road ranging between 28 cm (sunny day) to 89 cm (100-year flood). This road provides access to approximately 285 structures upstream on Shabomeka and Shawenegog Lakes. The flooding over the road (el. 271.1 m) would occur 1:37 after the dam break and rises slowly.

Snyder Bay Lane located 1.6 km downstream from the dam crosses Semicircle Creek. The geometry of this crossing is not known, but it could be flooded or washed away. This road is a private road according to the Transportation Maps (North Frontenac, 2017) of the Township of North Frontenac, and the Township does not perform any maintenance on this private road. It does not have a lot of circulation (not a thru way), and only Lower Mazinaw lakeside residents use it. Even if the Snyder Bay Lane flood remains for a long period, we didn't foresee any risk of life associated with this road inundation due to a dam break.

Upper and Lower Mazinaw Lakes will have an incremental water level, shown in the last column of **Table** 5-3 through **Table** 5-5. The incremental water level in Upper and Lower Mazinaw lakes ranges between 0.25 m and 0.38 m, and since it is a lake, there are no water velocities. The application of the 2x2 rule (MNRF, 2011) states the criterion for the assessing life safety. No velocities in the lake are considered; therefore, the only criterion is if water depth exceeds 0.8 m. The normal summer operating water level is 267.8 m and the maximum water level after dam break (100-year flood) is 268.99 m; therefore, a total height of 1.19 m is over normal operating level. It is supposed that houses are built at least 50 cm higher than normal operating level; therefore, a maximum water depth would be 0.69 m. Consequently, there is no risk of life loss in Upper and Lower Mazinaw Lakes due to flooding.

5.3.1.3 Environmental and Cultural Values

Release of water from the lake is unlikely to cause toxic effects, given that Shabomeka Lake is a fishing lake of good quality. The release of sediment may cause some short-term, localized environmental effects. Potential physical effects to spawning fishes present the greatest environmental risk resulting from a breach.

To our knowledge, no archaeological and cultural heritage concerns are present in the flooded zone. Potential impacts and mitigation measure for environmental and cultural values have been addressed in more detail in the Class EA study for the proposed works.

The dam classification does not consider the upstream Lake Trout spawning shoals that would be out of water.

5.3.1.4 Infrastructure and economics

Shabomeka Lake Road would be only flooded laterally by less than a meter, and no damages should occur at the road. Snyder Bay Lane crossing could be damaged by a dam break, but it is a private road with no maintenance done by the Township. The failure of the dam would affect the Shabomeka lakeside residents (recreational use). Due to the small extension of the dam, it could be reconstructed relatively rapidly, and the loss would only be temporary. CDA Guidelines indicates that the damage to the dam owner's property may be excluded from the estimate and left to the owner to consider separately. No pre-existing potential for development is located within the incremental area. Potential impacts and mitigation measures for infrastructure and economics have been addressed in more detail in the Class EA study for the proposed works.

The dam classification considering this criterion is "Significant".

5.3.2 Dam Classification

The previous criteria is summarized in the following **Table** 5-6. Shabomeka Dam is considered a "Significant" consequence dam, according to CDA Guidelines 2007 (revision 2013) , and a "Moderate" consequence dam, according to the MNRF Technical Bulletin.

Table 5-6: CDA Guidelines Dam Classification

Criteria	Consequence of failure	Classification according to CDA Guidelines 2007 (revision 2013)
Population at risk	Temporary only (users of Snyder Bay Lane crossing)	Significant
Loss of life	There is no possibility of loss of life other than through unforeseeable misadventure	Low
Environment and cultural	Short-term impacts	Low
Infrastructure and economics	Losses of infrequently used transportation routes (Private road: Snyder Bay Lane crossing) Temporary loss of recreational uses to Shabomeka lakeside residents	Significant
Overall classification		**Significant**

Table 5-7: MNRF Dam Classification

Criteria	**Consequence of failure**	**Ontario MNRF Technical Bulletin (2011)**
Life Safety	No potential loss of life	Low
Property Losses	Private road (Snyder Bay Lane crossing) Minimal damage to residential areas	Moderate
Environment losses	Minimal loss	Low
Cultural – Built Heritage Losses	None	Low
Overall classification		**Moderate**

5.3.3 Inflow Design Flood (IDF)

CDA Guidelines indicates that a “Significant” dam has an annual exceedance probability (AEP) between 1/100 year and 1/1000 year selected on the basis of incremental flood analysis, exposure, and consequences of failure. Note that the “Significant” classification is dictated only because of the potential damages to Snyder Bay Lane crossing. This crossing is a private road, and no maintenance is done by the Township. In case of losing of this crossing, the resident could cross the creek using all terrain vehicle (ATV) (after the dam break flood). The crossing can easily be rebuilt (short term consequence). The exposure to risk is low since water levels do not rise rapidly. Therefore, we consider that the 1/100-year IDF flow is the most appropriate to be used as Inflow Design Flood.

The MNRF Technical Bulletin indicates that a “moderate” dam has an annual exceedance probability (AEP) between 1/100 year and 1/1000 year. For the same reasons presented above, we consider that the 1/100-year IDF flow is the most appropriate to be used as Inflow Design Flood.

6.0 STRUCTURAL CONCRETE

6.1 EXISTING CONCRETE STRUCTURE

The existing structure was built as a temporary structure which appears to be constructed upon an older timber structure. There are numerous reports indicating that the current structure is to be removed and reconstructed (refer to Shabomeka Lake Dam – Mississippi River – Project Summary – February 1989).

As a result of the settlements (most likely due to failure of the underlying timber structure), we have provided a new structure to replace the existing. There is limited information on the timber structure.

6.2 DESIGN OF NEW STRUCTURE

The assessment of the global stability of the structure will be conducted in accordance with the Canadian Dam Safety (CDA) Guidelines 2007-R13. The following includes the loads and values used to assess the stability.

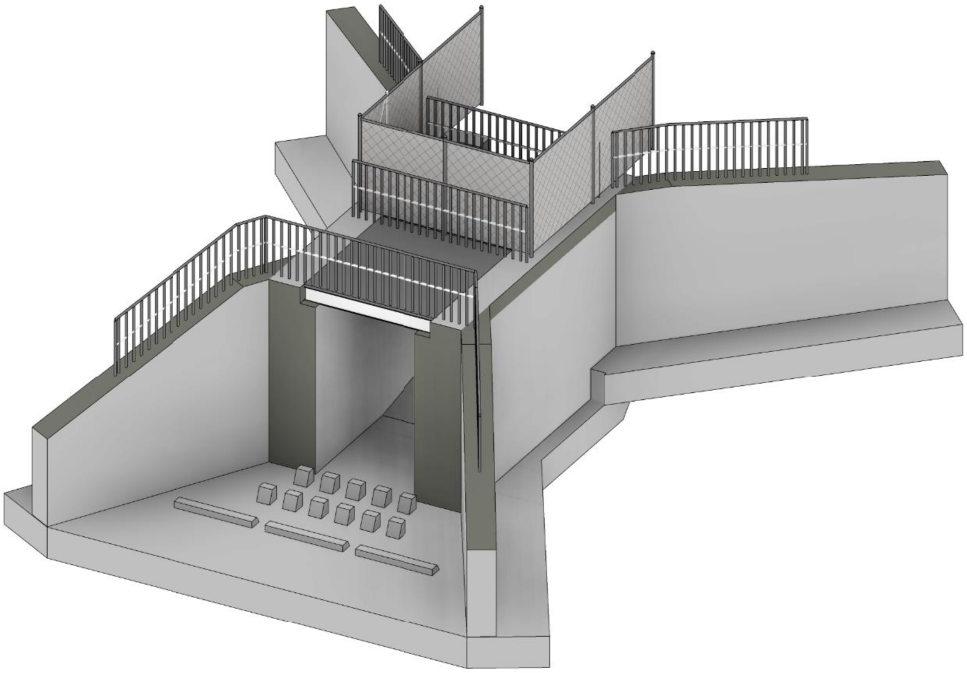


Figure 6-1: 3D View of the New Structure

6.2.1 Dead Loads

Dead load considered during the assessment are:

- Unit weight of Mass Concrete = 22.8 kN/m^3
- Unit weight of structural reinforced concrete = 24.0 kN/m^3
- Unit weight of water = 9.81 kN/m^3

Any weights associated with equipment (such as lifting machines, stop logs...), embedded items (such as angles, channels...) and operational platforms (such as catwalks/handrails) have also been neglected which is common in DSR.

6.2.2 Hydrostatic Loads

The hydrostatic loads considered in the analysis are based on values provided by the hydraulic analysis and are as follows:

Table 6-1: Design Conditions Water Levels

Load Case	**Headwater Level (HWL) (m)**	**Tailwater Level (TWL) (m)**
Summer (Full Supply Level)	271.10	269.60
Winter	269.60-269.80	268.93
Flood - IDF	271.56	270.25
Seismic	271.10	269.60

Uplift pressure applied to the under the concrete structure will be assumed to vary linearly from headwater pressure at the upstream side and reduced to tailwater pressure on the downstream side. If the tailwater is below the base, then the tailwater pressure will be assumed to be zero at the toe of the structure. If the base has cracked during the analysis, then the headwater pressure will be assumed to be full headwater pressure to the end of the crack under the structure. In the case of a verified working grout curtain, then the uplift will be reduced to 2/3 at the grout line and back to tailwater pressure at the downstream side of the structure. In the case where drains are provided, then the water pressure will be modified based on the following diagrams taken from the CDA Guidelines.

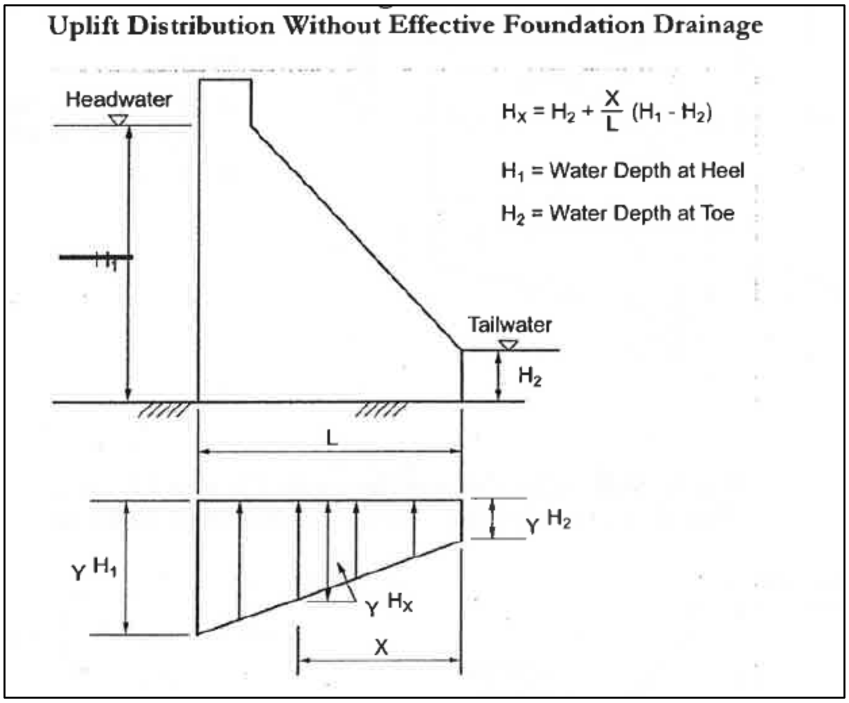


Figure 6-2: Uplift Distribution

6.2.3 Live Loads

Live loads associated with people or equipment have been neglected.

6.2.4 Ice Loads

The force of ice against the structure will be 73 KN/m at 300 mm below the water surface, which is common for structures of this size based on Ontario Ministry of Natural Resources loading data.

6.2.5 Earthquake (Seismic) Loads

The seismic forces used in the analysis have been provided from the Government of Canada 2015 National Building Code Seismic Hazard Calculation software. The peak ground acceleration of 0.101g (10.1%g) is based on a probability of (1: 2475yrs).

The analysis will be based on a pseudo-static seismic force analysis. The increase in hydraulic pressures are based on the Westergaard equations for loads and distribution.

The horizontal force required to accelerate the concrete mass is calculated as:

$Qh = \alpha_h * W$ where: Qh = horizontal seismic load (kN)

α_h = horizontal seismic coefficient = 1/2*PGA

PGA = peak ground acceleration= 0.101

Structural concrete

W = concrete weight, kN

The vertical force required to accelerate the concrete mass is calculated as:

$Qv = \alpha_v * W$ where: Qv = vertical seismic load (kN)

α_v = vertical seismic coefficient = $1/2 * \alpha_h$

W = concrete weight, kN

Since an earthquake produces oscillating forces, the horizontal PGA and vertical PGA cannot occur at the same time. To account for this in the stability calculations, three separate combinations of vertical and horizontal seismic combinations were considered, but only the maximum value will be reported. The three combinations of vertical and horizontal seismic load are as follows:

Table 6-2: Stability Analysis – Seismic Coefficient

Seismic Combination	Horizontal	Vertical
100% Horiz. (Full PGA) No Vertical	$\alpha_h=0.101$	-
100% Horiz. 30% Vertical	$\alpha_h=0.0505$	$\alpha_v=0.015$
30% Horiz. 100% Vertical	$\alpha_h=0.015$	$\alpha_v=0.025$

Table 6-3: Stress Analysis – Seismic Coefficient

Seismic Combination	Horizontal	Vertical
100% Horiz. No Vertical	$\alpha_h=0.101$	-
100% Horiz. 30% Vertical	$\alpha_h=0.101$	$0.3\ \alpha_v$
30% Horiz. 100% Vertical	$0.3\ \alpha_h$	$\alpha_v=0.07$

6.2.6 Drag Forces

Drag loads generated on piers are relatively small for the size of this type of structure and thus are neglected.

6.2.7 Soil Forces

Soil forces will be applied to the face of the upstream and downstream face if applicable. As site information has been provided, we have assumed a saturated density of 18 kN/m^3 and a dry unit density of 16kN/m^3.

6.2.8 Concrete-Rock Interface

The residual value considered in the analysis was 30 degrees and zero cohesion since a geotechnical investigation has not been provided. The stability results need to be validated with a geotechnical investigation.

6.2.9 Cases of Load Combinations

The load combinations considered in the review are described in **Table** 6-4 below:

Table 6-4: Load Scenarios

Case	**Load Combination**	**Description**
Summer (Regulated Water Level)	Usual	D+H+U
Winter	Usual	D+H+U+I
Flood IDF	Unusual	D+H(IDF)+U
Earthquake	Extreme	D+H+U + Q
Post- Earthquake*	Extreme	D+H+U*

*if post cracking occurs, it may increase uplift pressures

- D = Dead Load
- H = Hydraulic Loads
- U = Uplift associated with corresponding water pressures
- I = Ice Load

6.3 ACCEPTANCE CRITERIA

The acceptance criteria have been adopted from the CDA Guidelines. For further clarification refer to the CDA Dam Safety Guidelines 2013.

6.3.1 Sliding Safety Factor

The sliding safety factor will be considered acceptable if the following is satisfied. During this review cohesion is assumed to be zero unless otherwise validated.

Table 6-5: Acceptable Sliding Safety Factor

Loading Combination	Position of resultant force (percentage of base in compression)	Normal Compression stress [note 1]	Sliding safety factor		
			Friction Only	Friction and cohesion [note 2]	
				With tests	Without tests
Usual	Preferably within the kern (middle third of the base: 11% compression); however, for existing dams, it may be acceptable to allow a small percentage of the base to be under 0 compression if all other acceptance criteria are met [note 3]	<0.3 x f_c'	≥1.5	≥2.0	≥3.0
Unusual	75% of the base in compression, and all other acceptance criteria must be met	<0.5 x f_c'	≥1.3	≥1.5	≥2.0
Extreme flood	Within the base, and all other acceptance criteria must be met	<0.5 x f_c'	≥1.1	≥1.1	≥1.3
Earthquake	Within the base, except where an instantaneous occurrence of resultant outside the base may be acceptable	<0.9 x f_c'	[note 4]		
Post-earthquake	Within the base	<0.5 x f_c'	≥1.1 [note 5]	[note 6]	

Note 1. Where f_c' = compressive strength of concrete.

Note 2. Given the significant impact a very small amount of cohesion can have on shear resistance of small and medium-sized dams, the use of a cohesive bond in calculating the sliding safety factor should be done with extreme caution.

Note 3. It is very important to verify that all possible failure modes have been addressed under a potential cracked base scenario.

Note 4. The earthquake load case is used to establish post-earthquake condition of the dam.

Note 5. If post-earthquake analysis indicates a need for remedial action, this condition should not be allowed to remain for any length of time. Remedial action should be carried out as soon as possible such that factors of safety are increased to the level of the pre-earthquake conditions.

Note 6. Shear resistance based on friction and cohesion needs to be considered carefully, since the analysis surface may not remain in compression throughout the earthquake but may result in cracking, which will change the resistance parameters.

6.3.2 Resultant Location and Perpendicular Stresses

The location of the force resultant will be considered acceptable if it is found to be located as follows:

Structural concrete

Table 6-6: Acceptable Positions of Force Resultant

Load Combination	Position of the Force Resultant
Usual	Preferably within the kern (middle third of the base: 100% compression); however, for existing dams, it may be acceptable to allow a small percentage of the base to be under 0 compression if all other acceptance criteria are met.
Unusual	75% of the base in compression and all other acceptance criteria must be met.
Extreme	Within the base and all other acceptance criteria must be met.

Compressive stresses are considered acceptable as follows:

Table 6-7: Acceptable Compressive Stresses

Load Combination	Normal Compressive Stress
Usual	$<0.3 \times f_c'$
Unusual	$<0.5 \times f_c'$
Extreme – Flood	$<0.5 \times f_c'$
Extreme – Earthquake	$<0.9 \times f_c'$
Post-earthquake	$<0.5 \times f_c'$

6.4 STABILITY RESULTS

The assessment of the stability of the structure based on the previous section and in accordance with the Canadian Dam Safety (CDA) Guidelines 2007-R13 has the following results:

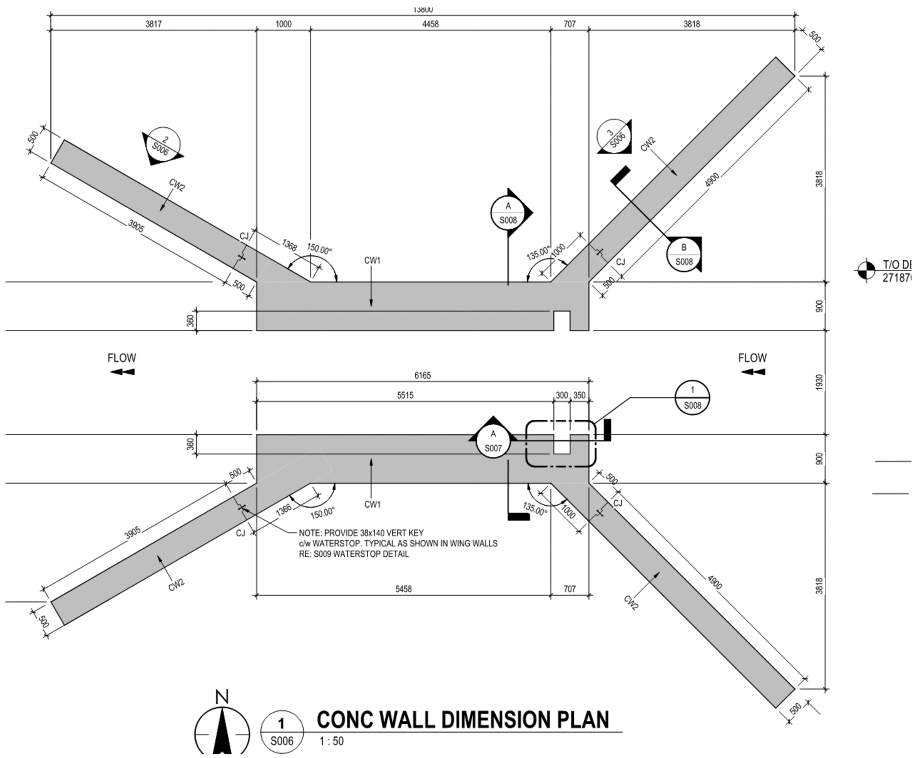


Figure 6-3: Plan View of the Structure Showing Base Slab SL2

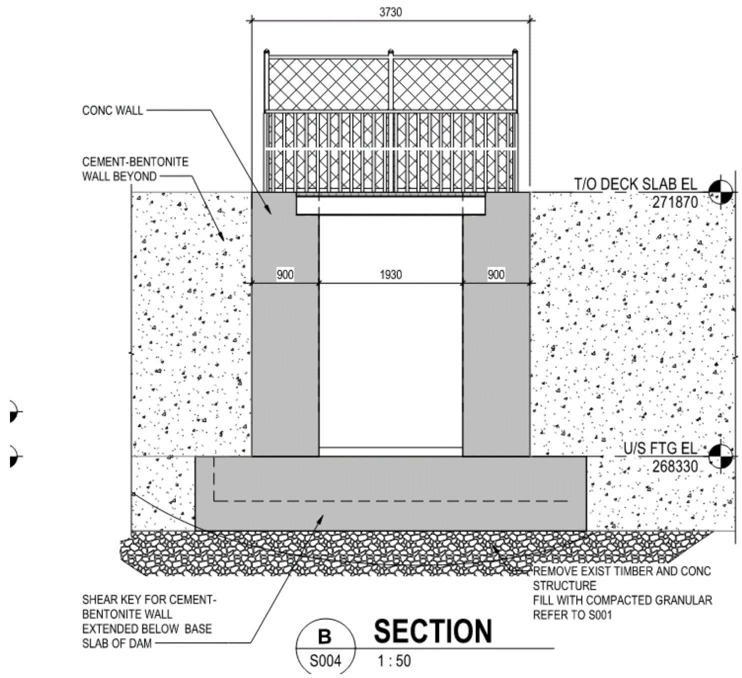


Figure 6-4: Section Through the Structure Showing Piers and Deck

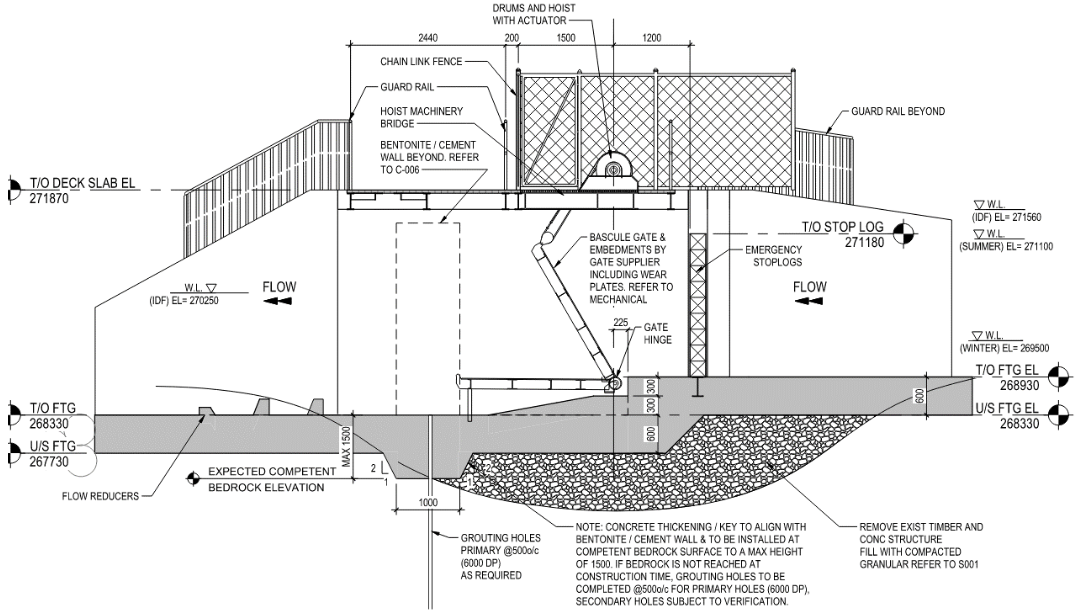


Figure 6-5: Section Through the Structure Showing Stoplogs

Table 6-8: Stability Results (Angle = 30 degrees, Cohesion = 0 kpa)

Case	Sliding Safety Factor (SSF)	Floatation Safety Factor (FSF)	Maximum Base stress (kpa)	Position of resultant	Length of base in compression	Comments
Summer (Regulated Water Level)	6.71	1.76	33	Kern	100%	
Winter	1.73	2.68	42	Kern	100%	I=75kN/m
Winter (Unusual)	1.47	2.68	57	Kern	100%	I=90kN/m
Winter (Extreme)	1.24	2.68	43	Kern	100%	I=108kN/m
Flood IDF (1:100)	7.68	1.57	27	Kern	100%	
Earthquake (no crack)	1.60	1.69	31	Kern	100%	100% Horiz. 30% Vertical

7.0 GEOTECHNICAL

7.1 EXISTING EMBANKMENT

The Shabomeka Lake dam consists of a single concrete control structure separating two earth embankments. As no information concerning these embankments zoning is available, they are considered homogeneous. The North and South embankments are 48.8 m and 12.2 m long, respectively. The height varies to a maximum of about 3 m with a crest width of approximately 4 m and a crest elevation of 271.34 m. The slope gradients vary from 2H:1V to 3H:1V downstream and are generally 2H:1V upstream.

Based on previous geotechnical investigation (**Appendix F**) carried out by Trow Associates Inc. (July 20, 2004), the subsurface conditions at the embankment location were observed to be generally fill overlying bedrock. The fill materials are composed of sand and gravel near the surface, changing to silt with some sand with depth. The standard penetration test N-values measured in the fill ranged between 2 and 41, indicating the soil is in a very loose to dense state. Note that very loose conditions were observed in the boreholes located one metre on either side of the concrete structure.

At the embankment location, bedrock was encountered directly below the fill at elevation ranging from 266.8 m to 269.2 m. Based on the investigation results, the bedrock appears to dip from north east to south west at an approximate slope of 4 to 5H:1V in the vicinity of the concrete control structure.

7.2 REHABILITATION OF THE EXISTING EMBANKMENT

The proposed freeboard elevation for the rehabilitated embankment is presented in Section 4.3 of this report. For the most stringent scenario, the freeboard study concluded that the embankment elevation needs to be set at elevation 271.87 m if the stoplogs are removed during the passage of IDF, which is the most realistic scenario. In the existing conditions, the embankment crest is at elevation 271.34 m; therefore, a heightening of 53 cm is required.

If MVCA wants to consider that the stoplogs could not be removed during the passage of the IDF, the embankment would be required at elevation 272.69 m, thus a heightening of 1.35 m.

The design of rehabilitation works is shown in detail on Drawings C01 to C06, with typical sections shown on drawing C06. Rehabilitation of the embankment will include the following features:

- Heightening of the dam crest by 53 cm to the elevation of 271.87m.
- Widening of the dam crest to the minimum of 5 m for embankment stability purposes (the access width for ATV is limited to 1.88 m). Flattening of the downstream slopes to 3H:1V by placement of the fine rockfill material (0 – 150 mm).
- Placement of the riprap (D50 – 200 mm or R50 – OPSS1004) layer (370 mm thick) on the upstream slope.

- Installation of the Soil-Cement-Bentonite cut-off wall from the dam crest to the bedrock foundation.
- Installation of the emergency spillway (canal) in the slope, near the north abutment.

The SCB Cut-off

The proposed soil-cement-bentonite (SCB) cut-off wall is the main design feature of the upgraded earth dam as it will provide an impermeable barrier and will also provide strengthening and some stabilization of the embankment by provision of the cement based CSB mix with design UCS strength up to 1,000 kPa. The SCB cut-off will be installed down to bedrock from the crest. The SCB cut-off wall should be installed at a minimum distance of 0.8 m downstream of the existing wood planking cut-off wall and should have a minimum width of 1.0 m. The thickness of the cut-off wall is based on design requirements to resist hydrofracturing, cracking and erosion as well as the width of the bucket to be used for the excavation of the cut-off wall. Considering the bedrock at the embankment location is fairly shallow, the cut-off wall must extend into the bedrock or dense impermeable till if bedrock is beyond the reach of the excavator bucket. If the bedrock surface proves to be very irregular during construction, smoothing of its surface by excavation equipment might be required. Surface bedrock mapping would help assess this topic.

A granular platform should be built downstream of the proposed cut-off wall location to allow sufficient space for the excavator during construction of the cut-off. Therefore, the width of the crest on the downstream side of the dam should be widened by a minimum of 3 m during construction with a downstream slope of 3H:1V.

The Emergency Spillway

The emergency canal (spillway) geometry was designed in Section 4.6. The design sections of this structure are illustrated on Drawing C006. The canal will be 5 m wide and 1.4 m deep, excavated into the finished embankment. The overflow of the spillway will be controlled by the finished SCB cut-off at the elevation 271.35 m. The canal will be lined with the composite layer of bitumen geomembrane sandwiched between geotextile (BGM layer); this will provide a robust erosion protection of the earth embankment. The SCP cut-off in the spillway will be also wrapped with the BGM layer for erosion protection. The armour stone riprap layer (D50 of 450mm) will be placed over the BGM layer for additional erosion protection during the flood event. Placement of the armour stone over the BGM layer has to be very careful to avoid damages to the BGM, however the double geotextile protection was designed to prevent perforation during construction. Overall, if minor perforation to the BGM layer occurs it will not impact overall long-term performance of the spillway, as it may be in use only during emergency event i.e., once in a few years.

7.3 STABILITY ANALYSIS

The slope stability analyses were performed using Slope/W software which is a component of Geo-Studio 2012 (version 8) developed by Geo-Slope International Ltd and based on the Limit Equilibrium Method. Among the alternative options available for the Limit Equilibrium Method analysis, the Morgenstern-Price Method (1965) was adopted for this study, which satisfies both force and moment equilibriums and considers normal and shear inter-slice forces within the analyses.

Geotechnical slope stability assessment of the embankment was conducted for upstream and downstream slopes with consideration for static, rapid drawdown, IDF and seismic (pseudo-static) loading conditions. The highlights of the analytical model development are presented hereafter:

- The analytical model was developed using the results of a previous geotechnical investigation on the dam, topographical survey and, where applicable, observation during the dam inspections. The analytical model considered geometry, soil stratigraphy and strength parameters of the dam and foundation materials, and phreatic surface.
- The slope stability analyses were performed based on dam conditions during and after the construction of the rehabilitation described above.
- With the limit equilibrium method of slope stability analysis, the Factor of Safety (FOS) was calculated for the particular slope being analyzed. The calculated FOS was then compared to the MNR 2011 acceptance criteria.

7.3.1 Loading Cases and Acceptance Criteria

The Ministry of Natural Resources in Ontario issued a technical guidance for the design and management of dams summarized in the series of technical bulletins, including a Technical Bulletin “Geotechnical Design and Factors of Safety”, issued in August 2011 (MNR, 2011).

Table 7-1 below lists the minimum factors of safety (FOS) required by the MNR 2011.

Table 7-1: MNR 2011 Guidelines Factors of Safety for Dam Embankment Slope Stability

Loading Conditions	Minimum FOS	Zone
End of construction before reservoir filling	1.3	Upstream and Downstream
Long term with Normal Supply Level	1.5	Upstream and Downstream
IDF loading conditions	1.3	Upstream and Downstream
Full or Partial rapid drawdown to Low Supply Level	1.2 to 1.3	Upstream
Pseudo-static under seismic loadings	Greater than 1.0	Upstream and Downstream
Post earthquake	1.1	

The FOS against rotational failure was evaluated using a two-dimensional model, for the following loading conditions:

- Static conditions to simulate long term stability assuming normal water level, higher operating range of 271.28m.

- Pseudo-static conditions to simulate earthquake conditions (seismic coefficient equal to 50% of PGA (EDGM = 0.0505g) was used for horizontal acceleration to simulate 1 in 2,500 earthquake conditions) (normal operating water level).
- IDF water level at 271.56m.
- Drawdown conditions after the pond water level sudden drawdown from the maximum operating water level to minimum regulated water level (upstream failure).
- End of construction before reservoir filling (upstream and downstream failure).

The seismic slope stability of the dam was completed based on a pseudo-static approach. The seismic events were modeled using the 2015 National Building Code seismic peak ground acceleration (PGA) for the site.

Natural Resources Canada (NRCAN) provides an online service to estimate Earthquake Hazard Values based on Geological Survey of Canada's Seismic Hazard Model. The values calculated by this model are used in the National Building Code of Canada (NBCC). Stantec used this model to estimate the Peak Ground Acceleration (PGA) values for the project site.

Earthquake Design Ground Motion (EDGM) values were estimated by factoring PGA values with a seismic coefficient of 0.5 to account for sustained ground motion. This coefficient was found to be considered by Hynes-Griffin and Franklin (USACE, 1984).

In terms of consequence of failure, the dam has been classified as "Moderate". According to MNR 2011, the Earthquake Design Ground Motion (EDGM) acceptance criteria for a "Moderate" Hazard Potential Classification is AEP = 1/1000. However, for the pseudo-static analysis, a PGA was selected based on a "High" consequence dam (AEP = 1/2500) to increase the conservatism of the assessment. The reported PGA for these dams for a return period of 1 in 2500 years (2% probability of exceedance in 50 years) is 0.101 g. An EDGM of 0.0505 g was used for the analyses.

7.3.2 Material Properties

The dams' material properties were conservatively selected based on the previous geotechnical investigation results. Details of selected material properties are presented in **Table** 7-2 below.

Table 7-2: Selected Material Properties for Slope Stability Analyses

Material	Saturated Unit Weight (kN/m^3)	Moist Unit Weight (kN/m^3)	Cohesion C (kPa)	Angle of Friction Φ (°)
Sand and gravel fill	21.5	20.0	0	30
Silt fill	20.0	19.0	0	28
Rockfill	22.0	20.5	0	38
Soil Cement-Bentonite grout	13.0	Not Applicable	1000	0

7.3.3 Analysis Results

Stability analyses were carried out for the existing condition of the dam for both the downstream and upstream slopes.

Slope stability analysis is based on the limit equilibrium method of slope stability analysis. The SLOPE/W component of Geo-Studio 2016 was used. The analysis is performed according to the Morgenstern-Price method. This method has the advantage of satisfying both the force and moment equilibrium equations in the calculation of factor of safety.

SLOPE/W programs is designed to locate the slip surface with a minimum factor of safety. However, in some cases, it is appropriate to consider slip surfaces that do not necessarily produce the theoretical minimum factor of safety but would be more significant in terms of the consequences of failure. For instance, in slopes that contain cohesionless soils, at the face of the embankment slope, the lowest factor of safety may be found for very shallow (infinite slope) slip surfaces. These types of shallow sloughing are considered as maintenance issues and are usually much less important than deeper-seated sliding that may jeopardize the integrity of the dam.

Due to the granular nature of existing dam materials, a minimum slip surface depth of 1.5 m was considered for the stability analyses.

The results of the stability analyses for the embankment are presented in the following table and detailed in **Appendix E**.

Table 7-3: Summary of Slope Stability Analysis Results

Loading Conditions	**Factor of Safety**		
	Target (MNR 2011)	**Downstream**	**Upstream**
End of construction before reservoir filling	1.3	2	1.9
Static after construction (long term)	1.5	1.8	1.5
Pseudo-static after construction (seismic)	1.1	1.5	1.2
Rapid drawdown	1.2-1.3	Not Applicable	1.3
Static IDF conditions	1.3	1.8	1.5

Based on the dam stability results, it is concluded that the Shabomeka Lake dam satisfies all stability criteria as per MNR 2011 Guidelines. t is important to note that this study does not take into account the liquefaction potential of the dam and its foundation. The results of the previous geotechnical investigation show very loose soils that could be liquefied in case of earthquake (BH2 and BH3). These soils were located in the vicinity (within 1 m) of the existing concrete spillway, It is assumed that these soils will be excavated and replaced during construction (see Drawing C006).

8.0 ENVIRONMENTAL

As part of this project, Stantec performed an "Environmental Inventory/Existing Conditions Report". This report is presented in **Appendix D** and documents the various aquatic and terrestrial natural heritage features in the Study Area.

The review of potential impacts and mitigation measures for environmental features, including aquatic and terrestrial habitats that may be affected by the proposed works, will be included as part of the Class EA study, and it will be more specifically tailored to the preferred alternative.

The review of alternatives in the context of potential impacts to the natural environment included the following:

1) Do Nothing or Null Alternative.
2) Complete berm and structure deconstruction and reconstruction (2016 Assessment).
3) Berm rehabilitation and control structure reconstruction (2018 Assessment).

The Do Nothing alternative would not resolve the current deficiencies of the dam, and in the event of a catastrophic failure of the embankment or current structure, the impacts to the natural environment could be quite severe, possibly including, but not limited to:

1) Sudden release of massive volumes of water to Semicircle Creek with rapid escalation of erosive forces.
2) Release of large quantities of sediment to downstream habitat in Semicircle Creek.
3) Sudden draining of lake environment resulting in immediate impacts to fish habitat, possible fish stranding and effects on fish year class strength (if it occurs during spawning period).

The complete reconstruction alternative (2016 Assessment) would require the construction of a diversion channel around the entire work area to facilitate work "in the dry". This would result in increasing the disturbance footprint beyond the current berm to facilitate construction of a diversion, resulting in the loss of additional vegetation and introduction of a new discharge point downstream in Semicircle Creek. The duration of the site disturbance would be increased, with greater risk of impacts associated with weather events occurring during the prolonged construction period.

The berm rehabilitation and control structure reconstruction (2018 Assessment) does not require an increase to the footprint of the existing berm and control structure. The works can be timed to commence soon after normal annual lake drawdown and will include the excavation of a trench in the existing berm and installing an impervious dam core to stop the existing seepage. The embankment crest will be heightened slightly. The existing spillway will be demolished and reconstructed in the current footprint. Low winter flow movement from Shabomeka Lake to Semicircle Creek will be maintained during this portion of the work through the use of a temporary culvert or bypass pumping. The approach also shortens the duration of construction disturbance.

Environmental

The 2018 Assessment option is preferred from an environmental perspective, as it reduces the construction impact zone to the existing dam location, does not any new areas of disturbance and can be completed well within the winter period when lake levels are lowered on a typical annual basis.

The Class EA study discusses the analysis of alternatives in greater detail and was finalized in January 2020.

9.0 PUBLIC SAFETY REQUIREMENTS

9.1 PUBLIC ACCESS

The dam is currently open to operators' access, and this general arrangement will be maintained.

9.2 SIGNS

The public safety signs that are presently in place should be re-installed on the rehabilitated dam.

9.3 HANDRAILS

Since pedestrian access will continue, the design includes handrails on top of the concrete structure to avoid fall hazard.

10.0 RECOMMENDED ALTERNATIVE

The conclusions and recommendations of the previous 2005 and 2016 studies are presented in Section 1 of this report. All these recommendations have been analyzed, revised and improved where necessary. The recommended alternative consists of the features presented in **Table** 10-1.

Table 10-1: Recommended Alternative

	Work Description
Embankment	Construction of a Soil Cement-Bentonite cut-off just downstream of the existing wooden cut-off;
	Crest heightening to elevation 271.87 m;
	Protection of the embankment upstream face with 140-305 mm Riprap;
	Reconstruction of the embankment downstream face with 0-200 mm granular material;
Spillway	Remove existing Spillway and gabion approach wall;
	Construction of new concrete structure with bascule gate with the same dimension as the existing stoplogs including new abutment and wing walls;
Signs	Re-install existing signs
Handrails	On top of the concrete structure

The drawings of the recommended alternative are presented in **Appendix A**.

11.0 COST ESTIMATES

The construction costs, exclusive of HST, was initially estimated by Trow Associate in 2005 to be $109,000. This cost estimate does not reflect the existing market. An estimation of the construction cost was performed in 2016 by Houle Chevrier Engineering to be $679,375 excluding any provisional costs.

As part of the preliminary design of the dam, Stantec performed an estimation for rehabilitation works including 10% contingency and $25,000 for material testing. The total estimated cost for the recommended alternative is approximately **$1,533,990** including provisional allowances. Provisional allowances consist of**:**

- Allowance for geotechnical investigation for rock fault: $45,000.
- Allowance for sediment removal and disposal (accumulated upstream the dam): $12,000.
- Allowance for grouting rock fault: $60,000.

The cost estimate includes all the direct construction costs and some indirect costs (as indicated above). All other indirect costs have not been included in this estimation, such as MVCA Project Management, environmental remedial measure, and permitting costs. The detailed cost estimate is presented in **Appendix B**.

12.0 WORK SCHEDULE

Prior to commencing construction, permits and approvals may be required from Provincial or Federal agencies. These are examined in greater detail in the Class EA study. Once all required permits/approvals have been obtained, construction may commence, and will meet any applicable permit restrictions such as construction timing restrictions, erosion and sediment control requirements, and habitat or species protection measures, for example.

Construction activities cannot commence before lake levels are lowered. Per the OMS manual, the existing fall operation consist of removing one stoplog around September 10th. Three stoplogs are removed the next week (September 17th), and two stoplogs are removed the week after (September 24th). Therefore, a week later, it can be expected that the drawdown will be done (October 1st). Based on this schedule and pending any other scheduling restrictions that may be included in required permits/approvals, the following is the anticipated construction schedule for the dam structure.

Table 12-1: Proposed Works Schedule

	Embankment	Discharge structure
September	• Out of waterworks: Vegetation clearing, layout of the site	• Stoplog removal at normal OMS rate
Sept. 1st to Sept. 15th	• Installation of the cofferdam upstream and downstream of the site. Excavation of a middle trench and placement of cement-bentonite core	• Removal and protection of stoplogs removal structure • Demolition of existing discharge structure
Sept. 15th to Sept. 31st		• Construction of new discharge structure
Oct. 1st to Oct. 30th	• Embankment crest heightening • Placement of riprap on upstream face	
Nov. 1st to Nov.. 30th	• Removal of temporary culvert downstream the dam	• Installation of the bascule gate

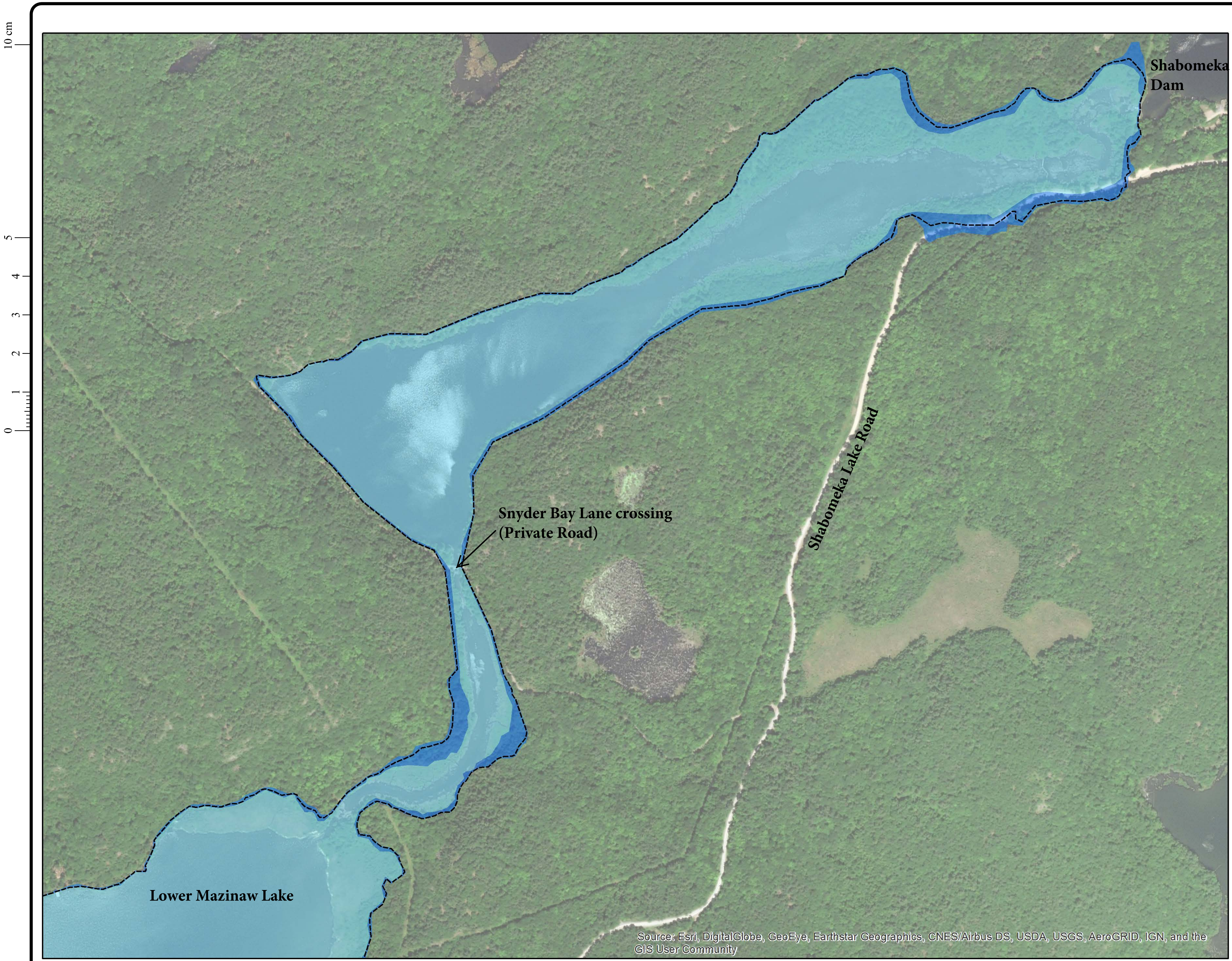
13.0 REFERENCES

- *CDA, 2013: Canadian Dam Safety Guidelines 2007 – Revised 2013.*
- *USBR, 1987: Design of Small Dams.*
- *Handbook of Hydraulics*, Brater, King, Lindell, Wei - TC160.B73 1996
- *Ontario Ministry of Natural Resources and Forestry, 2011, Classification and Inflow Design Flood Criteria.*
- Township of North Frontenac, Transportation Plan, Retrieved on December 1st, 2017, http://gis.northfrontenac.com/pdf/opw1map.pdf
- Houle Chevrier Engineering, "Estimated Construction Costs - Replacement of the Shabomeka Lake Dam, 2016".
- Houle Chevrier Engineering, "Wave Height and Minimum Freeboard - Shabomeka Lake Dam, 2016".
- Houle Chevrier Engineering, "Assessment of Shabomeka Lake Dam, 2016".
- *Wizniack, Shabomeka Lake Dam Mississippi River Project Summary, 1989*.
- *MVCA, Emergency Preparedness Plan*.
- *MVCA, Operation, Maintenance, and Surveillance Manual*.

APPENDIX A
DRAWINGS

APPENDIX B
DETAILED COST ESTIMATE

APPENDIX C
INUNDATION MAPPING



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

Legend

- ⬚ IDF - No Dam Break
- 🟦 IDF - Dam Break
- 🟦 Sunny Day - Dam Break

Client
Mississippi Valley Conservation Authority

Project
Shabomeka Dam

Title
Dam Breach Inundation Mapping

Stantec

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Discipline Hydrotechnical
Scale As shown
Date 2017-11-30

Project Manager
Ammar Taha, P.Eng.
Sequence No.
01 of 01

Serv. char.	Project	Wbs	Disc.	Type	Drawing No.	Rev.
	159100390	400	HY	D	001	01

References

APPENDIX D
ENVIRONMENTAL INVENTORY/EXISTING CONDITIONS REPORT

Shabomeka Lake Dam Environmental Assessment - Environmental Inventory/Existing Conditions Report

Prepared for:
Mississippi Valley Conservation Authority

Prepared by:
Stantec Consulting Ltd.
1-70 Southgate Drive,
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File 159100390
February 19, 2018

Sign-off Sheet

This document, entitled Shabomeka Lake Dam Environmental Assessment - Environmental Inventory/Existing Conditions Report was prepared by Stantec Consulting Ltd. ("Stantec") for the the Mississippi Valley Conservation Authority (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Prepared by ______________________________________
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Reviewed by ______________________________________
(signature)

Dan Eusebi
Senior Environmental Planner

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by the Mississippi Valley Conservation Authority (MVCA) to complete a Class Environmental Assessment and Preliminary Design for the Shabomeka Lake Dam (the Project). The Shabomeka Dam is located at the outlet of Shabomeka Lake (formerly known as Buck Lake) and discharges to Semicircle Creek, leading to Semicircle Lake which subsequently discharges to the southern end of Mazinaw Lake.

Shabomeka Lake is a headwater lake of the Missisippi River system, and is located approximately 10 kilometres northeast of Cloyne in the Township of North Frontenac. The natural heritage Study Area for the Project includes the immediate dam and control berm area, upstream and downstream aquatic and vegetation environments within 50m upstream and 50 m downstream of the dam, and an overview of the surrounding area (**Figure 1, Appendix A**).

1.1 HISTORY

The Shabomeka Lake Dam (formerly Buck Lake Dam) was originally constructed as a timber crib structure operated for log driving around the turn of the century. As the timber trade declined, the dam fell into disrepair. During the 1950's, Ontario Hydro reconstructed the dam for the Mississippi River Improvement Company (MRIC) to take over ownership (Trow, 2005).

During MRIC ownership the dam had undergone some major repair works in 1959 and 1970. The berm and dam structure was rehabilitated in 1988 to address structural, erosional and seepage problems, and included the removal of portions of the berm on either side of the sluiceway, clearing of the base of the lake upstream of the dam to facilitate stability of the structure on bedrock, reconstruction of the berm and repairs to the existing concrete sluiceway.

In January 1991, the MVCA assumed ownership of the dam, and maintained the same operating procedures. From June 14 to 17, 2002, there was over 150 millimetres of rainfall which caused overtopping of the earth embankments on both sides of the control structure (Trow, 2005).

Inspections of the dam conducted between 1990 and 2014 indicated the following deficiencies in the earth embankment and control structure:

- Seepage at the toe of the south embankment, and water flow due to seepage approximately half way up the slope of the north embankment
- Noticeable dips in the elevation of the north and south embankments at the control structures

- Several turtle holes and depressions along the edge of the north and south earth embankments which were considered as potential detriments to berm stability, as well as safety hazards for pedestrian movements
- Gabion baskets had settled and pulled away from the front face of the control structure, resulting in the safety railing become deformed and lifting off the deck of the control structure
- Longitudinal cracks were observed in the top of the north earth embankment on the upstream face of the dam in 2002
- Some erosion of the upstream faces of the earth embankments had occurred
- Locals had built rock dams below the outlet to make it easier to get across with ATVs, potentially causing serious impact to the structure during high flow periods due to the potential for backwater issues

In October 2015, Houle Chevrier Engineering carried out a visual inspection of the dam and provided a geotechnical review of the structure. As a result of the visual inspection, it was recommended that the dam be replaced with a new control structure being placed slightly north of the existing structure to avoid the known bedrock fault (Houle Chevrier 2016).

This Environmental Inventory / Existing Conditions report characterizes the significance and sensitivity of the natural features in the Study Area, and will be used to inform the analysis of dam rehabilitation alternatives, identify potential impacts of the project on these natural features, and recommend appropriate specific mitigation measures to avoid or minimize potential negative impacts once a preferred alternative is selected.

2.0 POLICY OVERVIEW

The natural heritage features and functions within the Study Area were assessed in accordance with the requirements of agency jurisdictions, and the policy and guideline documents described below.

2.1 MISSISSIPPI VALLEY CONSERVATION AUTHORITY

The Mississippi Valley Conservation Authority (MVCA) has the responsibility to regulate activities in wetlands, watercourses and hazard lands (e.g., areas in and near rivers, streams, floodplains, wetlands, slopes and shoreline) through the Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Regulation (O. Reg. 153/06, also known as the “Generic Regulation”). The MVCA implements the regulation by issuing permits for works in or near watercourses, valleys, wetlands, or shorelines, when required.

Development within 120 m of all provincially significant wetlands (PSW) and areas within 30 m of all other wetlands greater than 0.5 ha in size is regulated by the MVCA. There is one unevaluated wetland within the Study Area, located immediately downstream of the dam and associated control berm.

MVCA is the current owner of the Shabomeka Lake Dam.

Under the Conservation Authorities Act, the MVCA has prime responsibility for water management, in terms of water quantity and hazards related to flooding and erosion within areas under its jurisdiction. Section 21(1) of the Act provides administrative powers to the Conservation Authority to, among other things, construct dams, control the flow of surface waters and divert or alter watercourses in order to prevent hazards related to flooding and erosion. The construction, operation, maintenance and retirement (i.e., decommissioning) of dams are valid activities pursuant to MVCA’s mandate and are consistent with Water Management Policy in the Flood and Erosion Control Program Areas.

The Conservation Authorities Act, 1990 (CAA) was created in part to protect and manage water and other natural resources at the watershed level. The CAA is administered by the Ministry of Natural Resources and Forestry (MNRF); however, it enables Conservation Authorities with regulatory responsibility within their respective jurisdictions. Under Section 28 of the CAA and Ontario Regulation 97/04, Conservation Authorities may make regulations under their jurisdiction to prohibit, restrict, regulate or permit certain activities in and adjacent to watercourse, wetlands, valleylands, shorelines, and other hazards. Conservation Authorities represent both provincial and broader watershed interests in the watershed planning process.

Section 28 of the CAA and Ontario Regulation 97/04 is administered by Conservation Authority specific regulations. The Study Area is entirely with the jurisdiction of the MVCA and the implementing regulation is Ontario Regulation 153/06.

2.2 ONTARIO MINISTRY OF NATURAL RESOURCES AND FORESTRY

Specific to dams, the Ontario Ministry of Natural Resources and Forestry (MNRF) administers the Lakes and Rivers Improvement Act (LRIA). The purposes of the LRIA are outlined under Section 2 of the Act and include the following:

- the management, protection, preservation and use of the waters of the lakes and rivers of Ontario and the land under them
- the protection and equitable exercise of public rights in or over the waters of the lakes and rivers of Ontario
- the protection of the interests of riparian owners
- the management, perpetuation and use of the fish, wildlife and other natural resources dependent on the lakes and rivers
- the protection of the natural amenities of the lakes and rivers and their shores and banks,
- the protection of persons and of property by ensuring that dams are suitably located, constructed, operated and maintained

The LRIA requires dam owners to obtain approval from the Ministry of Natural Resources for:

- the construction of new dams
- certain repairs and alterations to existing dams
- certain water crossings and channelization works

MNRF also administers the Endangered Species Act.

2.3 ENDANGERED SPECIES ACT

Provincial species at risk are identified and assessed by the Committee on the Status of Species at Risk in Ontario (COSSARO). The Ontario *Endangered Species Act* (ESA), 2007, protects species listed by COSSARO as threatened, endangered or extirpated in Ontario and their habitats by prohibiting anyone from killing, harming, harassing or possessing protected species, as well as prohibiting any damage or destruction to the habitat of the listed species. Under the ESA, all listed species are provided with general habitat protection aimed at protecting areas that species depend on to carry out their life processes such as reproduction, rearing, hibernation, migration or feeding. For some species, detailed habitat regulations have been passed that go beyond the general habitat protection to define specifically the extent and character of protected habitats.

Activities that may impact a protected species or its habitat require a Permit from the Ministry of Natural Resources and Forestry (MNRF), unless the activities are exempted under the Regulation. The current Ontario Regulation 242/08 identifies activities which are exempt from the permitting requirements of the Act but which are subject to controls outside of the permit process, including registration of the activity and implementation of mitigation approaches. Activities that are not exempted under O. Reg. 242.08 require a complete permit application process.

Consultation with the MNRF, background review of species occurrences and targeted habitat assessments for species at risk determine whether species at risk have the potential to occur in the Study Area. Any species identified as having the potential to occur in the Study Area will be subject to the policies of the ESA.

2.4 FISHERIES ACT

The *Fisheries Act* prohibits causing serious harm to fish unless authorized by the Minister of Fisheries and Oceans, Canada (DFO). This applies to activities in or near waterbodies that support fish that are part of or that support a commercial, recreational, or Aboriginal (CRA) fishery. Since November 25, 2013, proponents can assess projects under the Self-Assessment process. If a project meets the Self-Assessment criteria (DFO 2016), DFO review is not likely required. If the Self-Assessment criteria cannot be met, the proponent should contact DFO for a formal review and possible Authorization under the *Fisheries Act*.

2.5 MIGRATORY BIRD CONVENTION ACT

The federal *Migratory Birds Convention Act*, 1995 (MBCA) protects migratory birds and their nests (S.4). Section 6 of the Migratory Bird Regulations (C.R.C., c. 1035) prohibits the disturbance, destruction or taking of a nest, egg, or nest shelter of a migratory bird. Nest disturbance during the course of vegetation clearing for a project may be considered as “incidental take”, and could be seen as a contravention of the MBCA.

2.6 FISH AND WILDLIFE CONSERVATION ACT

Nests and eggs of wild birds that are not protected by the MBCA, such as raptors (e.g. owls, hawks, and osprey), are protected from harm by the provincial *Fish and Wildlife Conservation Act, 1997* (FWCA).

2.7 PROVINCIAL PARKS AND CONSERVATION RESERVES ACT

The project site is in close proximity to Bon Echo Provincial Park. Any work that is required to occur on regulated Crown Land requires authorization under the Provincial Parks and Conservation Reserves Act, subject to the approval of the park superintendent.

3.0 METHODS

The scope of this Environmental Inventory/Existing Conditions report was designed to encompass a review of background information and a single site visit to ground truth site conditions of the Project and document aquatic and terrestrial environments in the immediate vicinity. Specific methods for the Background Review, Site Investigations and Evaluation of Significance are provided below.

3.1 BACKGROUND REVIEW

Terrestrial background data applicable to the Study Area were obtained through a review of existing documents and information available online. Background resources reviewed included:

- Land Information Ontario (LIO) database (MNRF 2017)
- Natural Heritage Information Centre (NHIC) database (MNRF 2017)
- Ontario Breeding Bird Atlas (Cadman et al. 2007)
- Ontario Reptile and Amphibian Atlas (Ontario Nature 2017)
- Atlas of the Mammals of Ontario (Dobbyn, 1994)
- Bon Echo Provincial Park OLL Additions Field Reconnaissance Report (Ontario Parks, 2001)

The MNRF LIO website was accessed to determine the presence and extent of designated natural features that may be located in the Study Area.

The Ontario Reptile and Amphibian Atlas, Ontario Breeding Bird Atlas and the Atlas of the Mammals of Ontario were accessed to identify species with known ranges that overlap with the Study Area, including species at risk and provincially rare species. The NHIC database was also accessed on the MNRF LIO website to identify records of species at risk and provincially rare species in the vicinity of the Study Area.

In addition to the background data described above, Information Requests were sent to the MNRF and MVCA for natural heritage data, including records of species at risk, provincially rare species, and natural features.

Fish and fish habitat data applicable to the Study Area were obtained through the review of existing documents and information available online. The following background resources were reviewed:

- Land Information Ontario Database (MNRF 2017)
- Fisheries and Oceans Canada Species at Risk Maps (DFO 2017)

- State of the Lake Environment Reports for Shabomeka Lake (MVCA 1998, 2003, 2008, 2013)
- Shabomeka Lake Website (www.shabomekalake.com)
- Shabomeka Spring Littoral Index Netting (SLIN) 2006 Summary Report (MNRF, 2006)
- Status of Shabomeka Lake Trout Recreation Fishery Report (MNRF, 2001)

In addition to the aquatic data described above, an Information Request was submitted to the MNRF and MVCA to request information pertaining to thermal regime of aquatic habitats, fish communities and sensitive habitats.

3.2 FIELD INVESTIGATIONS

Background information was supplemented with a field visit on August 4, 2017 to document existing conditions within the Study Area. The purpose of the field visit was to verify conditions from the desktop review exercise, including ground truthing preliminary observations made from the examination of aerial photography. No comprehensive botanical or faunal surveys were completed. Site photos were taken and representative photos are in Appendix C.

3.2.1 Vegetation Surveys

The vegetation survey included a review of existing vegetation communities associated with the control berm and the immediate surrounding environment.

Vegetation community assessments were conducted using the protocols outlined in the Ecological Land Classification (ELC) System for Southern Ontario (Lee et al. 1998). 2008 ELC code updates were used to classify vegetation communities that were not listed in the 1998 manual.

3.2.2 Wildlife and Wildlife Habitat

No formal wildlife surveys were conducted, however opportunistic sightings of wildlife and/or sign were recorded during the August 4, 2017 site visit. This was determined to be acceptable given that the area of future construction disturbance will be focused on the dam and berm area.

3.2.3 Aquatic Habitat Assessment

During the August 4, 2017 site visit, aquatic habitat was surveyed along the Shabomeka Lake shoreline on the upstream side of the dam and in Semicircle Creek downstream of the dam. The habitat survey consisted of a reconnaissance review of the lake and creek, (i.e. observations of dimensions, bank stability, morphology) and identification of features that typically contribute to fish habitat (i.e. in-water and riparian cover, substrate). Fish collections were not completed as part of the assessment given the availability of background information. Photographs were taken and *in situ* water quality parameters (dissolved oxygen, conductivity, pH and temperature) were measured and recorded upstream and downstream of the dam.

4.0 RESULTS

4.1 BACKGROUND REVIEW

4.1.1 Landscape Context

The Study Area is located on the southern edge of the Canadian Shield, with physiographic landforms characterized by bare rock ridges and shallow till.

Shabomeka Lake lies within the Middle Ottawa Section of the Great Lakes – St. Lawrence Forest Region (Rowe 1972), which is a transition zone between the southern deciduous forests and the coniferous boreal forests of the north. Common upland tree species of the section include Sugar Maple (*Acer saccharum*), American Beech (*Fagus grandifolia*), Yellow Birch (*Betula alleghaniensis*), Red Maple (*Acer rubrum*), Eastern Hemlock (*Tsuga canadensis*), White Pine (*Pinus strobus*) and Red Pine (*Pinus resinosa*). Also common are Balsam Fir (*Abies balsamea*), White Spruce (*Picea glauca*), Trembling Aspen (*Populus tremuloides*), White Birch (*Betula papyrifera*), Red Oak (*Quercus rubra*), and American Basswood (*Tilia americana*) (Ontario Parks, 2001).

The park is within Ecodistrict 5E-11, which includes the area between Algonquin Park to the north and the edge of the Canadian Shield to the south, and from the Ottawa Valley in the east to Lake Simcoe in the west (Ontario Parks, 2001).

4.1.2 Designated Areas

According to the LIO database, and consultation with MNRF and Ontario Parks staff, the Bon Echo Provincial Park boundary abuts the north shore of Shabomeka Lake and Semicircle Creek The park has been given a “Natural Environment” classification, and provides opportunities for high and low intensity recreational activities, while conserving natural and cultural features inside the park’s 6,644 hectares.

Other than Bon Echo Provincial Park, there are no provincially designated natural areas, including: areas of natural and scientific interest (ANSIs), provincially significant wetlands (PSWs), environmentally significant areas (ESAs), national parks, or conservation areas within 120 metres of the Project site. An unevaluated wetland is located approximately 25 m downstream of the Project site on Semicircle Creek and also along the shoreline of Semicircle Creek (**Figures 1 and 2, Appendix A**).

4.1.3 Species at Risk and Provincially Rare Species

The search of various wildlife atlases identified 74 birds, 9 amphibians, 6 reptiles, and 34 mammals with ranges that have the potential to occur in the Study Area. Seven species at risk and provincially rare species were identified as occurring in the square containing the Project site, as follows:

- **Birds:** Eastern Wood-Pewee (special concern)
- **Reptiles:** Common Five-lined Skink (Southern Shield population) (special concern);, Snapping Turtle (special concern), Blanding's Turtle (threatened)
- **Mammals:** Small-footed Myotis (endangered), Little Brown Myotis (endangered), Northern Myotis (endangered)

Four of these species are protected by the ESA: Blanding's Turtle, Small-footed Myotis, Little Brown Myotis and Northern Myotis. In addition to these, MNRF, in a response to an information request, indicated that Eastern Whip-poor-will (threatened) might be present in the area. The species is also protected by the ESA.

The complete list of wildlife, including scientific names, is provided in **Appendix B**.

The bird, mammal, reptile and amphibian range maps provided in the respective atlases are relatively coarse in nature and do not offer precise locations or information on concentrations / densities of records. For example, the Ontario Breeding Bird Atlas records are provided in 10 kilometre (km) by 10 km square grids. The NHIC database provides more precise mapping than the atlases (1 km by 1km squares) and is a better indicator of occurrence of significant species, particularly when used in combination with MNRF and MVCA correspondence. A review of the NHIC database identified records of the following species at risk/provincially rare (S3) species within 1 km of the Study Area (**Table 4-1**). Records in the NHIC database are considered historic (greater than 25 years old), as the last observations recorded were from 1954 for the two snake species, and from 1979 for the bladderwort. A review of the online NHIC records accessed through *Make a Map: Natural Heritage Areas* did not reveal any additional or new records associated with the Study Area.

Table 4-1 Species At Risk and Provincially Rare Species Records

Type	Common Name	Scientific Name	SRank	SARO Status	COSEWIC Status
Plant	Twin-stemmed Bladderwort	Utricularia geminiscapa	S3? S3?	END	END
Reptile	Eastern Ribbonsnake	Thamnophis sauritus	S3	SC	SC
Reptile	Eastern Milksnake	*Lampropeltis triangulum*	S3		SC

4.1.4 Aquatic Habitat Data

Shabomeka Lake reaches depths of approximately 32 metres at its deepest point and has a perimeter of approximately 14 kilometres. It is classified as supporting a cold-water fishery, with Lake Trout identified as a key species inhabiting the lake. Based on a review of data summarized in State of the Lake Reports prepared by MVCA for 1998, 2003, 2008 and 2013, the lake exhibits very good water quality and trends towards being oligotrophic, but has also occasionally exhibited characteristics associated with mesotrophic lakes. Oligotrophic lakes are deeper with very clear water, minimal nutrient inputs and subsequently little algae growth. Mesotrophic lakes are moderately enriched with some nutrient inputs, typically reflected in an increased level of algae and corresponding chlorophyll readings. Regardless of year to year variation in chlorophyll counts and sechhi disk readings, dissolved oxygen levels at greater depths remain high, which is a key component for optimal Lake Trout habitat.

Shabomeka Lake has been recognized by MNRF as key lake for Lake Trout (*Salvelinus namaycush)* management, and identifies the lake as a put-grow-take lake where stocking occurs to support recreational angling opportunities. Spawning habitat is available in the lake, however water level management surrounding the fall drawdown of the lake increases the possibility of exposure of potential spawning shoals which may affect the survival of eggs over the winter. There have been rehabilitation efforts on shoals along the south shore of the lake in the late 1980's, and Lake Trout were observed on one of the rehabilitation sites in 1990. From 2004 to 2006, additional efforts to encourage a native Lake Trout fishery were examined, including maintaining higher water levels during fall drawdown to keep potential spawning areas inundated to the extent possible. Follow-up monitoring suggested that modification to the water levels resulted in little change to the native population. Today, the lake continues to be managed as a put-grow-take fishery rather than a native fishery.

In addition to Lake Trout, data provided by MVCA, MNRF and information contained in State of the Lake reports indicates that the lake supports a variety of other fish species including:

- Lake Whitefish (*Coregonus clupeaformis*)
- Lake Herring (Cisco) (*Coregonus artedii*)
- Burbot (Ling) (*Lota lota*)
- Common White Sucker (*Catostomus commersonni*)
- Smallmouth Bass (*Micropterus dolomieu*)
- Largemouth Bass (*Micropterus salmoides*)
- Rock Bass (*Ambloplites rupestris*)
- Northern Pike (*Esox Lucius*)
- Sauger (*Sander Canadensis*)
- Yellow Perch (*Perca flavescens*)

- Pumpkinseed (*Lepomis gibbosus*)
- Northern Redbelly Dace (*Chrosomus eos*)
- Pearl Dace (*Chrosomus eos*)
- Brown Bullhead (*Ameiurus nebulosus*)

4.2 FIELD INVESTIGATIONS

4.2.1 Vegetation

The Study Area was characterized by a mixture of natural vegetation communities (forests and other treed areas, wetlands), and disturbed areas primarily associated with the dam and control berm, where the vegetation is maintained to prevent the establishment of deep rooting species that may compromise the integrity of the berm. The vegetation communities are summarized in **Table 4-2** and mapped on **Figure 2, Appendix A**. No provincially rare vegetation communities were identified.

Table 4-2 Ecological Land Classification (ELC) Vegetation Types

Property & ELC Vegetation Type	Community Description
SHORELINE COMMUNITIES	
OA Open Aquatic	These are the two open water areas, associated with Shabomeka Lake and Semicircle Creek, on either side of the control berm and dam structure.
MEADOW COMMUNITIES	
CUM1 Mineral Cultural Meadow	This community is present on the top and sides of the control berm and appears as a clearing between FOM to the north and FOD to the south. The community is forb-dominated, with aster (*Aster spp*.), milkweed (*Asclepias syriaca*), goldenrod (*Solidago sp*.), common primrose (*Primula vulgaris*), meadow rue (*Thalictrum sp*.), willow herb (*Epilobium sp.*), blackcurrant (*Ribes nigrum*), flowering raspberry (*Rubus odoratus*), common mullein (*Verbascum Thapsus*), thistle sp., Virginia creeper (*Parthenocissus quinquefolia*), with occasional shrubs of dogwood (*Cornus sp*.), alder (*Alnus sp*.) and whips of poplar (*Populus sp*.) and maple (*Acer sp*.). This area is subject to regular maintenance to prevent the establishment of woody vegetation.
FOREST COMMUNITIES	
FOM Mixed Forest	This upland forest community is extensive and widespread to the north of the dam and berm control structure (CUM1). A mix of coniferous and deciduous trees including maple, oak (*Quercus sp*.), white pine (*Pinus strobus*) and spruce (*Picea glauca*) typify this community. Staghorn sumac (*Rhus typhina*) grows along the edges associated with the disturbed control berm area.
FOD Deciduous Forest	This upland forest community is extensive and widespread to the south of the CUM1. It is dominated by deciduous species such as poplar, beech (*Fagus sp*.), maple, and oak intermixed with lesser numbers of white pine, cedar (*Thuja occidentalis*) and spruce.
MARSH COMMUNITIES	
MAS 3-1 Cattail Organic Shallow Marsh	This community extends along the north shoreline of Semicircle Creek to the northwest of the dam, and is mapped as an unevaluated wetland by LIO. It is dominated by narrow-leaved and broad-leaved cattails (*Typha sp*.) in variable proportions, established on organic soils.
MA Marsh	This is a small island in the mid-channel area of Semicircle Creek immediately downstream of the dam. The island supports growths of graminoid and forb vegetation

The CUM1 community associated with the control berm also contains some wetland species associated with the problem seepage areas on the creek side or western face of the dam. The seepage here is sufficient to provide a microenvironment for the establishment of species such as bracken fern (*Pteridium aquilinum*), sensitive fern *(Onoclea sensibilis)*, sedges (*Carex sp*.) and even occasional cattail. These species do not occur due to natural conditions, but rather are a result of deficiencies in the dam structure, which will be corrected through the design and implementation of the preferred dam rehabilitation approach determined through the EA study.

4.2.2 Wildlife Observations

Two reptiles, or signs thereof, were observed during field investigations on August 4, 2017:

- Three Eastern garter snakes (*Thamnophis sirtalis sirtalis*) were observed within the top gabion basket on the south side of the dam control structure. In addition, MVCA field staff regularly see Northern water snakes around the dam during every inspection visit.
- Two turtle nests that had been predated were noted near the top of the control berm immediately south of the dam structure (**Figure 2, Appendix A**). The nest sites consisted of excavations and scattered egg shell fragments surrounding the opening. The species that created the nests is unknown.

4.2.2.1 Species at Risk and Provincially Rare Species

No targeted surveys were completed for species at risk given that the dam and berm are artificial structures subject to annual disturbance, and that habitat for rare species is not present on these structures.

Field investigations did not survey for presence / absence of endangered bat species, however they may use forested areas (FOD/FOM) in the Study Area for maternity roosts.

4.2.3 Aquatic Habitat Assessment

4.2.3.1 Shabomeka Lake

During the August 4 site visit, water levels in Shabomeka Lake were slightly elevated due to recent heavy rains. In the immediate area of the dam and berm, the shoreline is characterized by an approximate 3 m wide shallow shelf ranging in depth from 10 cm near the shore and sloping gradually to 40 cm at the extent of the shelf, where depth gradually drops off to 3 m in the central area approximately 15 m in front of the dam. In the shelf area, substrates consist of large fractured rock, smaller rounded cobbles and interspersed gravels. Off the south end of the berm exists an embayment where the shallow shelf extends approximately 15 m from the shoreline and deposition provides substrates suitable for the establishment of submerged and emergent aquatic vegetation.

Shoreline substrates are suitable for spawning areas for sunfish and cyprinids, and a single pumpkinseed was noted in the shallows just north of the sluice during the site visit. Although substrates in the shallows are suitable for smallmouth bass spawning areas, water depths, bass tend to prefer spawning areas in 1 to 5 m of water which are likely present in deeper water shoals offshore. They also tend to choose preferred substrates near some form of cover such as larger boulders or logs.

Water chemistry parameters were measured in situ within the entry sluice at a depth of approximately 1.5 m below surface and the following results were recorded:

Temperature	23.7°C
Dissolved Oxygen	8.8 mg/L
Conductivity	65.9 µS/cm
pH	7.66

In the entry sluice to the dam control structure, water depths are approximately 2 deep. Gabion baskets that line the sluice channel are filled with fractured rock that could provide some interstitial habitat for invertebrates. During the site visit, 7 smallmouth bass were observed holding in the sluice channel. The sluiceway does not provide any critical or specific habitat functions and the bass were likely holding in the area due to the current and possible feeding opportunities, such as seeking out small baitfish in the shallow shelves on either side of the sluice.

4.2.3.2 Semicircle Creek

The Shabomeka Lake dam discharges into a short tailrace that flows into a shallow riffle/run approximately 2 m in depth which extends for a distance of approximately 10 m downstream of the dam. Beyond this distance, flow velocities slow considerably in a low gradient environment where the creek is flanked by a large depositional wetland on its north shore. Substrates immediately downstream of the dam consist of large rounded cobble and gravels where flows are swift and provide scouring during elevated discharges from the lake. Approximately 5 m downstream of the dam, a cobble ridge exists where water depths are much shallower at 15cm. This is a fording location utilized by ATV's and other vehicles accessing the north shore of the lake, where cottages are water access only and no formal road access is available. During the August 4 site visit, ATVs were observed fording the creek on two occasions. As a result of this activity, habitat associated with the riffle downstream of the dam is considered disturbed, and periodic vehicle access would be considered a disruption to fish habitat function.

Water chemistry parameters were measured in situ within the entry sluice at a depth of approximately 0.5 m below surface and the following results were recorded:

Temperature	23.7°C
Dissolved Oxygen	8.8 mg/L
Conductivity	65.5 µS/cm
pH	7.77

No fish were observed in the creek during the August 4 assessment, however it is expected that a typical assemblage of baitfish would utilize the area immediately downstream of the dam for feeding, particularly given that dam discharges produce turbulent waters with high oxygen content. Smaller fish that feed on plankton would also be attracted to the discharge to feed on floating organisms in the current from the tailrace. Fish resting in the riffle zone would be subject to periodic disturbance associated with ATVS and other vehicles driving through the creek at this location.

5.0 NATURAL FEATURES AND SENSITIVIES

The following natural heritage features were identified during the Background Review and Field Investigations:

- **Designated Natural Features** – Bon Echo Provincial Park abuts the north shore of Shabomeka Lake and Semicircle Creek. A mapped unevaluated wetland and other unassociated wetland vegetation is present approximately 25 m downstream of the dam and control berm. No other designated features are present in the vicinity of the Project.
- **Fish Habitat** – is present in Shabomeka Lake and Semicircle Creek
- **Other features** – turtle nesting evidence is often present on the top of the control berm.

The existing dam and control berm structure are artificial structures, which as noted in Section 1.1, have been subject to various maintenance and rehabilitation activities over the years. Vegetation associated with the berm is annually disturbed by cutting to ensure that woody vegetation and its root structure does not take hold and compromise the integrity of the dam. The structure and its immediate surrounding environs are disturbed environments with decreased sensitivity to any planned reconstruction of the dam.

As a result of previous disturbances, and ongoing annual disturbances associated with dam and berm maintenance, no sensitive habitats exist in the vicinity of the project area. The preferred alternative for rehabilitation/reconstruction of the dam should include a number of mitigation approaches that will reduce the risk of impact to upstream and downstream environments during construction, and restoration measures should be employed with the intent of re-establishing any minor habitat functionality following construction and stabilization.

6.0 POTENTIAL IMPACTS AND MITIGATION RECOMMENDATIONS

The potential impacts to natural features that might reasonably be expected to occur as a result of the proposed dam improvements have been preliminarily identified and discussed in this section. Potential direct and indirect impacts, associated with the Project have been considered and appropriate mitigation measures recommended.

6.1 POTENTIAL IMPACTS TO VEGETATION

Direct loss will occur where vegetation removal is required to facilitate construction, including temporary work areas. Direct loss of vegetation will be restricted to areas within the dam and berm area and additional property requirements for staging (yet to be identified).

Other potential impacts associated with the Project are limited, but could include siltation and / or spills of deleterious substances into natural areas, in particular nearby downstream wetlands. Sedimentation and spills may alter species composition in adjacent areas by smothering vegetation and introducing toxins and other substances that are harmful to vegetation and wildlife. Additional disturbance may be required to facilitate clean-up activities. Where they occur, these impacts are expected to be localized to the construction area and adjacent areas. Standard mitigation measures are available to reduce these potential indirect impacts to the extent possible.).

6.2 POTENTIAL IMPACTS TO WILDLIFE

Reptiles and other ground-dwelling animals may enter work areas from time to time. Interaction with wildlife during construction may result in direct mortality. Based on field observations of predated turtle nests and garter snakes in the gabion baskets associated with the sluiceway structure, there is potential for direct mortality of reptiles during construction. Snakes are particularly vulnerable during hibernation emergence, re-entrance and basking activities. The gabion basket structures would not be considered candidate hibernacula, as they are set below the waterline for most of the year, and above ground and susceptible to freezing during the winter period when hibernacula are typically used by snakes for overwintering. Turtles are vulnerable during hibernation and during nesting, and migration to and from overwintering sites. Standard mitigation measures are available to reduce potential for interaction with reptiles and other wildlife.

6.3 POTENTIAL IMPACTS TO AQUATIC HABITAT

Potential impacts to fish habitat can include direct habitat loss or indirect impacts to habitat.

Direct impacts may result from the placement of structures or fill below the high water mark, including any modifications to the lake shoreline and river banks associated with dam and berm. If an increase in the Project footprint is required, impacts related to loss of habitat from the increased footprint may be offset by creating or enhancing habitat conditions elsewhere. This may include substrate enhancements to promote spawning habitat.

Indirect impacts may result from the potential for sediment transport from exposed soil surfaces, potential entry of construction debris (e.g. concrete slurry, dust, etc.) into the water and spills associated with refueling of equipment. Sediment introductions can affect fish due to increased turbidity of the water column, which can impair vision and subsequent feeding by fish that are sight-hunters. Suspended sediments can also abrade gill membranes leading to physical stress, and impact prey organism’s behavioral changes (i.e. avoidance, etc.). Heavier sediments can deposit on bottom substrates that may be used for spawning, incubation of juvenile fish, or food production, thereby impacting those habitat functions.

In general, potential impacts to aquatic habitat can be mitigated through site control measures, such as previously mentioned sediment and erosion controls, and other measures to prevent the entry of substances and debris into the water. If in-water work or access is required, construction timing windows can be employed to reduce the risk of impacts occurring during sensitive life periods such as spawning and emergence of young fish. For works in Shabomeka Lake or Semicircle Creek, no in-water work or access should take place from May 1 to July 15. Harm to fish can be reduced through isolation of work areas using coffer dams or other work area isolation techniques, removal of fish from the isolated area and performing works in the dry work area to reduce resuspension of sediments during construction. It may be preferable to schedule any dam reconstruction to the winter months, following the annual fall drawdown of the lake, when water levels are low and manageable. If active flow is present from the lake to Semicircle Creek, it can be maintained through a temporary culvert or pumping as required during construction activities.

6.4 ENVIRONMENTAL PROTECTION MEASURES

6.4.1 Sediment and Erosion Control

Mitigation measures for sedimentation, erosion, and dust control should be implemented to prevent sediment and dust from entering sensitive natural features. The primary principles associated with sedimentation and erosion protection measures are to: (1) minimize the duration of soil exposure; (2) retain existing vegetation, where feasible; (3) encourage re-vegetation; (4) divert runoff away from exposed soils; (5) keep runoff velocities low; and to (6) trap sediment as close to the source as possible. To address these principles, the following mitigation measures are proposed:

- Silt fencing and/or barriers should be used along all construction areas adjacent to any natural areas.

- No equipment should be permitted to enter any natural areas beyond the vegetation protection fencing.
- All exposed soil areas should be stabilized and re-vegetated, through the placement of seed and mulching or seed and an erosion control blanket, promptly upon completion of construction activities.
- Equipment should be re-fueled a minimum of 30 m away from the lake and creek to avoid potential impacts, in the event that an accidental spill occurs. Spill control materials, including absorbent barriers and mats, should be kept on site to quickly address any accidental spills immediately.
- In addition to any specified requirements, additional silt fence should be available on site, prior to grading operations, to provide a contingency supply in the event of an emergency.
- All sediment and erosion controls should be monitored regularly and properly maintained, as required. Controls are to be removed only after the soils of the construction area have been stabilized and adequately protected or until cover is re-established.
- Disturbed natural areas should be restored to pre-construction conditions, or better.

6.4.2 Vegetation and Potential SAR Habitat

The primary mitigation strategy for direct loss of vegetation is to reduce the area of impact to the extent possible. Temporary removal of vegetation cover is mitigated using standard measures for erosion and sediment protection measures identified above, including use of construction barrier fencing along natural areas, and re-vegetation of all disturbed substrates using mixes of native seed suitable for site conditions.

Disturbance to nesting birds covered under the Migratory Birds Convention Act can be avoided through restriction of tree clearing activities between April 1 and August 31.

Suitable Habitat for SAR Bats

Suitable maternity roost habitat will be surveyed prior to construction to determine presence / absence of SAR bats. Surveys will include identification of suitable snag trees during leaf-off (winter months) and acoustic monitoring during the peak maternity season (June). If SAR bats are detected, consultation with MNRF is required to determine authorization requirements under the ESA. Mitigation may include tree removal outside the maternity season, and compensation for loss of snag trees via installation of bat boxes or similar.

6.4.3 Avoidance of Wildlife

Reptile barrier fencing should be installed before any construction activity is initiated if reptile movements into the construction zone pose a concern. Installation should occur before June 1 or after September 1 (i.e., during the reptile active season, and outside of turtle nesting season) to define work areas and inhibit the movement of reptiles into the area.

If construction is initiated during the turtle nesting season, the qualified biologist should also visually inspect the site for turtle nests and adult turtles and direct installation of barrier fencing whereby all nests are avoided. The site should also be inspected to identify and avoid potential snake hibernacula if possible. If potential snake hibernacula features cannot be avoided, a qualified biologist should inspect the feature to determine use by snakes during the suitable season. Typically snakes emerge on warm sunny days in the spring, bask in the sun on surrounding rocks (and potentially roads) to overcome the physiological effects of hibernation, and retreat to the hibernacula at night when temperatures are below freezing. After a few days or weeks, they begin to disperse to the summer range (SWHTG DSS; 2000).

A thorough visual search of the area should be conducted by construction contractors each day to avoid interaction with reptiles. Visual searches should include inspection of machinery and equipment, prior to starting equipment, particularly during the peak reptile activity period from April 15 to November 1. In the event reptiles are encountered during construction work at that location should be stopped until the reptiles are no longer present.

Specifications for reptile barrier fencing should follow Best Practices Technical Note – Reptile and Amphibian Exclusion Fencing (MNR 2013). A qualified biologist should be required as part of the construction contract to be onsite during the installation of reptile fencing to minimize potential for reptiles or habitat to be destroyed or disturbed during construction.

7.0 POTENTIAL AUTHORIZATION/APPROVAL REQUIREMENTS

Depending on the preferred design, a number of approvals or authorizations from various agencies may be required. The following provides a summary of potential approvals that should be considered, and will be determined more fully once the preferred design is selected.

7.1 FISHERIES ACT

The *Fisheries Act* prohibits projects causing serious harm to fish unless authorized by the Minister of Fisheries and Oceans Canada (DFO). This applies to activities in or near waterbodies that support fish that are part of, or that support a commercial, recreational or Aboriginal (CRA) fishery. Since November 25, 2013, proponents can assess projects under DFO's Self-Assessment process. If the Self-Assessment criteria cannot be met, proponents should contact DFO to make a Request for Review which can lead to an advanced formal review of the project by a DFO biologist, resulting in a Letter of Advice or authorization under the *Fisheries Act*. The requirements for DFO involvement and the resulting process are usually determined at the detail design stage, when specific design elements that may potentially impact fish habitat are more clearly defined.

7.2 ENDANGERED SPECIES ACT

Authorization from MNRF is required for any work that may cause harm to ESA species. To determine authorization requirements under the ESA, an Information Gathering Form will be submitted to the MNRF for review and comment.

7.3 LAKES AND RIVERS IMPROVEMENT ACT

Under the Lakes and Rivers Improvement Act (LRIA), approval must be obtained from the MNR for:

- Dams;
- Water Crossings – Bridges, Culverts and Causeways;
- River Channels – Channelization of rivers, including dredging, diverting or enclosing a channel except for the installation or maintenance of a drain subject to the Drainage Act;
- Enclosures;
- Buried Pipelines and Cables – installing cables and pipelines where they will hold back, forward or divert water; or,
- Municipal and Other Drains.

Specific to dams, under Ontario Regulation 454/96, approval must be obtained from the MNRF to construct, decommission, alter, improve or repair a dam that holds back water in a river, lake, pond or stream to raise the water level, create a reservoir to control flooding or divert the flow of water. The application under the *Lakes and Rivers Improvement Act* (LRIA) is required for all heights of dams on permanently flowing watercourses.

7.4 PROVINCIAL PARKS AND CONSERVATION RESERVES ACT

The Bon Echo Provincial Park boundary extends along the north shore of Shabomeka Lake and Semicircle Creek. The Study Area extends into the park boundary, and while the control structure is not within the park boundary, it appears that northern portions of the control berm may be. Any work that is required to occur on regulated Crown Land requires authorization under the Provincial Parks and Conservation Reserves Act, subject to the approval of the park superintendent. It is recommended that consultation be undertaken with Ontario Parks and the superintendent of Bon Echo Provincial Park during the design of the project and limits of any construction and staging areas to obtain clarification on the exact location of the park boundary in relation to the berm structure

8.0 SUMMARY

This Environmental Inventory/Existing Conditions report provides supporting documentation for the Project and describes vegetation communities, potential wildlife and aquatic habitat within the Study Area, and discusses various approvals that may be required for the Project.

The Study Area encompasses a mix of deciduous and coniferous forest environments flanking a disturbed cultural meadow area distinctly associated with the control berm. Subject to annual maintenance, the vegetation associated with the berm is common and widespread, and does not constitute a constraint to any construction activity.

Once the preferred alternative is selected, further recommendations regarding mitigation approaches will be provided, and any additional activities associate with pre-construction surveys such as to document presence / absence of snake hibernacula and / or turtle nesting areas in work areas, or the presence of bat maternity habitat in access road areas, can be identified.

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APPENDIX A: FIGURES

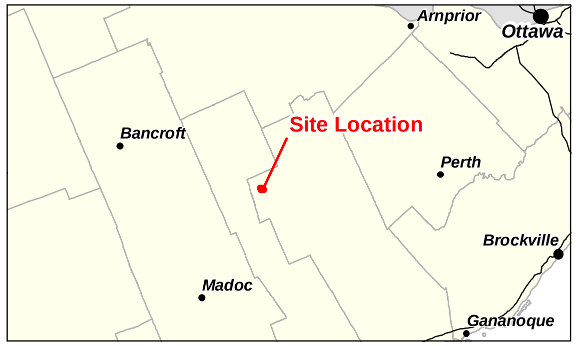
Legend

—	Minor Road
- -	Watercourse (Intermittent)
▨	Provincial Park
▨	Wetland, Not evaluated per OWES

0 100 200 m

1:5,000 (At original document size of 11x17)

Notes
1. Coordinate System: NAD 1983 UTM zone 18N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018.
3. Orthoimagery © First Base Solutions, 2018. Imagery Date, 2008.



Project Location
Frontenac County

159100390 REVA
Prepared by PW on 2018-02-14
Technical Review by AW on 2018-02-14
Independent Review by SS on 2018-02-14

Client/Project
MISSISSIPPI VALLEY CONSERVATION AUTHORITY
SHABOMEKA LAKE DAM
ENVIRONMENTAL ASSESSMENT

Figure No.
1

Title
Project Location

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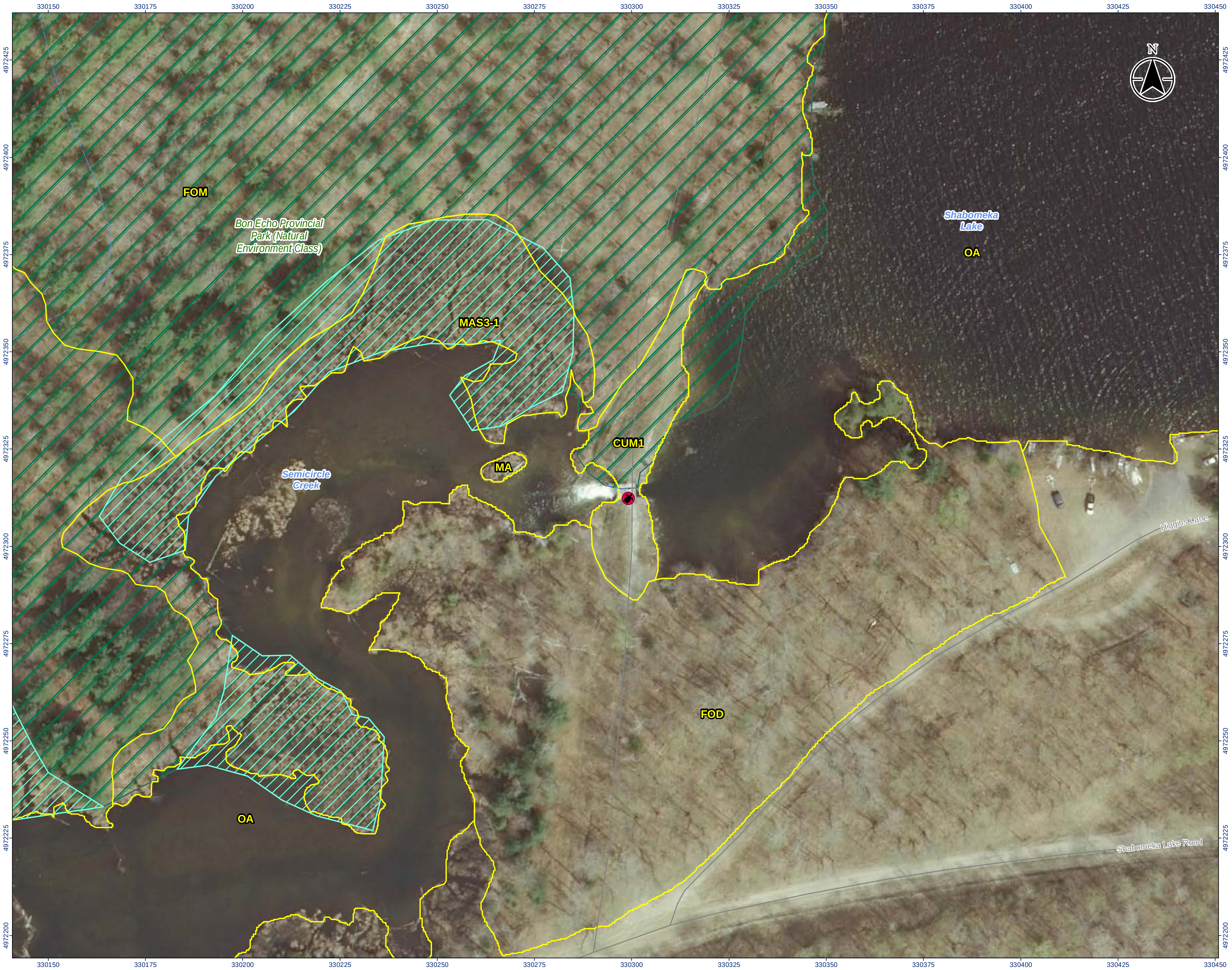

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Stantec

Legend

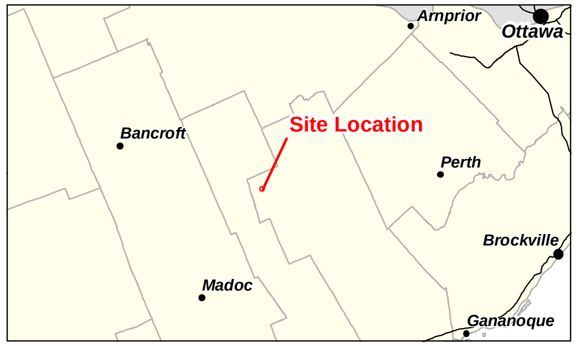
- ● Turtle Nesting Location
- — Minor Road
- - - Watercourse (Intermittent)
- ▨ Provincial Park
- ▨ Wetland, Not evaluated per OWES
- ☐ ELC Vegetation Community

ELC Code	Description
CUM1	Mineral Cultural Meadow Ecosite
FOD	Deciduous Forest
FOM	Mixed Forest
MA	Marsh
MAS3-1	Cattail Organic Shallow Marsh Type
OA	Open Aquatic

0 25 50 m

1:1,000 (At original document size of 11x17)

Notes
1. Coordinate System: NAD 1983 UTM zone 18N
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © Queen's Printer for Ontario, 2018.
3. Orthoimagery provided by © First Base Solutions, 2018. Imagery from 2008.



Project Location
Frontenac
County

159100390 REVA
Prepared by PW on 2018-02-14
Technical Review by AW on 2018-02-14
Independent Review by SS on 2018-02-14

Client/Project
MISSISSIPPI VALLEY CONSERVATION AUTHORITY
SHABOMEKA LAKE DAM
ENVIRONMENTAL ASSESSMENT

Figure No.
2

Title
ELC Vegetation Communities

APPENDIX B:
WILDLIFE SPECIES LIST

Appendix B Wildlife Species List compiled from Background Review

COMMON NAME	***SCIENTIFIC NAME***	**ONTARIO STATUS**	**GLOBAL STATUS**	**COSSARO**	**COSEWIC**
AMPHIBIANS					
Northern Redback Salamander	*Plethodon cinereus*	S5	G5		
American Toad	*Anaxyrus americanus*	S5	G5		
Western Chorus Frog (great lakes - shield)	*Pseudacris triseriata*	S3	G5	NAR	THR
Spring Peeper	*Pseudacris crucifer*	S5	G5		
Bullfrog	*Lithobates catesbeiana*	S4	G5		
Northern Green Frog	*Lithobates clamitans*	S5	G5		
Pickerel Frog	*Lithobates palustris*	S4	G5	NAR	NAR
Wood Frog	*Lithobates sylvatica*	S5	G5		
Northern Leopard Frog	*Lithobates pipiens*	S5	G5	NAR	NAR
REPTILES					
Snapping Turtle	*Chelydra serpentina*	S3	G5	SC	SC
Blanding's Turtle	*Emydoidea blandingi*	S3	G4	THR	THR
Five-lined Skink (south shield)	*Eumeces fasciatus*	S3	G5	SC	SC
Eastern Gartersnake	*Thamnophis sirtalis*	S5	G5		
Northern Watersnake	*Nerodia sipedon sipedon*	S5	G5T5	NAR	NAR
Smooth Greensnake	*Opheodrys vernalis*	S4	G5		
BIRDS					
Wood Duck	*Aix sponsa*	S5	G5		
American Black Duck	*Anas rubripes*	S4	G5		
Mallard	*Anas platyrhynchos*	S5	G5		
Hooded Merganser	*Lophodytes cucullatus*	S5B,S5N	G5		
Common Merganser	*Mergus merganser*	S5B,S5N	G5		
Ruffed Grouse	*Bonasa umbellus*	S5	G5		
Wild Turkey	*Meleagris gallopava*	S5	G5		
Common Loon	*Gavia immer*	S5B,S5N	G5	NAR	NAR
American Bittern	*Botaurus lentiginosus*	S4B	G4		
Great Blue Heron	*Ardea herodias*	S5	G5		
Osprey	*Pandion haliaetus*	S5B	G5		
Red-shouldered Hawk	*Buteo lineatus*	S4B	G5		NAR
Red-tailed Hawk	*Buteo jamaicensis*	S5	G5	NAR	NAR
Virginia Rail	*Rallus limicola*	S5B	G5		
Spotted Sandpiper	*Actitis macularia*	S5	G5		
American Woodcock	*Scolopax minor*	S4B	G5		
Mourning Dove	*Zenaida macroura*	S5	G5		
Ruby-throated Hummingbird	*Archilochus colubris*	S5B	G5		
Yellow-bellied Sapsucker	*Sphyrapicus varius*	S5B	G5		
Downy Woodpecker	*Picoides pubescens*	S5	G5		
Hairy Woodpecker	*Picoides villosus*	S5	G5		
Northern Flicker	*Colaptes auratus*	S4B	G5		
Pileated Woodpecker	*Dryocopus pileatus*	S5	G5		

COMMON NAME	***SCIENTIFIC NAME***	**ONTARIO STATUS**	**GLOBAL STATUS**	**COSSARO**	**COSEWIC**
Eastern Wood-Pewee	*Contopus virens*	S4B	G5	SC	SC-NS
Least Flycatcher	*Empidonax minimus*	S4B	G5		
Eastern Phoebe	*Sayornis phoebe*	S5B	G5		
Great Crested Flycatcher	*Myiarchus crinitus*	S4B	G5		
Eastern Kingbird	*Tyrannus tyrannus*	S4B	G5		
Blue-headed Vireo	*Vireo solitarius*	S5B	G5		
Red-eyed Vireo	*Vireo olivaceus*	S5B	G5		
Blue Jay	*Cyanocitta cristata*	S5	G5		
American Crow	*Corvus brachyrhynchos*	S5B	G5		
Common Raven	*Corvus corax*	S5	G5		
Tree Swallow	*Tachycineta bicolor*	S4B	G5		
Black-capped Chickadee	*Poecile atricapillus*	S5	G5		
Red-breasted Nuthatch	*Sitta canadensis*	S5	G5		
White-breasted Nuthatch	*Sitta carolinensis*	S5	G5		
Brown Creeper	*Certhia americana*	S5B	G5		
Winter Wren	*Troglodytes hiemalis*	S5B	G5		
Sedge Wren	*Cistothorus platensis*	S4B	G5	NAR	NAR
Marsh Wren	*Cistothorus palustris*	S4B	G5		
Veery	*Catharus fuscescens*	S4B	G5		
Swainson's Thrush	*Catharus ustulatus*	S4B	G5		
American Robin	*Turdus migratorius*	S5B	G5		
Gray Catbird	*Dumetella carolinensis*	S4B	G5		
Brown Thrasher	*Toxostoma rufum*	S4B	G5		
European Starling	*Sturnus vulgaris*	SNA	G5		
Cedar Waxwing	*Bombycilla cedrorum*	S5B	G5		
Ovenbird	*Seiurus aurocapilla*	S4B	G5		
Northern Waterthrush	*Parkesia noveboracensis*	S5B	G5		
Black-and-white Warbler	*Mniotilta varia*	S5B	G5		
Tennessee Warbler	*Oreothlypis peregrina*	S5B	G5		
Common Yellowthroat	*Geothlypis trichas*	S5B	G5		
American Redstart	*Setophaga ruticilla*	S5B	G5		
Blackburnian Warbler	*Setophaga fusca*	S5B	G5		
Yellow Warbler	*Setophaga petechia*	S5B	G5		
Chestnut-sided Warbler	*Setophaga pensylvanica*	S5B	G5		
Black-throated Blue Warbler	*Setophaga caerulescens*	S5B	G5		
Pine Warbler	*Setophaga pinus*	S5B	G5		
Yellow-rumped Warbler	*Setophaga coronata*	S5B	G5		
Black-throated Green Warbler	*Setophaga virens*	S5B	G5		
Eastern Towhee	*Pipilo erythrophthalmus*	S4B	G5		
Chipping Sparrow	*Spizella passerina*	S5B	G5		
Field Sparrow	*Spizella pusilla*	S4B	G5		
Song Sparrow	*Melospiza melodia*	S5B	G5		

COMMON NAME	SCIENTIFIC NAME	ONTARIO STATUS	GLOBAL STATUS	COSSARO	COSEWIC
Swamp Sparrow	*Melospiza georgiana*	S5B	G5		
White-throated Sparrow	*Zonotrichia albicollis*	S5B	G5		
Scarlet Tanager	*Piranga olivacea*	S4B	G5		
Indigo Bunting	*Passerina cyanea*	S4B	G5		
Red-winged Blackbird	*Agelaius phoeniceus*	S4	G5		
Common Grackle	*Quiscalus quiscula*	S5B	G5		
Baltimore Oriole	*Icterus galbula*	S4B	G5		
Purple Finch	*Haemorhouspurpureus*	S4B	G5		
American Goldfinch	*Carduelis tristis*	S5B	G5		
MAMMALS					
Pygmy Shrew	*Sorex hoyi*	S4	G5		
Northern Short-tailed Shrew	*Blarina brevicauda*	S5	G5		
Star-nosed Mole	*Condylura cristata*	S5	G5		
Small-footed Myotis	*Myotis leibii*	S2S3	G3	END	
Little Brown Myotis	*Myotis lucifugus*	S4	G5	END	END
Northern Myotis	*Myotis septentrionalis*	S3?	G4	END	END
Eastern Cottontail	*Sylvilagus floridanus*	S5	G5		
Snowshoe Hare	*Lepus americanus*	S5	G5		
European Hare	*Lepus europaeus*	SNA	G5		
Eastern Chipmunk	*Tamias striatus*	S5	G5		
Woodchuck	*Marmota monax*	S5	G5		
Grey Squirrel	*Sciurus carolinensis*	S5	G5		
Red Squirrel	*Tamiasciurus hudsonicus*	S5	G5		
Beaver	*Castor canadensis*	S5	G5		
White-footed Mouse	*Peromyscus leucopus*	S5	G5		
Muskrat	*Ondatra zibethicus*	S5	G5		
Meadow Vole	*Microtus pennsylvanicus*	S5	G5		
Porcupine	*Erethizon dorsatum*	S5	G5		
Coyote	*Canis latrans*	S5	G5		
Grey Wolf	*Canis lupus occidentalis*	S4	G4	NAR	NAR
Red Fox	*Vulpes vulpes*	S5	G5		
Black Bear	*Ursus americanus*	S5	G5	NAR	NAR
Raccoon	*Procyon lotor*	S5	G5		
Marten	*Martes americana*	S5	G5		
Fisher	*Martes pennanti*	S5	G5		
Ermine	*Mustela erminea*	S5	G5		
Long-tailed Weasel	*Mustela frenata*	S4	G5		
Mink	*Mustela vison*	S4	G5		
Striped Skunk	*Mephitis mephitis*	S5	G5		
River Otter	*Lutra canadensis*	S5	G5		
Lynx	*Lynx canadensis*	S5	G5		NAR
Bobcat	*Lynx rufus*	S4	G5		

COMMON NAME	***SCIENTIFIC NAME***	**ONTARIO STATUS**	**GLOBAL STATUS**	**COSSARO**	**COSEWIC**
White-tailed Deer	*Odocoileus virginianus*	S5	G5		
Moose	*Alces alces*	S5	G5		

SUMMARY

Total Amphibians:	***9***
Total Reptiles:	***6***
Total Birds:	***74***
Total Mammals:	***34***

SIGNIFICANT SPECIES

Global (G1-G3):	***1***
National (SC, THR, END):	***7***
Provincial (SC, THR, END):	***7***

Explanation of Status and Acronyms

COSSARO: Committee on the Status of Species at Risk in Ontario

COSEWIC: Committee on the Status of Endangered Wildlife in Canada

REGION: Rare in a Site Region

S1: Critically Imperiled—Critically imperiled in the province (often 5 or fewer occurrences)

S2: Imperiled—Imperiled in the province, very few populations (often 20 or fewer),

S3: Vulnerable—Vulnerable in the province, relatively few populations (often 80 or fewer)

S4: Apparently Secure—Uncommon but not rare

S5: Secure—Common, widespread, and abundant in the province

SX: Presumed extirpated

SH: Possibly Extirpated (Historical)

SNR: Unranked

SU: Unrankable—Currently unrankable due to lack of information

SNA: Not applicable—A conservation status rank is not applicable because the species is not a suitable target for conservation activities.

S#S#: Range Rank—A numeric range rank (e.g., S2S3) is used to indicate any range of uncertainty about the status of the species

S#B- Breeding status rank

S#N- Non Breeding status rank

?: Indicates uncertainty in the assigned rank

G1: Extremely rare globally; usually fewer than 5 occurrences in the overall range

G1G2: Extremely rare to very rare globally

G2: Very rare globally; usually between 5-10 occurrences in the overall range

G2G3: Very rare to uncommon globally

G3: Rare to uncommon globally; usually between 20-100 occurrences

G3G4: Rare to common globally

G4: Common globally; usually more than 100 occurrences in the overall range

G4G5: Common to very common globally

G5: Very common globally; demonstrably
secure

GU: Status uncertain, often because of low search effort or cryptic nature of the species; more data needed.

GNR: Unranked—Global rank not yet
assessed.

T: Denotes that the rank applies to a subspecies or variety

Q: Denotes that the taxonomic status of the species, subspecies, or variety is **questionable**.

END: Endangered

THR: Threatened

SC: Special Concern

2, 3 or NS after a COSEWIC ranking indicates the species is either on Schedule 2, Schedule 3 or No Schedule of the Species At Risk Act (SARA)

NAR: Not At Risk

IND: Indeterminant, insufficient information to assign status

DD: Data Deficient

6: Rare in Site Region 6

7: Rare in Site Region 7

Area: Minimum patch size for area-sensitive species (ha)

H- highly significant in Hamilton Region (i.e.
rare)

m- moderately significant in Hamilton Region (i.e. uncommon)

L1- extremely rare locally (Toronto Region)

L2- very rare locally (Toronto Region)

L3- rare to uncommon locally (Toronto
Region)

HR- rare in Halton Region, highly significant

HU- uncommon in Halton Region, moderately significant

* The Pileated Woodpecker will incorporate smaller woodlots into its homerange, therefore it may not be a true area-sensitive species (Naylor et al. 1996)

LATEST STATUS UPDATE

Odonata: April 2015

Butterflies: July 2014

Bumble Bees: January 2016

Other Arthropods: July 2014

Terrestrial Molluscs: January 2016

Amphibans: July 2014

Reptiles: April 2015

Birds: January 2016

Mammals: January 2016

S and G ranks and explanations: December 2011

NOTE

All rankings for birds refer to breeding birds unless the ranking is followed by N

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APPENDIX C:
SITE PHOTO LOG

Berm south of sluice structure.

Berm north of sluice structure.

Cut vegetation on berm north of sluice structure.

Shoreline substrates looking at staff gauge in lake.

Lake embayment at end of south berm.

Entrance to sluice.

Client/Project Shabomeka Dam EA – Environmental Inventory	Date 04/08/2017
	Project No. 159100390
Title Existing Conditions Photo Log	Page Page 1 of 3

Turtle nest and egg fragments on south berm.

Eastern garter snake on south gabions.

Semicircle Creek immediately downstream of sluice.

Wetland fringe on Semicircle Creek

ATV trail to stream crossing south of dam. Note seepage along edge of berm.

Seepage along front edge of north berm. Note wetland vegetation (sedges).

Client/Project	Date
Shabomeka Dam EA – Environmental Inventory	04/08/2017
	Project No. 159100390
Title Existing Conditions Photo Log	Page Page 2 of 3

ATV fording location in front of dam.

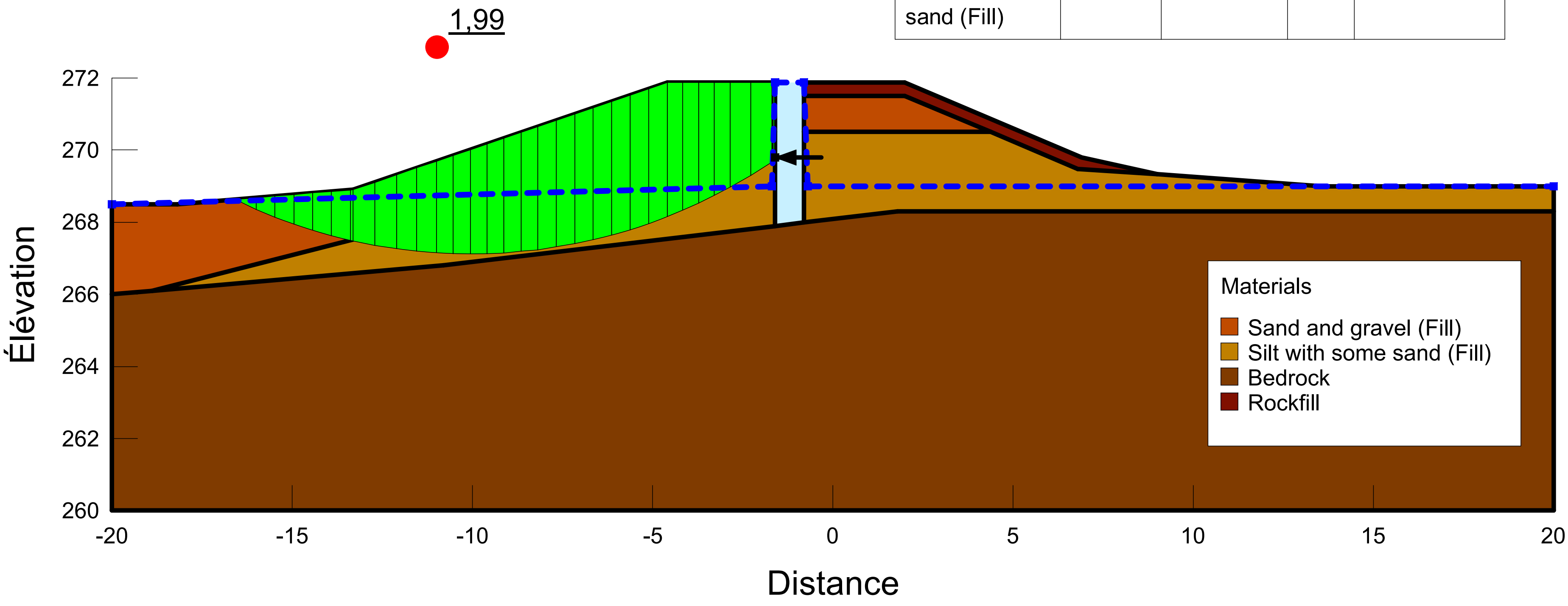
ATV crossing Semicircle Creek downstream of dam.

Berms flanking sluice structure, taken from south berm looking north.

All photos taken August 4, 2017.

APPENDIX E
EMBANKMENT STABILITY ANALYSIS

Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Bedrock				
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19

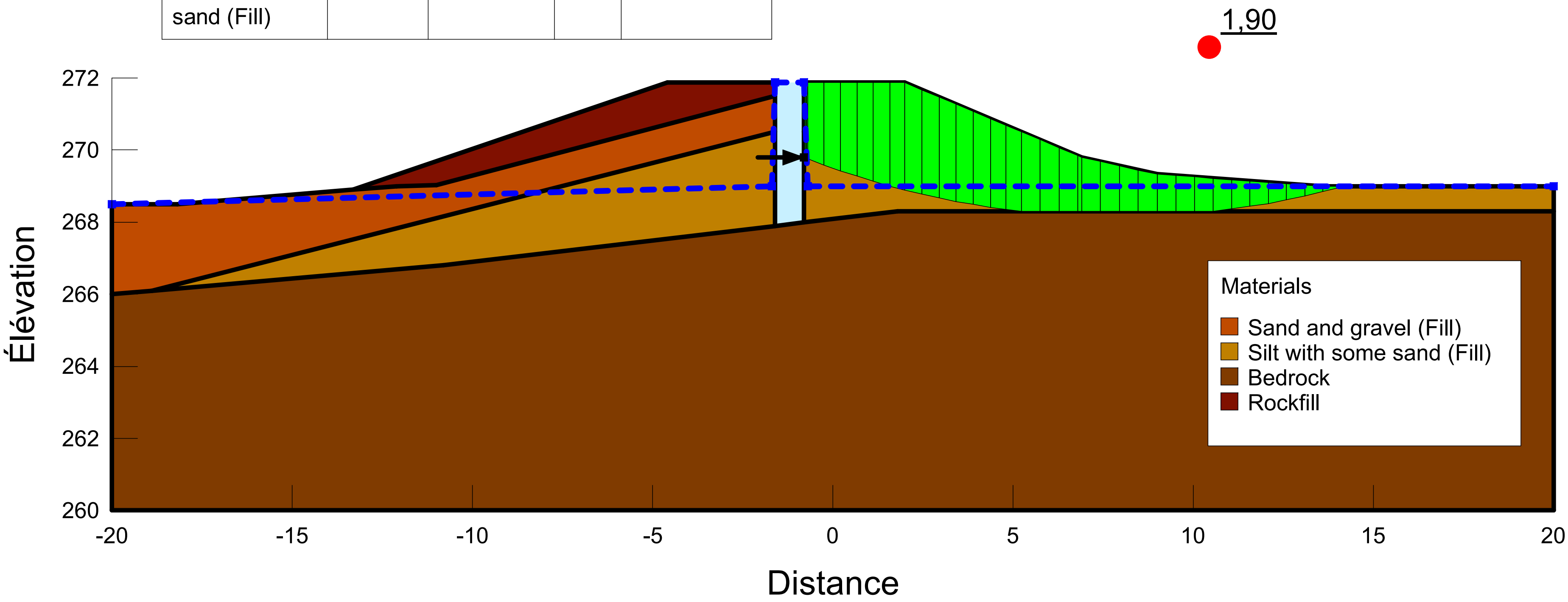


SLOPE/W Analysis

Shabomeka Dam - Downstream - During Construction.gsz

2018-02-06 1:178

Name	**Unit Weight (kN/m³)**	**Cohesion' (kPa)**	**Phi' (°)**	**Constant Unit Wt. Above Water Table (kN/m³)**
Bedrock				
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19



SLOPE/W Analysis
Shabomeka Dam - Upstream - During Construction.gsz
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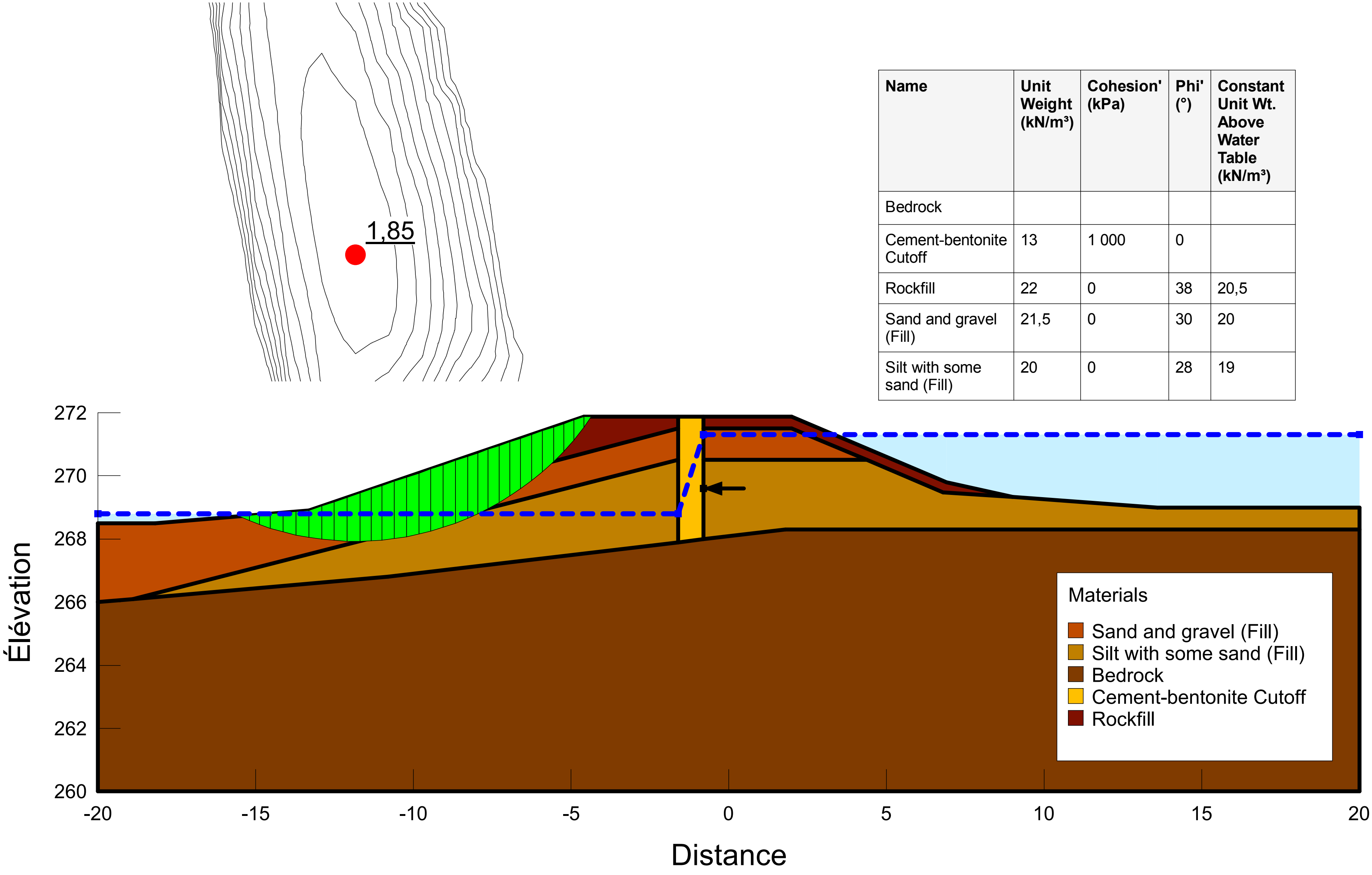


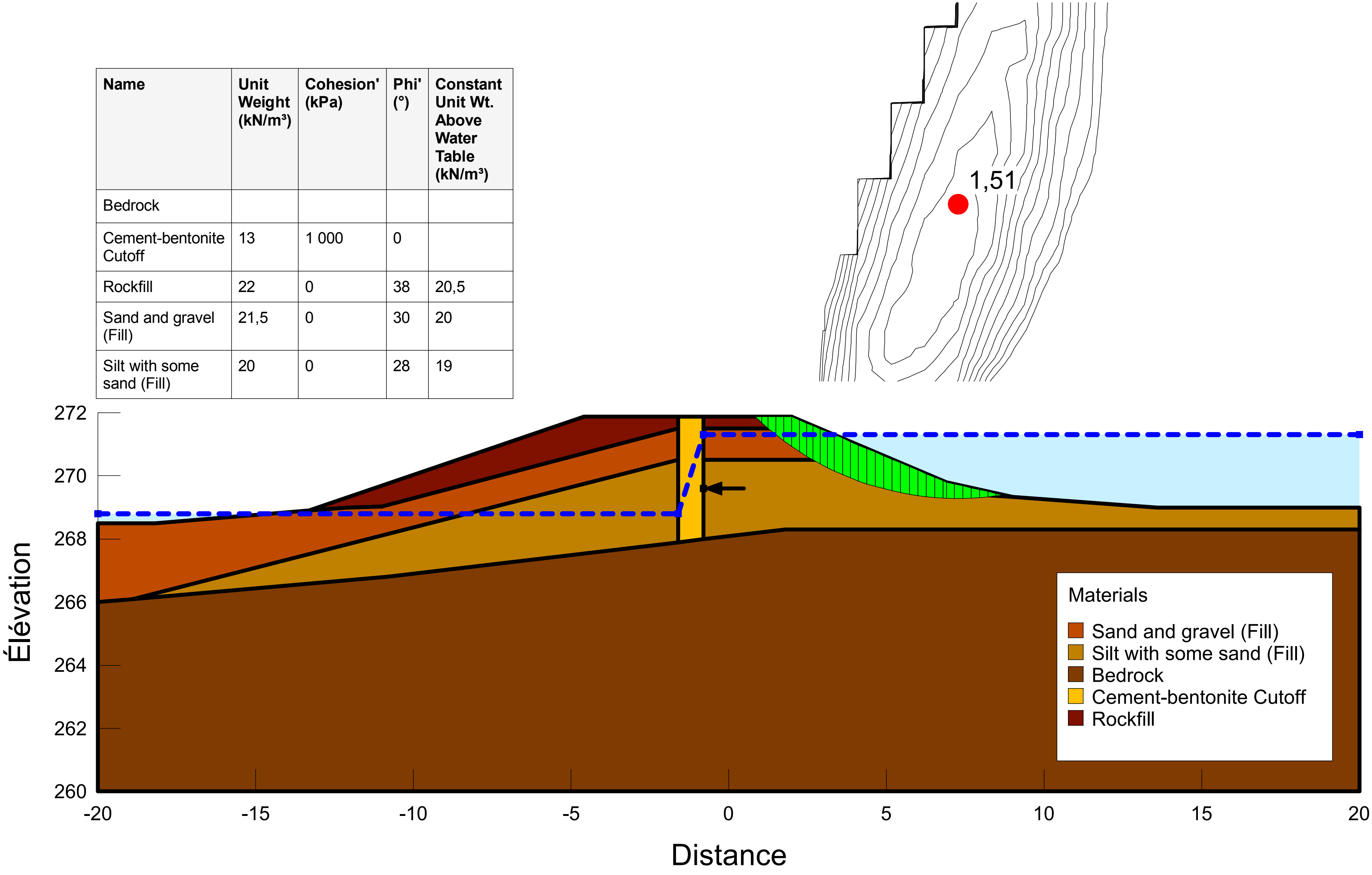
Name	**Unit Weight (kN/m³)**	**Cohesion' (kPa)**	**Phi' (°)**	**Constant Unit Wt. Above Water Table (kN/m³)**
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19

SLOPE/W Analysis

Shabomeka Dam - Downstream Static - After Construction.gsz

2018-02-06 1:178

Name	**Unit Weight (kN/m³)**	**Cohesion' (kPa)**	**Phi' (°)**	**Constant Unit Wt. Above Water Table (kN/m³)**
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19



SLOPE/W Analysis

Shabomeka Dam - Upstream Static - After Construction.gsz

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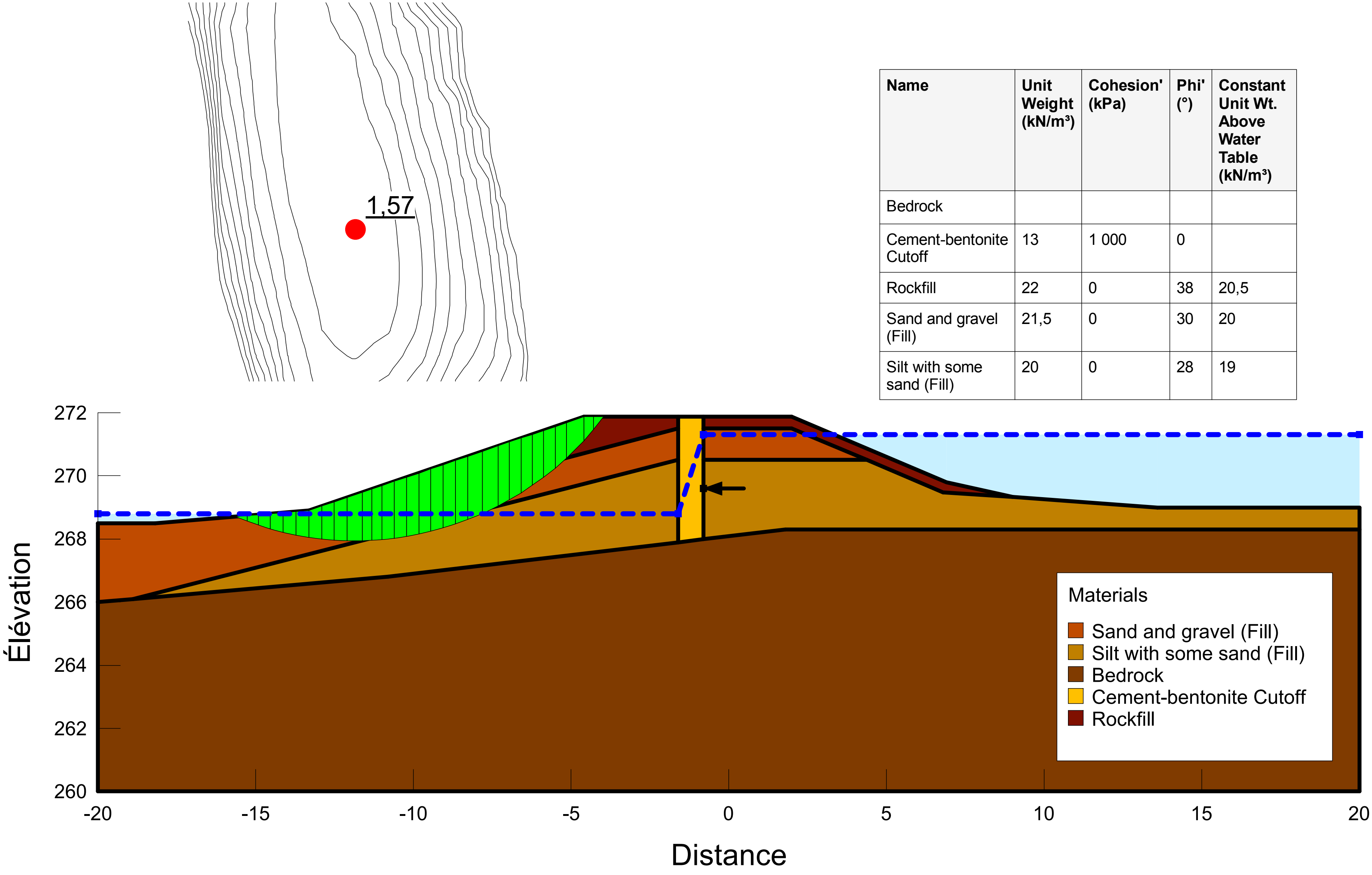

Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19

SLOPE/W Analysis

Shabomeka Dam - Downstream Seismic - After Construction.gsz

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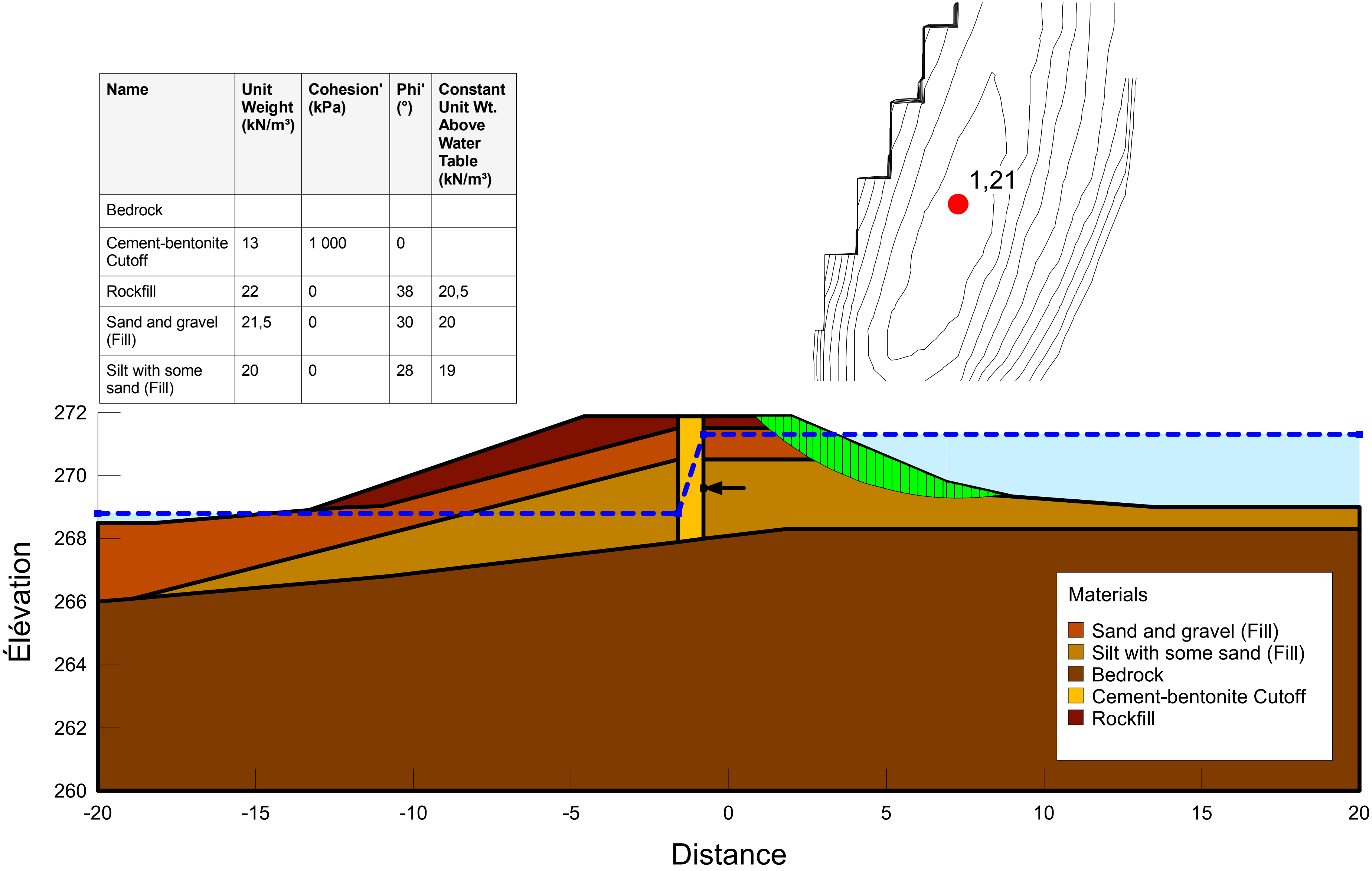

Name	**Unit Weight (kN/m^3)**	**Cohesion' (kPa)**	**Phi' (°)**	**Constant Unit Wt. Above Water Table (kN/m^3)**
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19

SLOPE/W Analysis

Shabomeka Dam - Upstream Seismic - After Construction.gsz

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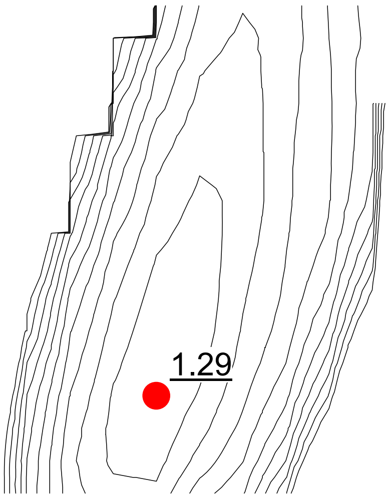
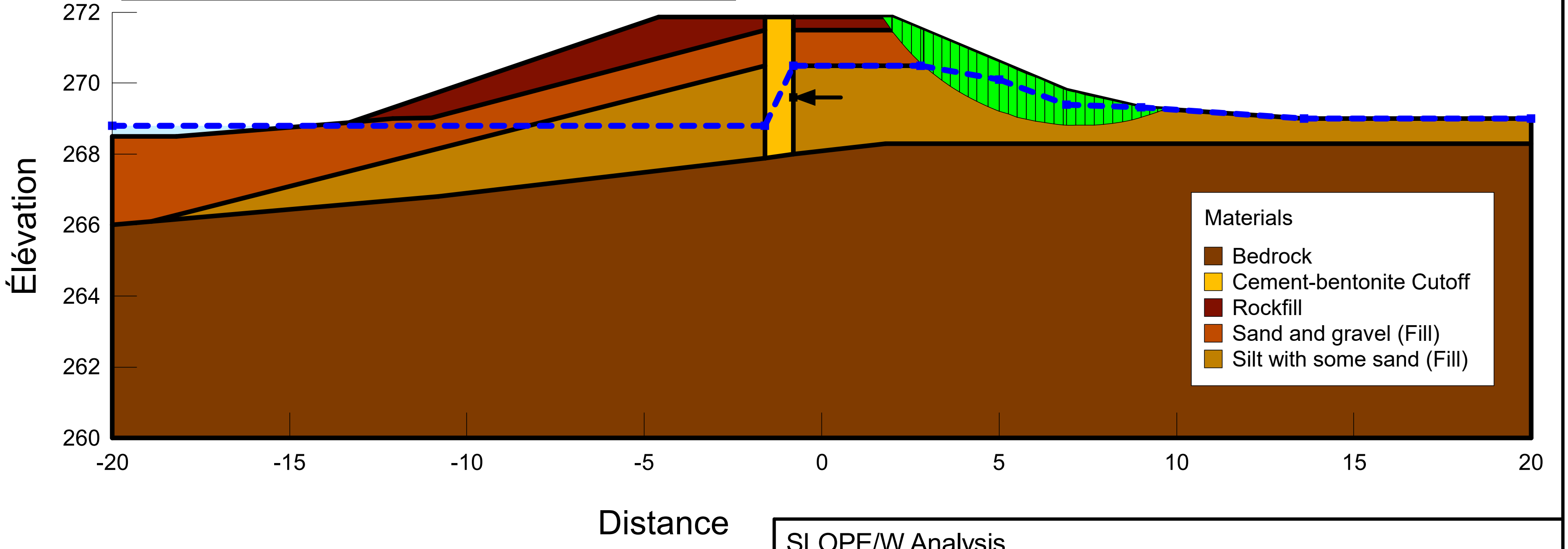
Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19





SLOPE/W Analysis

Shabomeka Dam - Rapid Drawdown-2021.gsz

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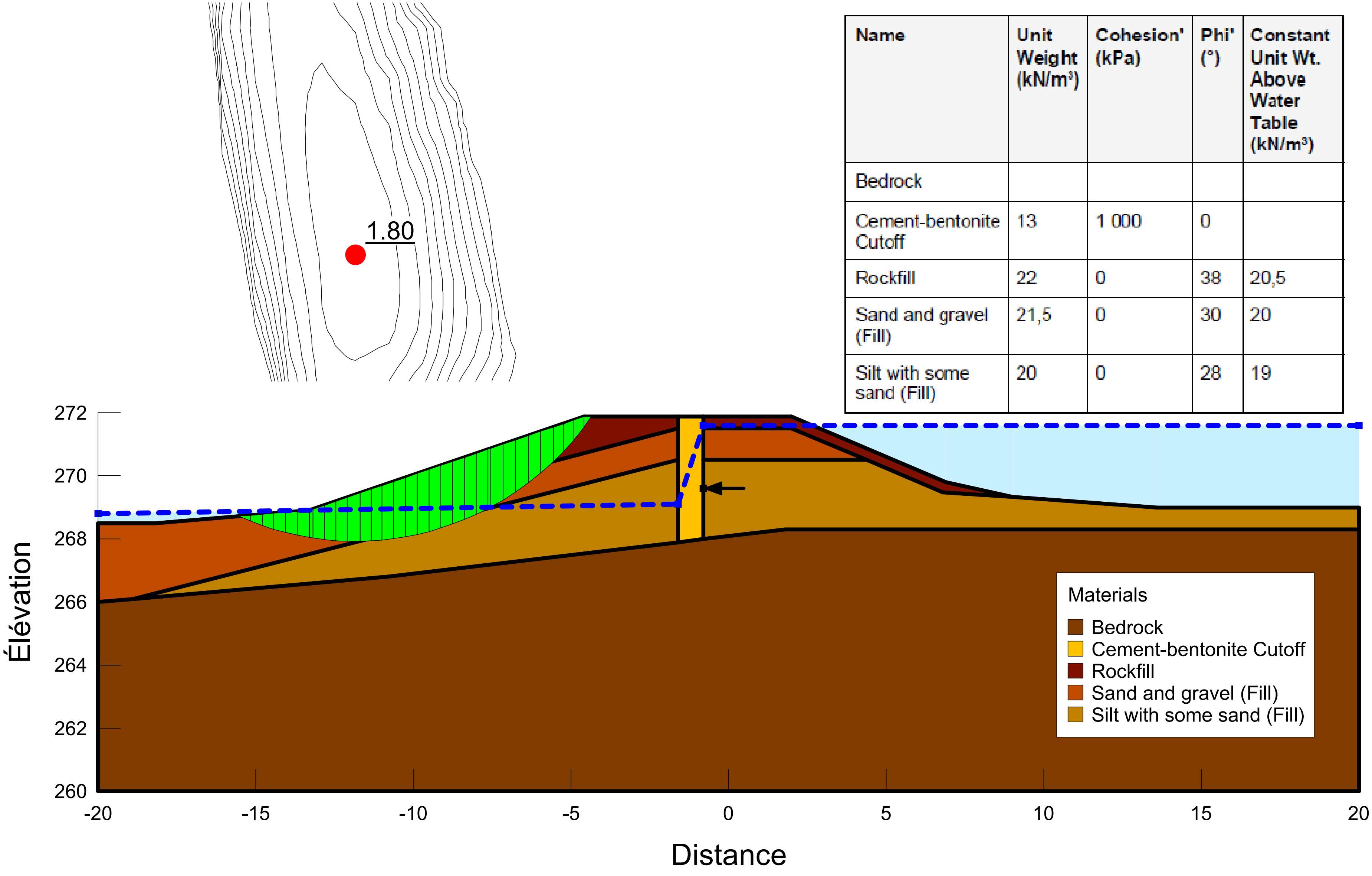


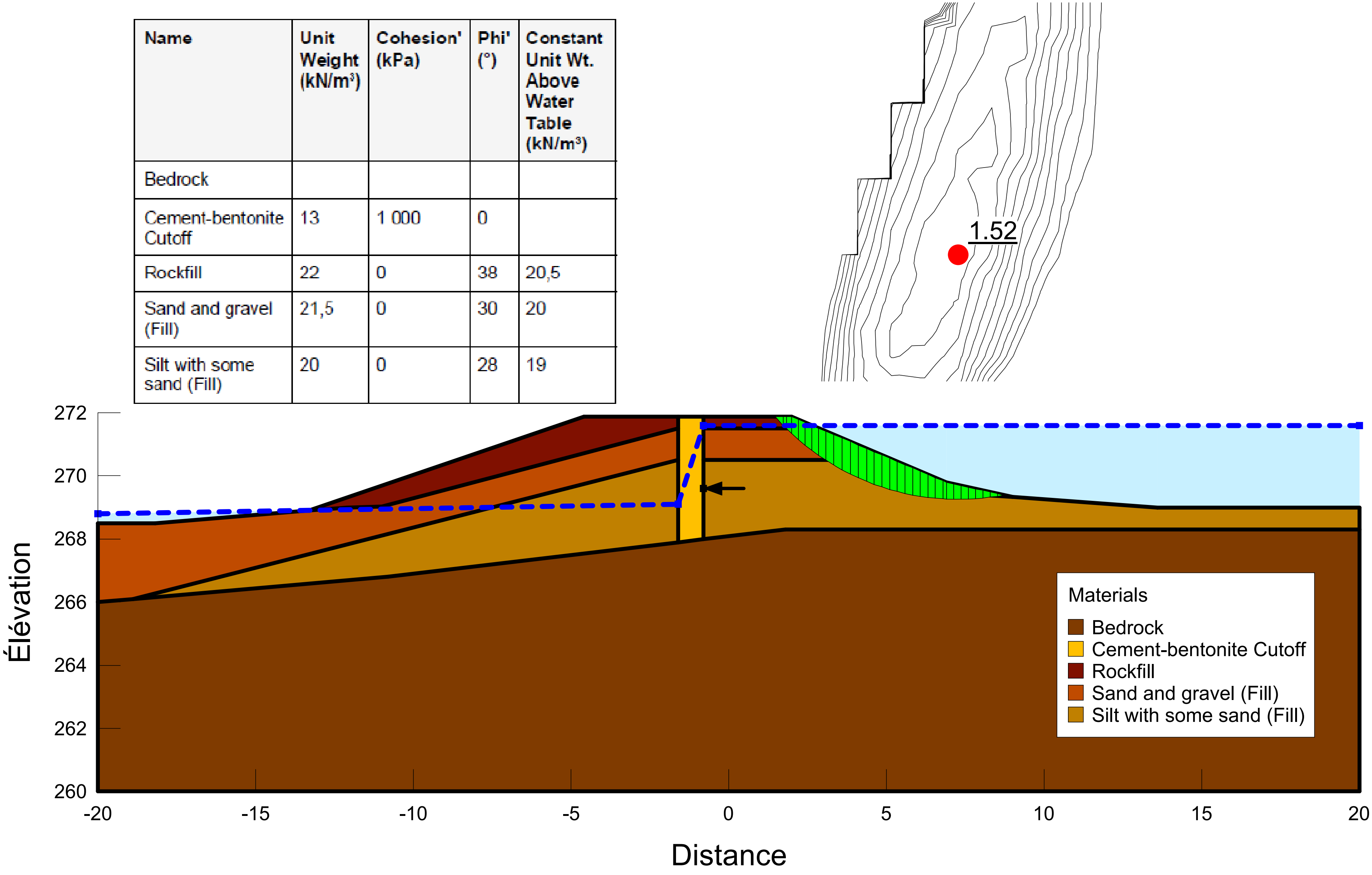
Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m³)
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19

SLOPE/W Analysis

Shabomeka Dam - Downstream Static - IDF.gsz

03/22/2021 1:178

Name	Unit Weight (kN/m^3)	Cohesion' (kPa)	Phi' (°)	Constant Unit Wt. Above Water Table (kN/m^3)
Bedrock				
Cement-bentonite Cutoff	13	1 000	0	
Rockfill	22	0	38	20,5
Sand and gravel (Fill)	21,5	0	30	20
Silt with some sand (Fill)	20	0	28	19



SLOPE/W Analysis

Shabomeka Dam - Upstream Static IDF.gsz

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APPENDIX F
GEOTECHNICAL INVESTIGATION

Geotechnical Investigation
Shambomeka lake Dam
North Frontenac, Ontario

Prepared for:

Mississippi Valley Conservation Authority
P. O. Box 268
2 Miles North of Lanark of Highway 11
Lanark, Ontario
K0G 1K0

Trow Associates Inc.

1595 Clark Boulevard
Brampton, Ontario L6T 4V1
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BRGE00134016a
July 20, 2004

Table of Contents

Appendices

1. Introduction

As part of a safety assessment of the Shambomeka Lake Dam in the Township of North Frontenac, an investigation has been carried out primarily to obtain subsurface information, assess the stability and seepage conditions of the earth embaknments, and to provide input for possible rehabilitation of the dam. The investigation has been completed in accordance with a proposal submitted to Mississippi Valley Conservation Authority. The results of the investigation are presented in this report.

The dam was constructed in the 1950's with earth embankments and a wooden sluice gate which was later changed to concrete. In 1988, rehabilitation work was carried out to the control structure to strengthen the concrete, and clay backfill was reportedly placed on both sides of the structure to minimize seepage. Gabion walls were constructed at that time on the upstream side of the control structure to prevent erosion.

At present the earth embankments are about 30 to 50 m long, and maximum 3 m high. The slope gradients vary from 2H:1V to 3H:1V downstream, and are generally 2H:1V upstream. It appears that the embankments are in more or less their original condition since the 1950's. The concrete structure is 3.8 m wide by 3.2 m high and is founded on rock at about El. 268.5 m. The single stop log bay in the control structure is 2.44 m wide.

2. Field Investigation Procedures

The field investigation consisted of drilling eight boreholes to depths ranging from 1.7 to 5.8 m. The boreholes were drilled on May 6 and 7, 2004, using a track mounted power auger drill rig. Drawing No. 1 attached in Appendix 'D' of this report shows the borehole locations.

In the boreholes, soil samples were taken at 0.76 m intervals of depth using the standard penetration test method. Upon reaching auger refusal, Borehole 3 was advanced another 3.1 m by coring the rock using BX size core barrels which recovered 47 mm diameter rock cores. Next to Borehole 2, another boreholes (Borehole 2A) was augered to the surface of the rock without sampling, followed by coring of the rock for 1.5 m.

Throughout the fieldwork, the water levels in the boreholes were closely monitored. The water level of the lake was also recorded and two samples of the lake water were taken.

As part of this investigation, the visible defects on the exposed faces of the concrete structure were mapped, and six core samples of the concrete were taken for visual examination and strength tests.

The soil samples and rock cores were taken to our laboratory where they were classified in detail, and selectively tested. The tests included moisture content evaluation and grain size analyses on selected soil samples, uniaxial compressive strength tests on two pieces of the rock cores, and pH and sulphate content tests on two soil and two water samples. The results of the moisture content tests and uniaxial compressive strength tests are summarized on the borehole logs. The results of the grain size analyses are shown plotted on Figures 1 and 2 attached in Appendix 'B' of this report.

The borehole elevations were established with reference to the top of the concrete structure. The geodetic elevation of this local benchmark has been established in a previous survey as 271.68 m.

Upon completion of the fieldwork, all boreholes were backfilled with soil cuttings mixed with bentonite.

3. Subsurface Conditions

Boreholes 1 to 6 encountered fill overlying bedrock. The fill materials are mostly silt with some sand. Near the concrete structure, the upper one meter of the fill is composed of silty sand and gravel. Boreholes 7 and 8 located furthest from the concrete structure encountered 0.7 to 2.1 m of fill overlying native silt, silty clay and sand and gravel. The details of the subsurface conditions encountered at the borehole locations are presented in the attached borehole logs, Drawings 4 to 11 inclusive. Drawing No. 2 in Appendix 'D' shows cross sections of the dam at the borehole locations and the simplified soil profiles. In the following paragraphs, a brief description of the relevant properties of the various strata is presented.

It should be noted that the soil boundaries indicated on the borehole logs are inferred from field observations and non-continuous sampling during drilling. These boundaries are intended to reflect transition zones, for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The "Notes on Sample Description" proceeding the borehole logs form an integral part of and should be read in conjunction with the report.

3.1 Fill

All boreholes encountered fill to depths varying from 0.7 to 3.8 m below ground surface. The fill materials are composed of silty sand and gravel near the surface, changing to silt with some sand with depth. In some boreholes, the fill materials contain some rootlets and organic seams. The bottom 0.7 m of the fill in Borehole 2 appears to be crushed gravel, which could be concrete.

Standard penetration blow counts recorded in the fill ranged from of 2 to 41 blows per 0.3 m, which indicate variable degree of compaction from very loose to dense. The very loose conditions were observed in Boreholes 2 and 3 located one metre on either side of the concrete structure. The moisture contents of the fills vary from 8 to 53% (mostly less than 30%). Most of the fill materials were wet.

Figure 1 attached in Appendix B shows the grading curves of two samples of silt fill (2 to 6 % gravel, 9 to 15 % sand, 72 to 80 % silt, 7 to 9 % clay) and one sample of sand and gravel fill (38% gravel, 41% sand, and 21% silt and clay).

3.2 Silt

Native silt was found in Borehole 7 underlying the fill. Figure 2 in Appendix B shows two grading curves of the silt. The tested samples are composed of 0 to 1% gravel, 6 to 16% sand, 83 to 91% silt, and 0 to 3% clay.

SPT 'N' values of 4 to 25 blows per 0.3 m indicate loose to compact conditions. With moisture contents of 18 to 22%, the silt is saturated.

3.3 Silty Clay

An approximately 1.4 m thick layer of silty clay was found in Boreholes 8 under the fill. 'N' values of 20 to 36 blows per 0.3 m recorded in the clay suggest very stiff to hard consistency.

3.4 Sand and Gravel

The clay in Borehole 8 is underlain by sand and gravel in a compact state ('N' values of 22 to 23 blows per 0.3 m).

3.5 Bedrock

All boreholes reached auger refusal, probably on rock, at 1.7 to 4.3 m depth. In terms of elevation, the surface of the rock is at 269.2 to 266.8 m. In the vicinity of the concrete control structure, the rock appears to dip from north east to south west at an approximate slope of 4 to 5 H: 1V.

Bedrock at the site is a gray greywacke, which is a metamorphic rock. In Boreholes 2A and 3, the rock was cored using BX size core barrels, which recovered 47 mm diameter cores. The rock cores were fresh, and total core recovery ranged from 97 to 100%, and the RQD (Rock Quality Designation) of the BX cores varied from 70 to 97%, which indicates good to excellent quality rock. The rock cores were unweathered and contained some fractures dipping at about 0 to 60°. The fracture index (number of fractures per 0.3 m) is 0 to 5. Most of the fracture surfaces are rough and tight. Some of the steeply inclined fractures have a thin (less than 1 mm) film of clay on the fracture surfaces.

Uniaxial compressive strength tests were performed on two sections of the rock cores taken from Borehole 2A and the first core run from Borehole 3. The results were 164.3 and 276.0 MPa, which indicate very strong to extremely strong rock.

3.6 Groundwater Conditions

The groundwater levels in the open boreholes were recorded throughout the fieldwork, and the results are summarized on the borehole logs. The observed groundwater levels ranged from 269.7 to 271.0 m for the boreholes located on top of the earth embankments. The level of the lake was at 271.1 m at the time of the investigation. In Boreholes 5 and 6 located near the toe of the embankment, groundwater was at ground surface (269.0 to 269.2 m).

3.7 Results of Chemical Tests

Two soil samples (Borehole 2, 0.8 to 1.4 m, and Borehole 3, 0.8 to 1.4 m) and two water samples were sent to a certified laboratory for analysis of pH level and sulphate content. The results are shown in the Tables in Appendix 'C' of this report.

The pH of the tested samples range from 7.4 to 8.3. The sulphate content in the soil samples varies from 2.6 to 4.7 µg/g, and that of the water samples vary from 5.2 to 5.3 µg/mL. These results indicate that the soils and the water at the site are not corrosive towards concrete.

4. Concrete Condition

The exposed vertical surfaces of the control structure were visually examined and found to be in generally good condition. There were a few hairline cracks and minor exposure of aggregates but no major defects were noticeable. No efflorescence or sign of aggregate alkaline reaction was observed. The approximate locations of the minor defects and concrete cores are shown in Drawing No. 3 in Appendix 'D' of this report.

The concrete of the core samples appears to be in good condition, with no significant air voids. The compressive strength of the six concrete cores ranged from 36.5 to 57.9 MPa (average 43.7 MPa).

Table 1 – Summary of Concrete Core Compressive Strength

Core Number	1	2	3	4	5	6
Side of Wall (See also Drawing 3)	Exterior		Interior			
Compressive Strength, MPa	36.6	36.5	37.3	44.4	57.9	49.2

5. Seepage Analysis

A seepage analysis has been carried out to assess the quantity of flow and hydraulic gradient through the earth embankments. The geometry of the embankment has been established in a previous survey of the site. During the field investigation, we surveyed cross sections of the dam at the borehole locations and confirmed the previous survey results. The section at Borehole 2 is selected for the analysis.

The analysis was carried out with a finite element program (Seep/W) developed by Geo-Slope International of Calgary, Alberta. A steady state analysis was performed for an upstream lake level of 271.0 m, and a downstream tail water at 269.0 m, which are considered appropriate for normal operating conditions. The embankment fill is assumed to homogeneous and isotropic, and bedrock is assumed to be impervious.

The results of the seepage analysis, including the flow vectors and hydraulic gradient, are shown in Figure 9 in Appendix 'B'. The results show that the phreatic surface exists at the downstream face near the toe of the embankment, which is consistent with the water levels observed in Boreholes 5 and 6. During the fieldwork, we also observed some seepage near the toe of the embankment around Borehole 5. The maximum hydraulic gradient at exit is found to be 0.23.

The calculated quantity of seepage through the dam, using a hydraulic conductivity of 2.5 x 10^{-7} m/sec, is 0.0075 c.m. per day per metre of dam embankment. The actual seepage will depend on the hydraulic conductivity of the embankment fills. Based on the results of grain size analyses, we estimate that the hydraulic conductivity of the embankment fill materials could range from 5 x 10^{-8} m/sec for the silt fill, to about 2 x 10^{-5} m/sec for the silty sand and gravel fill. We expect that the seepage is mainly controlled by the silt fill.

6. Embankment Stability Analysis

We have also carried analyses to assess the stability of the earth embankments . This was accomplished using a computer software (Slope/W), and the embankment geometry and subsurface conditions obtained in this investigation. We have analyzed the embankment sections at Boreholes 1 and 2, using soil strength parameters estimated from the soil classification and index properties. For each section, we analyzed the downstream slope under normal upstream lake level (271.0 m) and high lake level (271.3 m). The phreatic surface in the embankment is as calculated in the seepage analysis. We have also analyzed the upstream slope during a Winter drawdown of lake level to 269 m in about four weeks.

The results of the analyses are shown in Figures 3 to 8 in Appendix 'B', and are summarized in the following table. The factors of safety shown in the table are calculated with the Simplified Bishop's Method for circular sliding surfaces.

Table 2 – Summary of Slope Analyses Results

Section	Slope	Upstream Lake Level (m)	Factor of Safety
Borehole 2	Downstream	271.0	1.33
		271.3	1.25
	Upstream	270.0	1.31
Borehole 1	Downstream	271.0	2.01
		271.3	1.88
	Upstream	270.0	1.59

7. Discussion and Recommendations

7.1 Dam Stability

7.1.1 Concrete Control Structure

The drawing for the dam indicates that the base of the concrete control structure is at Elevation 268.5 m. At that level, and based on the records of the rehabilitation works carried out in 1988, the concrete structure should be founded on bedrock. For the stability analysis of the structure, an allowable bearing capacity of 5 MPa, and an ultimate friction coefficient of 0.7 between the base of the dam and the bedrock, may be used. Since there is no known cut-off wall under the concrete structure, it should be analyzed for the uplift pressures due to full hydrostatic heads at the upstream and downstream ends of the structure (and varying linearly between those two points).

7.1.2 Earth Embankments

The stability analysis results indicate that immediately adjacent to the concrete control structure where the embankment slopes are steepest, the F.S. against rotational failure under normal operating condition is 1.33, which is below the value of 1.50 recommended by the Ontario Dam Safety Guidelines. It should be noted that three dimension effects, which should improve the F.S., were not taken into consideration in the stability analyses. The calculated F.S. of the upstream slope during rapid drawdown is acceptable (1.31).

At a distance of about 10 m from the concrete structure, the F.S. increases to 2.01 for the downstream face under steady state seepage, and 1.59 for the upstream face under rapid drawdown condition. These F.S. are considered adequate.

The results of the stability analyses are applicable to the earth embankments beyond the end of the wing walls at the downstream end of the control structure. The zone of low F.S. is judged to be less than 5 m from the ends of the wing walls. Nevertheless, rehabilitation works should be considered to improve the margin of safety against sliding failure.

7.2 Seepage

The results of the seepage analysis indicates that the expected seepage through the embankments should be relatively minor. For analyzing the effect of the seepage on the storage capacity of the dam, a value of 0.5 c.m. per year per metre of embankment may be used.

The seepage analysis also indicates a phreatic surface exiting at the toe of the embankment at Elevation 269 m, and that the exit hydraulic gradient is 0.23. For gravel, this hydraulic gradient is considered sufficiently low such that piping failure is unlikely to occur. For fine sand, an exit gradient not exceeding 0.15 is required. Silty sand and gravel was encountered at the surface of Borehole 6, but silty sand was found in Borehole 5. It is recognized that the embankments have been under the same condition since 1988 without evidence of significant piping or erosion problems. Nevertheless, to ensure long term safety against piping failure, consideration should be given to placing a zone of gravel or a drainage blanket at the downstream toe of the embankment.

Although the concrete structure is founded on bedrock, some seepage under the base of the structure is likely to be occurring since the structure does not have a cut-off wall. Moreover, seepage is also likely occurring along the exterior faces of the concrete wall, and the seepage pressure is likely causing the loosening of the backfill adjacent to the walls, as indicated by the very loose conditions of the backfill in Boreholes 2 and 3. So far the seepage has not resulted in noticeable erosion/stability problems. In the long term, consideration should be given to providing a cut-off at the concrete structure into the earth embankment.

7.3 Earth Pressure

For the structural and stability analyses of the concrete control structure, the following lateral earth pressure parameters may be used. The earth pressure on the walls of the structure may be calculated as follows:

$$p = K \cdot \gamma \cdot d$$

where
p = earth pressure intensity at depth d, kPa
K = earth pressure coefficient = 0.5
γ = total unit weight of backfill = 20 kN/m^3
d = depth below top of wall, m

Below the phreatic surface, the submerged soil unit weight should be used, and water pressure should be taken into consideration.

8. General Comments

Trow Associates Inc. should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, Trow Associates Inc. will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

More specific information with respect to the conditions between samples, or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, during the future development of the property, conditions not observed during this evaluation may become apparent; should this occur, Trow Associates Inc. should be contacted to assess the situation, and additional testing and reporting may be required. Trow has qualified personnel to provide assistance in regards to future geotechnical and environmental issues related to this property.

9. Closure

We trust that this report has provided all the necessary information for the safety assessment of the dam, and the design of the rehabilitation works. Should there be any questions regarding this report, please do not hesitate to call the undersigned.

Trow Associates Inc.

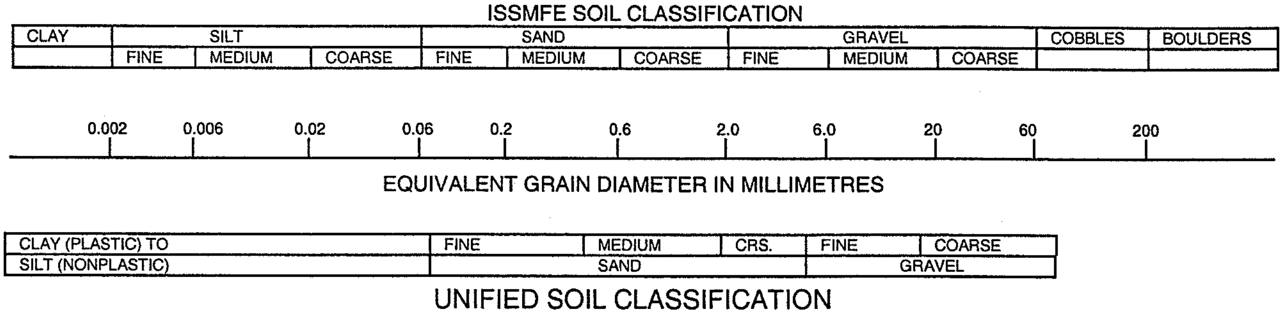
James Ng, P.Eng., M.Eng., MICE
Senior Engineer
Geotechnical Division

Stan Gonsalves, P.Eng.
Principal Engineer

Appendix A: Borehole Logs

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Society for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by Trow Consulting Engineers Limited also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

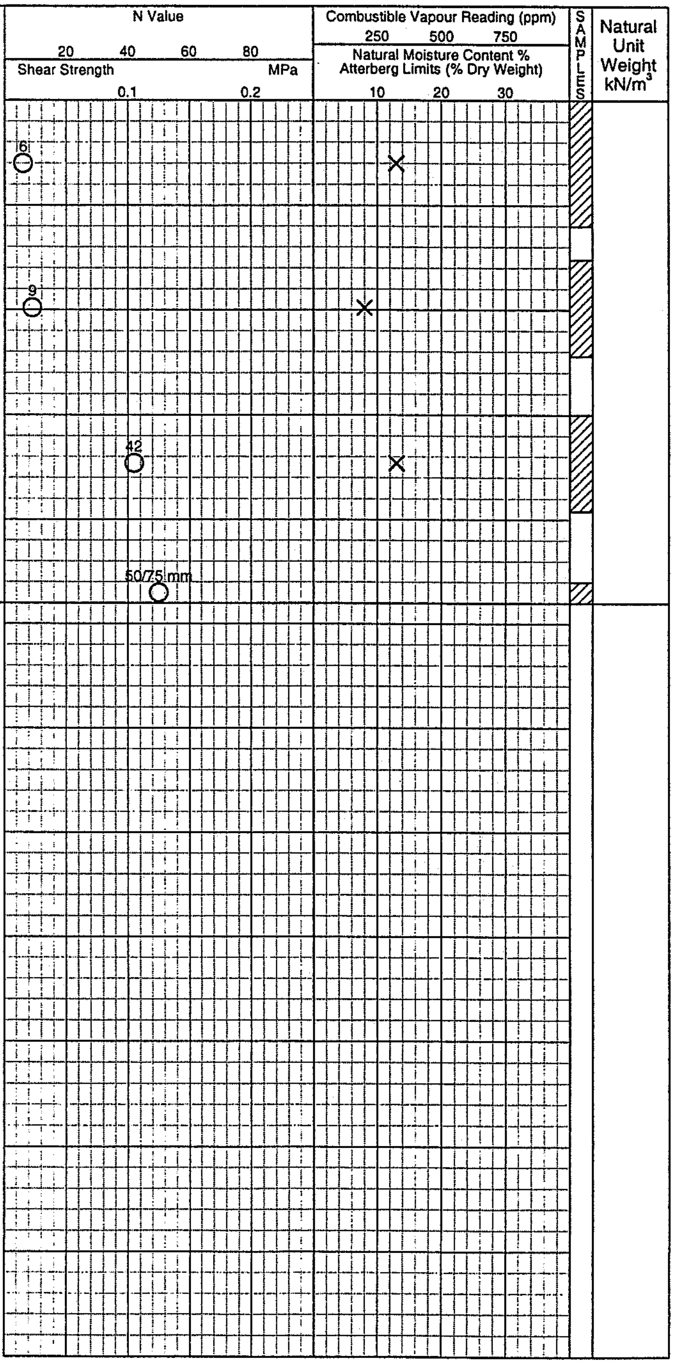
Log of Borehole 1

Project No. brge00134016a
Drawing No. 4
Project: Shabomeka Lake Dam
Sheet No. 1 of 1
Location: Region of Frontenac, Ontario

Date Drilled: 06.05.2004
Drill Type: Solid auger
Datum: Geodetic

Auger Sample ☒
SPT (N) Value ○ ▨
Dynamic Cone Test —
Shelby Tube ■
Field Vane Test ✚ S
Combustible Vapour Reading ☐
Natural Moisture ×
Plastic and Liquid Limit ⊢—○
Undrained Triaxial at % Strain at Failure ⊕
Penetrometer ▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH
		Ground surface	271.60	0
	▩	**FILL** sandy silt some gravel wet brown		
				1
▼			270.40	
		trace of organics		
				2
		boulder	269.2	
		End of borehole Auger refusal probably on rock		



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Trow

Time	Water Level (m)	Depth to Cave (m)
Completion	1.9	2.2
5 pm 07.05.2004	1.3	1.5
09.05.2004	1.2	1.3

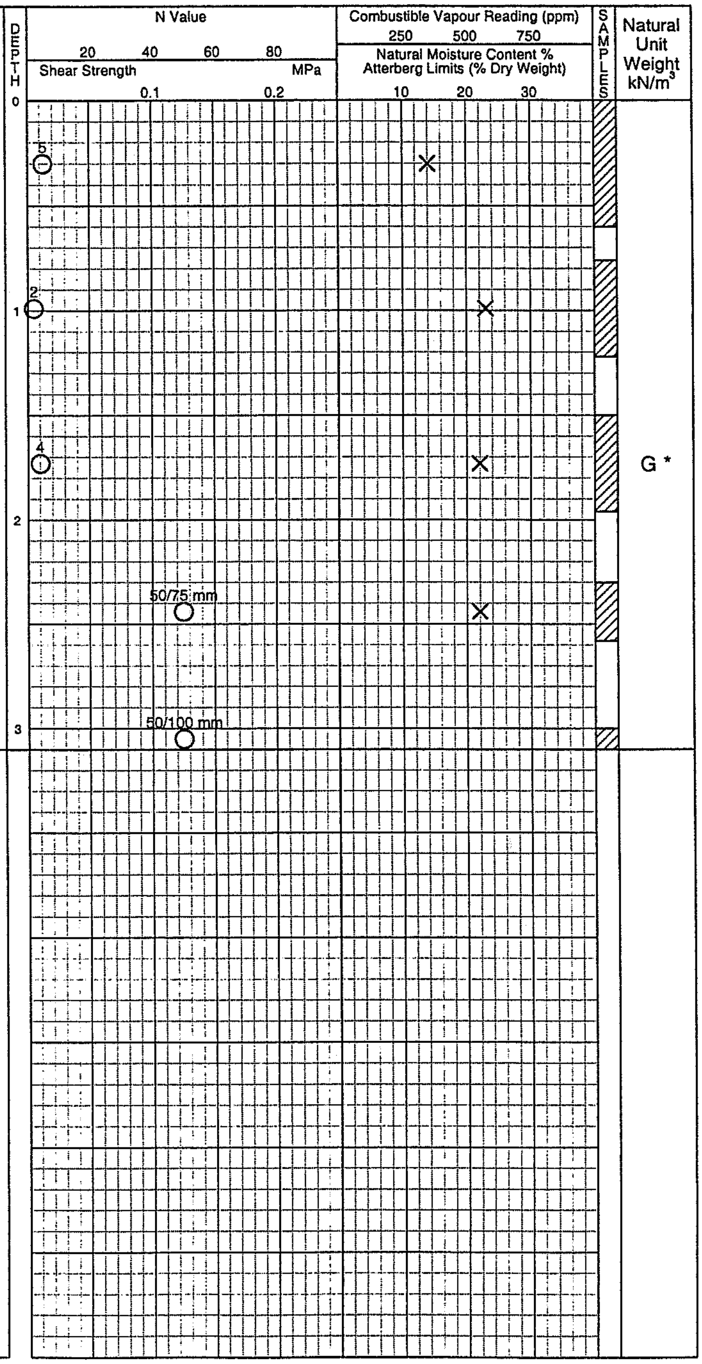
Log of Borehole 2

Project No.: brge00134016a
Drawing No. 5
Project: Shabomeka Lake Dam
Sheet No. 1 of 1
Location: Region of Frontenac, Ontario

Date Drilled: 06.05.2004
Drill Type: Solid auger
Datum: Geodetic

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├─○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	Symbol	Soil Description	ELEV. m
		Ground surface	271.40
	▨	**FILL** sand and gravel, some silt trace of roots, very moist, gray	
	▨	silt, some sand trace of clay wet brown	270.5
	▨	crushed gravel	269.0
		End of borehole Auger refusal probably on rock * G = grain size analysis	268.3



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Time	Water Level (m)	Depth to Cave (m)
Completion	0.7	2.2
5 pm 07.05.2004	wet cave	0.3
09.05.2004	wet cave	0.0

Trow

Log of Borehole 2A

Project No. brge00134016a

Drawing No. 5A

Project: Shabomeka Lake Dam

Sheet No. 1 of

Location: Region of Frontenac, Ontario

Date Drilled: 06.05.2004

Drill Type: Solid auger and coring

Datum: Geodetic

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	N Value 20 40 60 80 Shear Strength MPa 0.1 0.2	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 10 20 30	SAMPLES	Natural Unit Weight kN/m³
		Ground surface	271.40	0				
		Augered to 2.1 m without sampling For overburden condition, see Borehole 2		1				
		Auger refusal	269.2	2				
	▨	**BEDROCK** greywacke switched to coring see core log on next page	267.7	3			■	
		End of borehole						

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Trow

Time	Water Level (m)	Depth to Cave (m)

CORE LOG

BH NO. 2A

PROJECT Shabomeka Lake Dam	**ORIENTATION** Vertical	**ELEVATION (m)** 271.4	**DATUM** Geodetic	**PROJECT NUMBER** brge00134016a
LOCATION Region of Frontenac, Ontario	**DATE STARTED** 05/06/04	**COMPLETED** 05/06/04	**LOGGED BY**	**DRAWING NUMBER** 5A
CLIENT Mississippi Valley Conservation Authority	**DRILLER**	**DRILL TYPE** Power auger	**CORE BARREL** BX	**SHEET** 2 of 2

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	JOINT CHARACTERISTICS NO. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE (mm)	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NUMBER	RECOVERY (%)	RQD	WATER RECOVERY (%)	WATER COLOUR
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
269.2	3	▨	**BEDROCK** greywacke fresh 6 fractures in 1.2 m gray very strong Uniaxial compressive strength at 2.4 to 2.5 m depth = 164.3 MPa	1 1 1 1 1 1	C C C C C C	F V D F F V	C C C C C C	RP RP RP RP RP RP	T T T T T T				■	1	98	73	100	Gray
268.0		▨	Core lost in hole															
267.7	4 5 6 7 8		End of Borehole at 3.7 m															

Log of Borehole 3

Project No. brge00134016a

Drawing No. 6

Project: Shabomeka Lake Dam

Sheet No. 1 of ·

Location: Region of Frontenac, Ontario

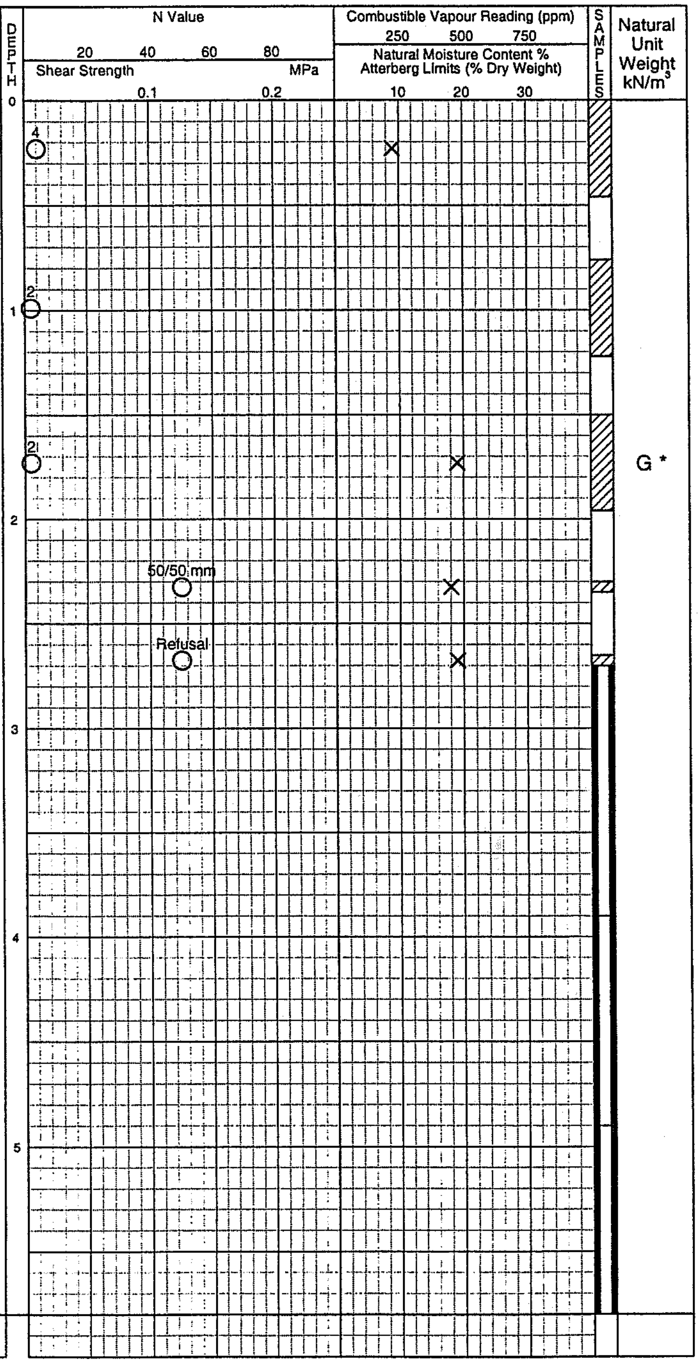
Date Drilled: 07.05.2004

Drill Type: Solid auger and coring

Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├─○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	Soil Description	ELEV. m
	Ground surface	271.30
	FILL sand and gravel, some silt moist, brown	
▼		271.00
	silt some sand trace of clay wet brown	270.6
	metal wire	
	BEDROCK greywacke switched to coring see core log on next page	268.6
	* G = grain size analysis	
	End of borehole	265.5



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Time	Water Level (m)	Depth to Cave (m)
Completion	0.2	
5 pm 07.05.2004	0.2	0.7
09.05.2004	0.3	0.4

Trow

CORE LOG

BH NO. 3

PROJECT Shabomeka Lake Dam	**ORIENTATION** Vertical	**ELEVATION (m)** 271.3	**DATUM** Geodetic	**PROJECT NUMBER** brge00134016a
LOCATION Region of Frontenac, Ontario	**DATE STARTED** 05/07/04	**COMPLETED** 05/07/04	**LOGGED BY**	**DRAWING NUMBER** 6
CLIENT Mississippi Valley Conservation Authority	**DRILLER**	**DRILL TYPE** Power auger	**CORE BARREL** BX	**SHEET** 2 of 2

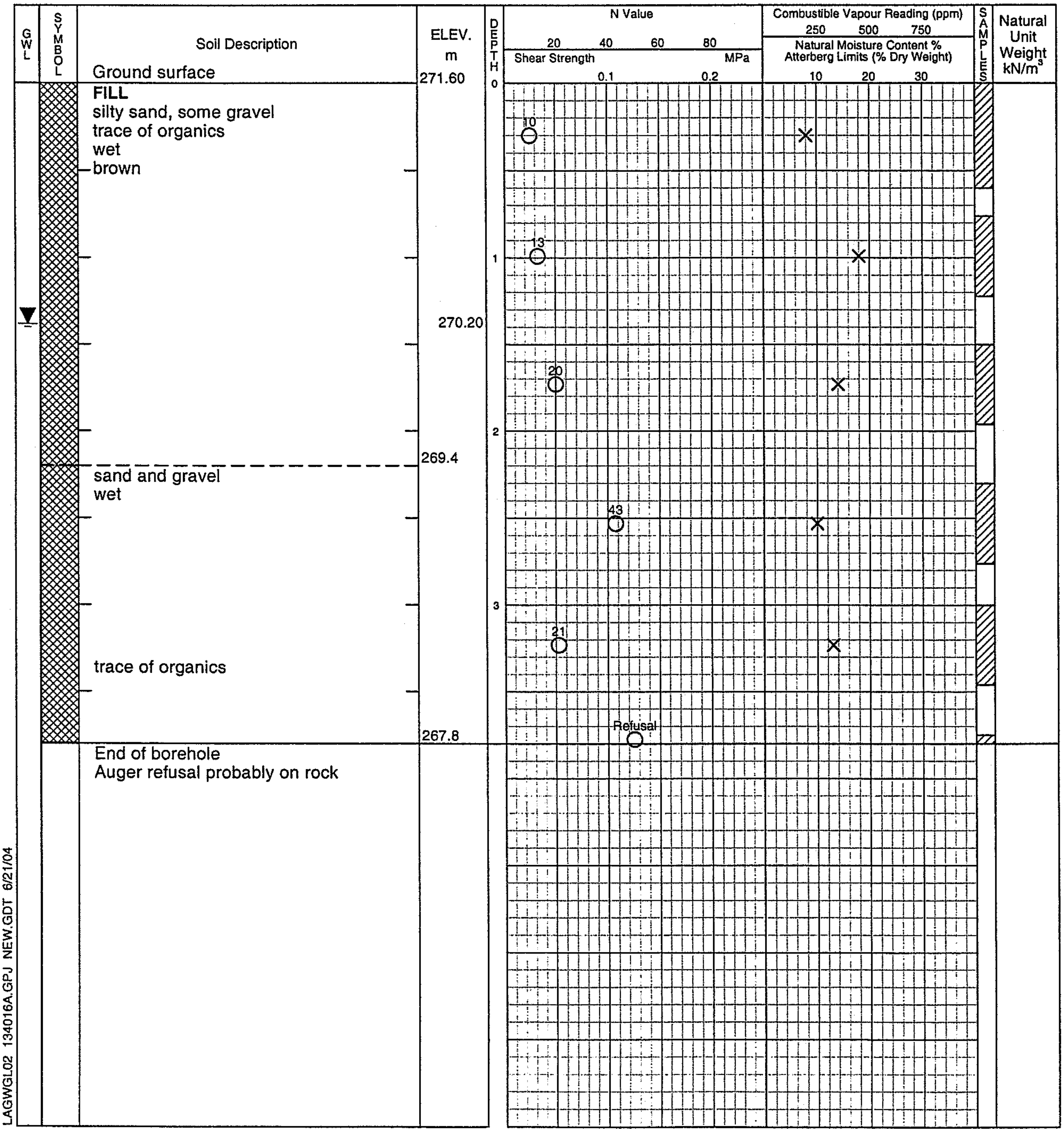
				JOINT CHARACTERISTICS														
ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	NO. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE (mm)	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NUMBER	RECOVERY (%)	RQD	WATER RECOVERY (%)	WATER COLOUR
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
268.6	3	▨	**BEDROCK** greywacke fresh gray very strong 1 fracture in 1.2 m Uniaxial compressive strength at 3.5 to 3.8 m = 276.0 MPa	1	C	F	W	RP	T					1	97	97	100	Gray
267.4	4	▨	3 fractures in 1 m thin clay film on surface of steep fractures	1 1 1	C C C	V F V	M M M	RP RP RP	NC T NC	<1 <1				2	100	70	100	Gray
266.4	5	▨	no fracture											3	100	91	100	Gray
265.5	6 7 8		End of Borehole at 5.8 m															

Log of Borehole 4

Project No. brge00134016a
Drawing No. 7
Project: Shabomeka Lake Dam
Sheet No. 1 of 1
Location: Region of Frontenac, Ontario

Date Drilled: 07.05.2004
Drill Type: Solid auger
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



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Trow

Time	Water Level (m)	Depth to Cave (m)
Completion	1.7	2
5 pm 07.05.2004	1.4	1.7
09.05.2004	1.4	1.7

Log of Borehole 5

Project No. brge00134016a

Drawing No. 8

Project: Shabomeka Lake Dam

Sheet No. 1 of 1

Location: Region of Frontenac, Ontario

Date Drilled: 07.05.2004

Drill Type: Solid auger

Datum: Geodetic

Auger Sample ☒

SPT (N) Value ○ ▨

Dynamic Cone Test —

Shelby Tube ■

Field Vane Test ✚ S

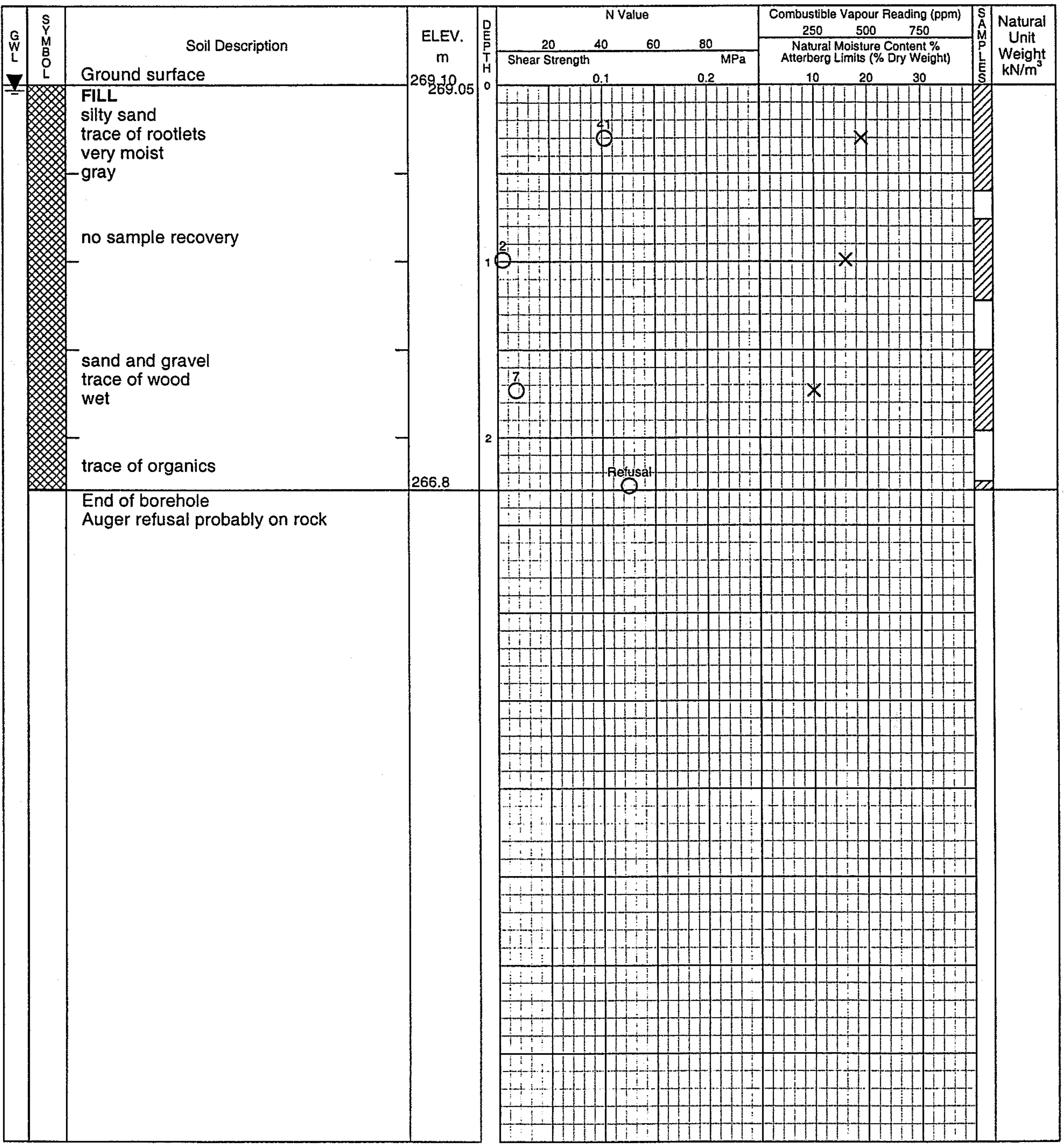
Combustible Vapour Reading ☐

Natural Moisture ×

Plastic and Liquid Limit ├─○

Undrained Triaxial at % Strain at Failure ⊕

Penetrometer ▲



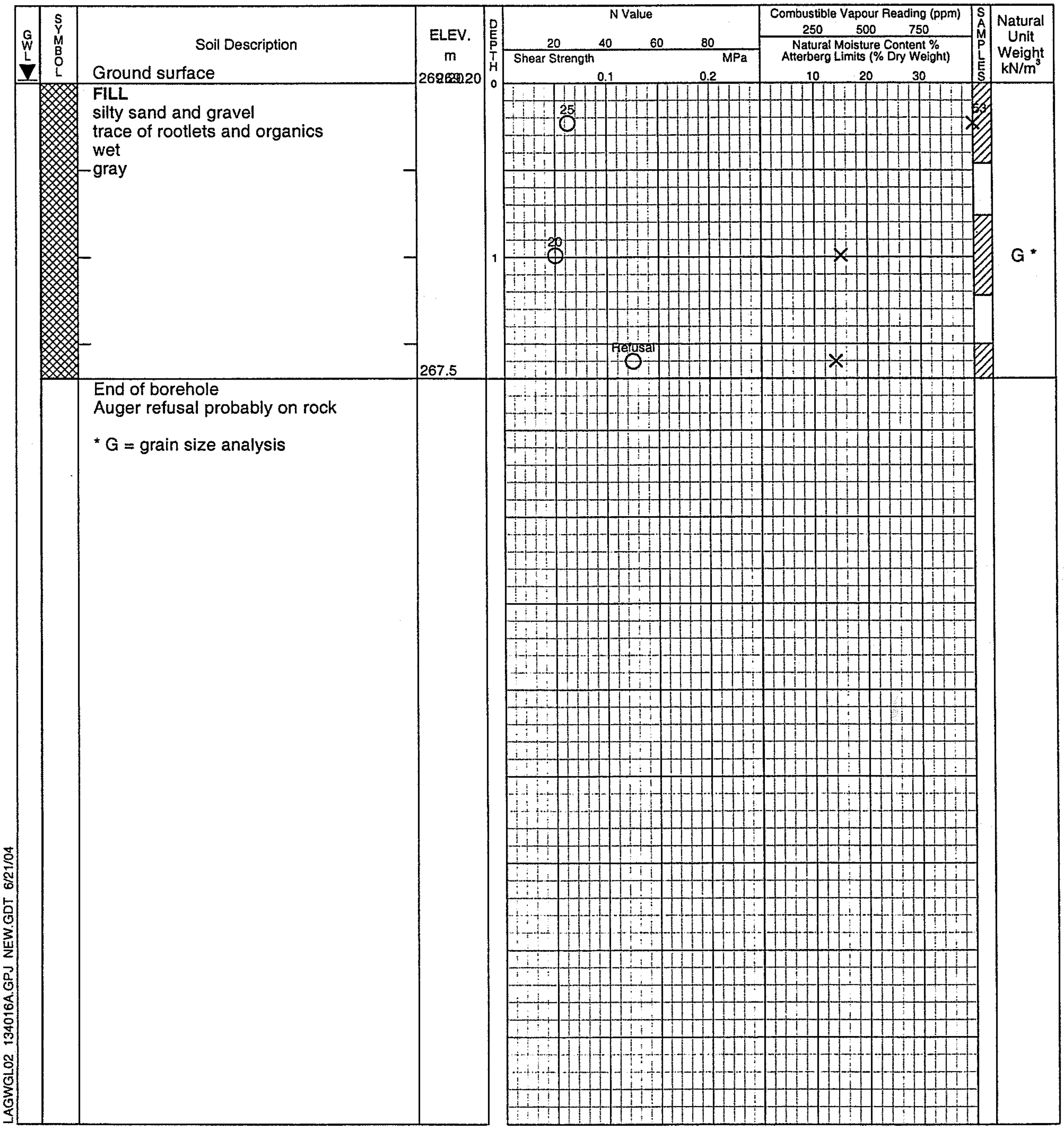
Trow

Time	Water Level (m)	Depth to Cave (m)
Completion	0.5	0.6
5 pm 07.05.2004	0.2	0.5
09.05.2004	0.05	0.5

Log of Borehole 6

Project No.	brge00134016a	Drawing No.	9
Project:	Shabomeka Lake Dam	Sheet No.	1 of 1
Location:	Region of Frontenac, Ontario		
Date Drilled:	07.05.2004		
Drill Type:	Solid auger		
Datum:	Geodetic		

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲



Ground surface 26926920

FILL
silty sand and gravel
trace of rootlets and organics
wet
gray

267.5

End of borehole
Auger refusal probably on rock

* G = grain size analysis

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Trow

Time	Water Level (m)	Depth to Cave (m)
Completion	0.1	0.2
5 pm 07.05.2004	0	0.1
09.05.2004	0	0.1

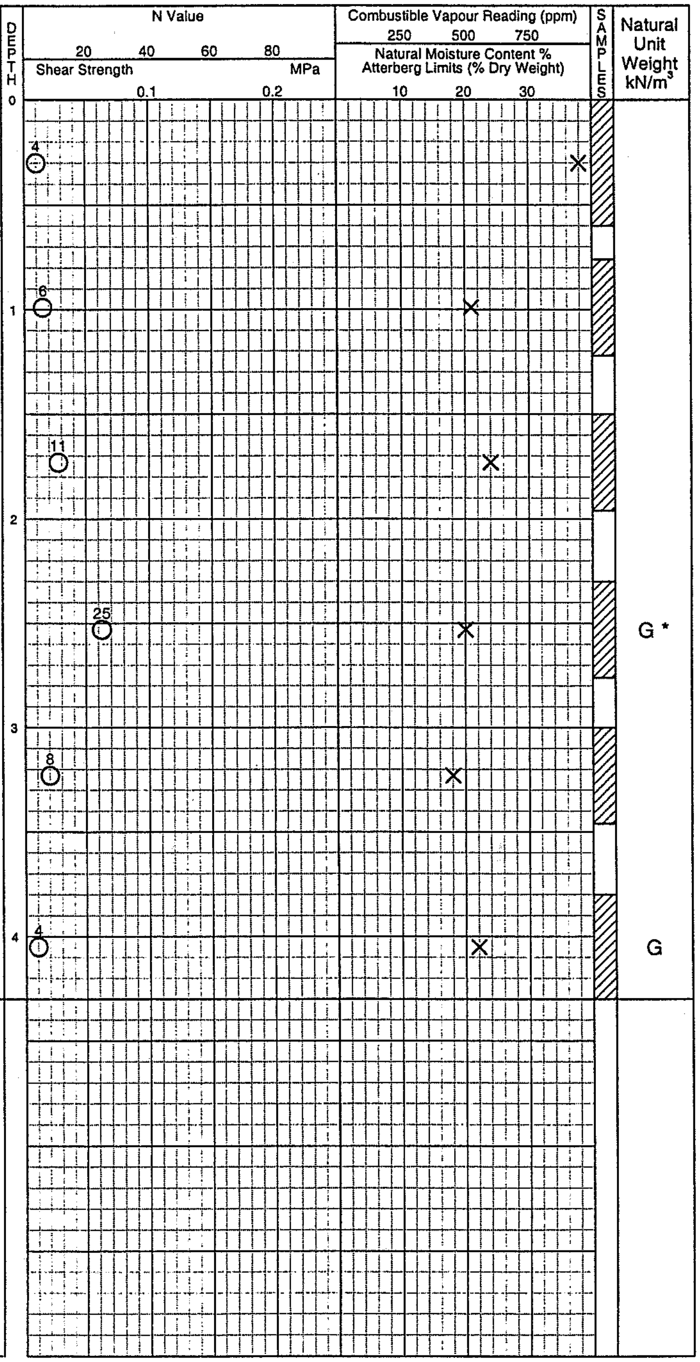
Log of Borehole 7

Project No. brge00134016a
Drawing No. 10
Project: Shabomeka Lake Dam
Sheet No. 1 of 1
Location: Region of Frontenac, Ontario

Date Drilled: 06.05.2004
Drill Type: Solid auger
Datum: Geodetic

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	⊢—⊖
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
		Ground surface	271.40
▼	▩	**FILL** silty sand and gravel organic seam (peaty) very moist	271.00
	▩	mixture of silt, sand and gravel wet, brown	270.7
	▩	silt to clayey silt trace vegetation wet brown	270.0
	▥	**SILT** trace to some sand very moist to wet gray compact to loose	269.3
	▥	wet	
		End of borehole Auger refusal probably on rock * G = grain size analysis	267.1



LAGWGL02 134016A.GPJ NEW.GDT 6/21/04

Trow

Time	Water Level (m)	Depth to Cave (m)
Completion	0.7	2.4
5 pm 07.05.2004	0.4	1.1
09.05.2004	0.4	1.0

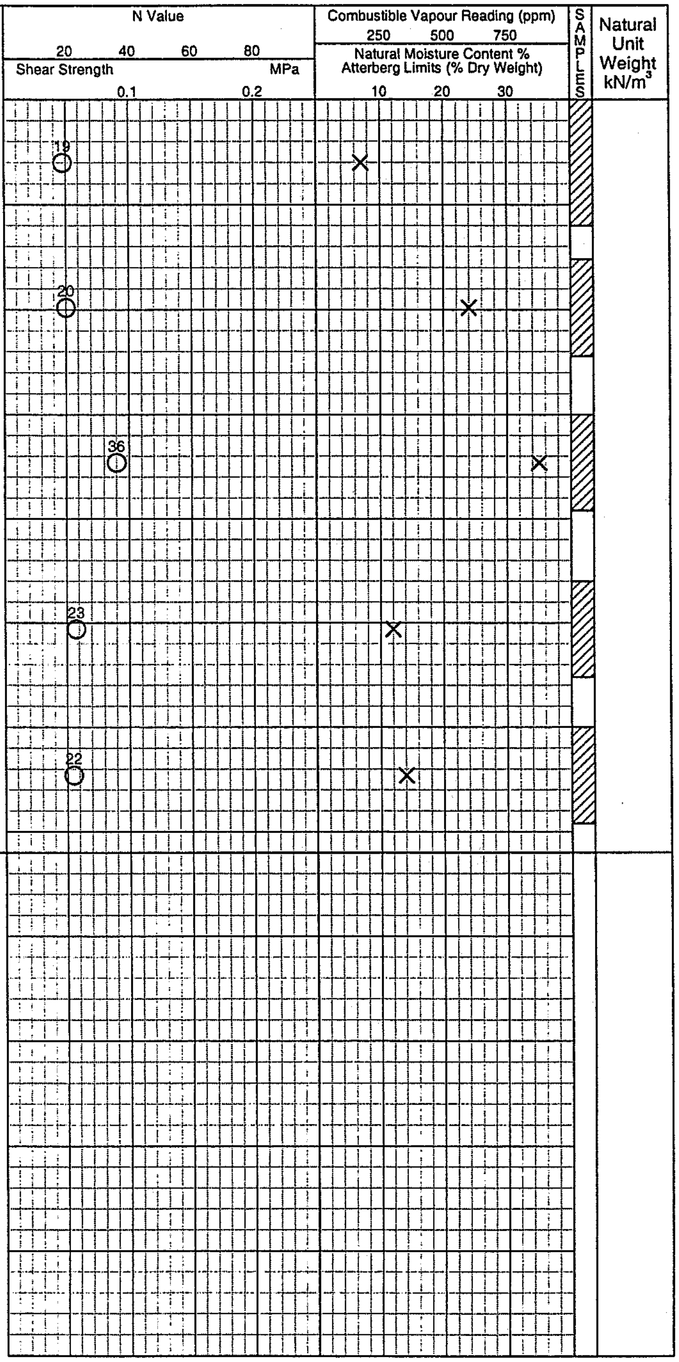
Log of Borehole 8

Project No. brge00134016a
Drawing No. 11
Project: Shabomeka Lake Dam
Sheet No. 1 of 1
Location: Region of Frontenac, Ontario

Date Drilled: 07.05.2004
Drill Type: Solid auger
Datum: Geodetic

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├─○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH
		Ground surface	271.10	0
		FILL sand, some gravel moist, brown		
▼			270.70	
			270.4	
		SILTY CLAY moist gray very stiff to hard		1
		very moist		
			269.0	2
		SAND AND GRAVEL some silt wet brown compact		3
			267.5	
		End of borehole Auger refusal probably on rock		

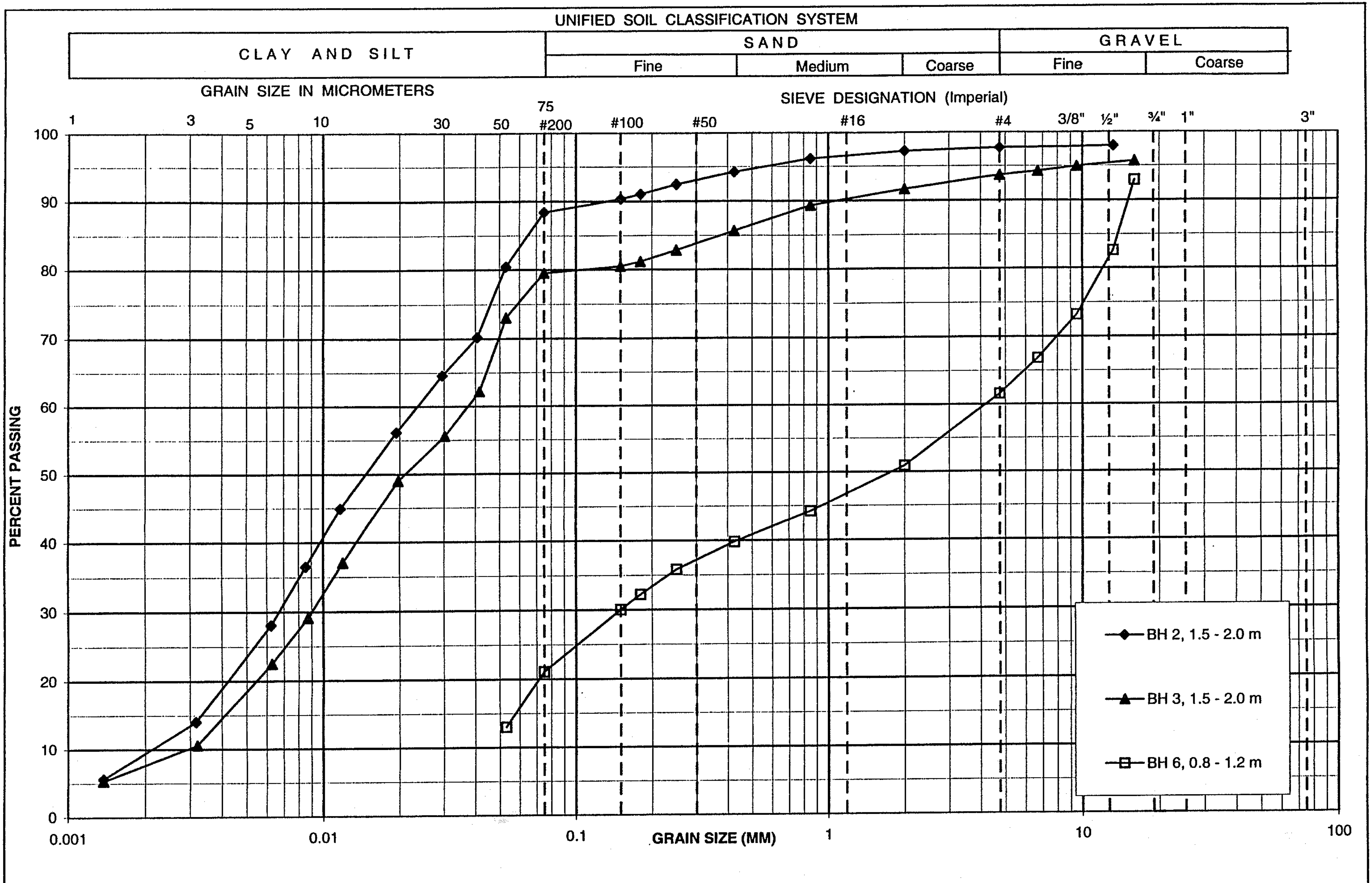


LAGWGL02 134016A.GPJ NEW.GDT 6/21/04

Time	Water Level (m)	Depth to Cave (m)
Completion	1	1.5
5 pm 07.05.2004	0.5	1.2
09.05.2004	0.4	

Trow

Appendix B:
Figures



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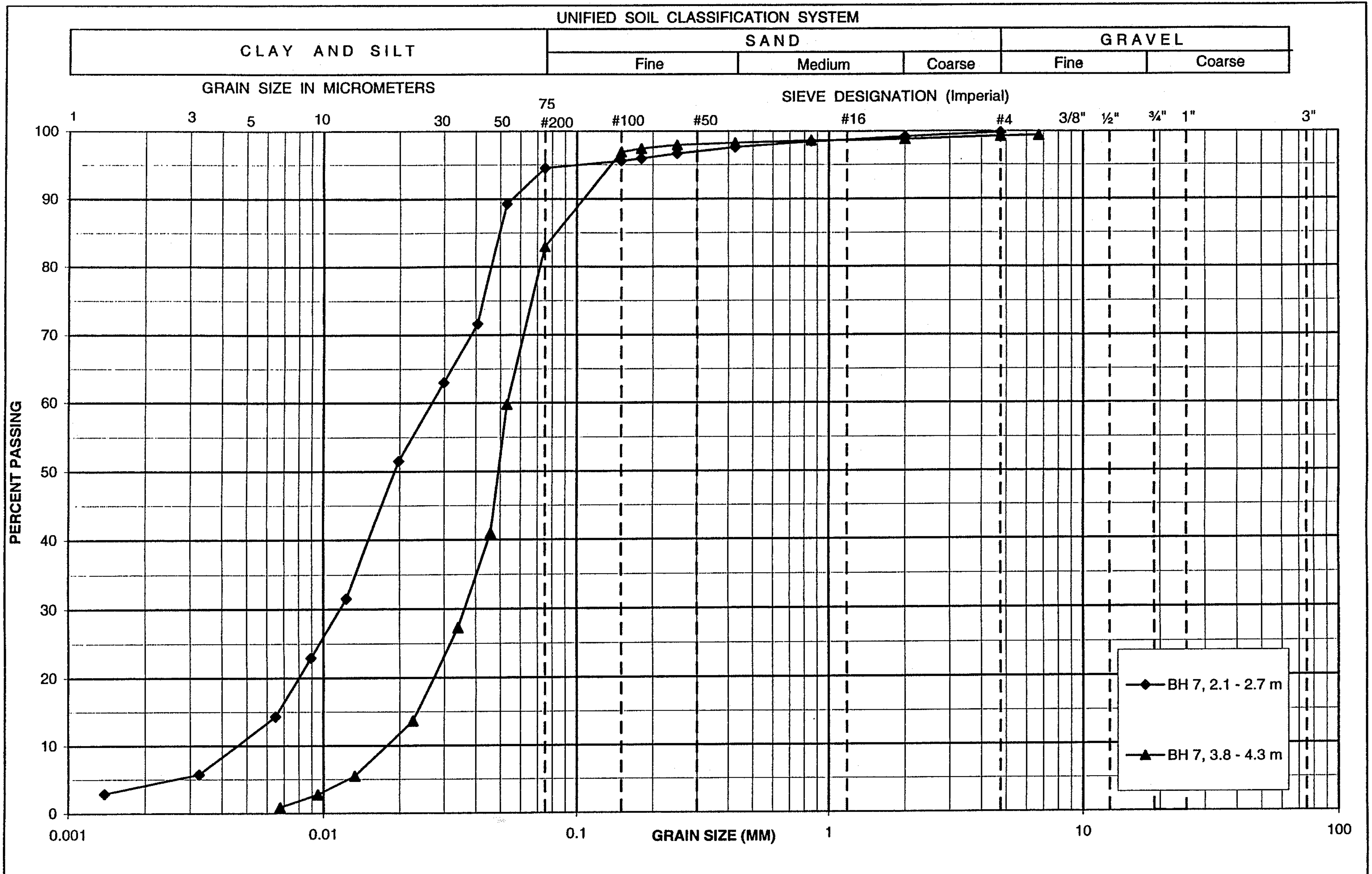
GRAIN SIZE DISTRIBUTION
SILT FILL, SILTY SAND & GRAVEL FILL

FIGURE No.	1
REF. No.	brge00134016a
DATE	June 2004



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GRAIN SIZE DISTRIBUTION
SILT

FIGURE No. 2
REF. No. brge00134016a
DATE June 2004

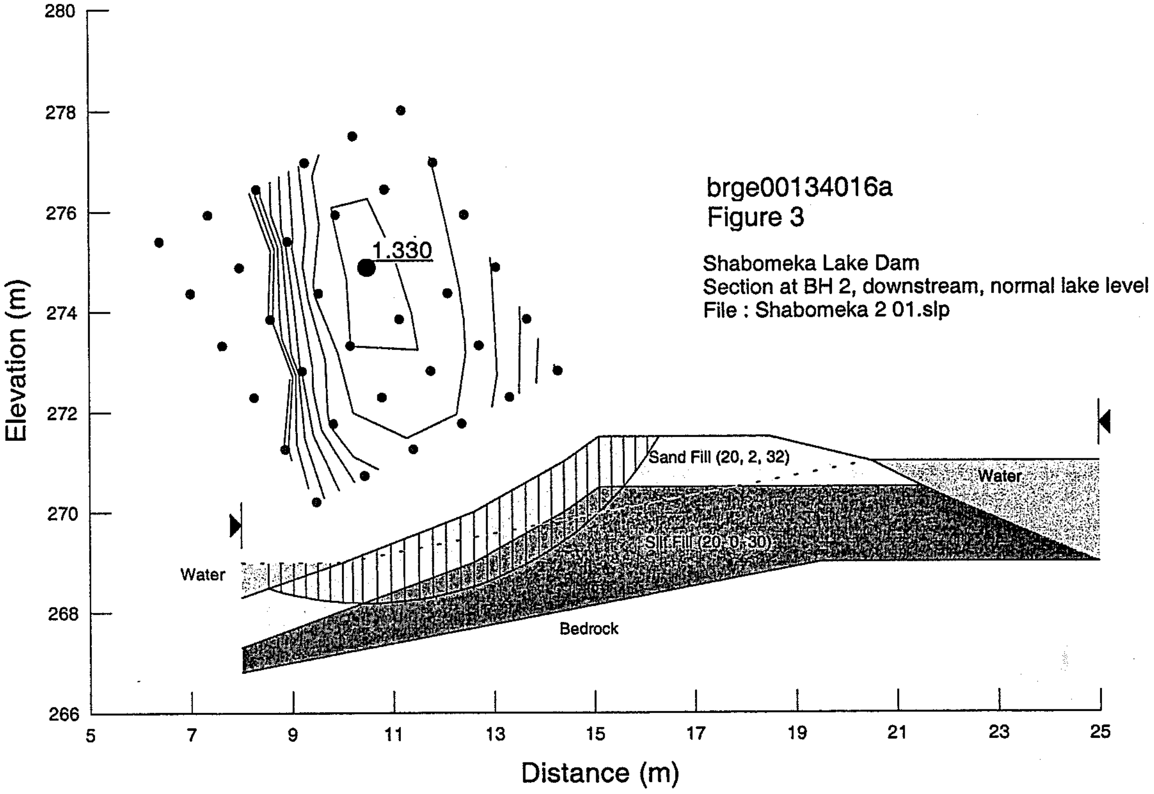

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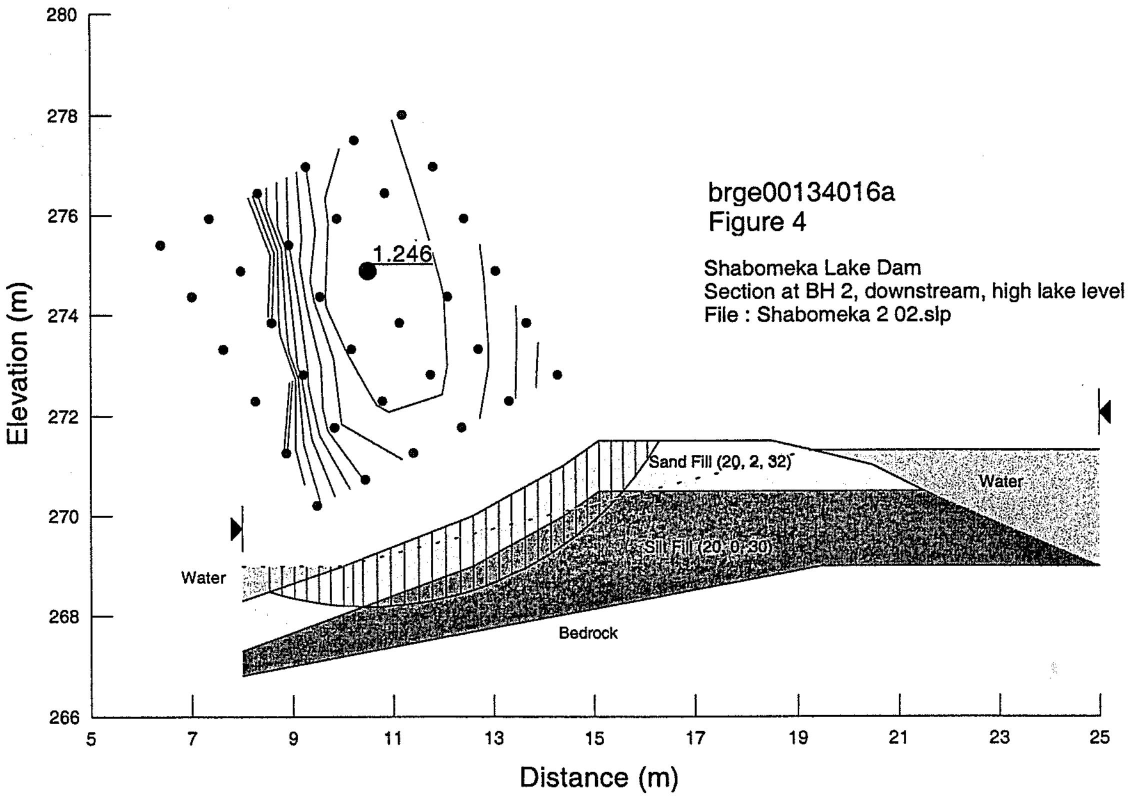
Figure 3

Shabomeka Lake Dam
Section at BH 2, downstream, normal lake level
File : Shabomeka 2 01.slp



brge00134016a
Figure 4

Shabomeka Lake Dam
Section at BH 2, downstream, high lake level
File : Shabomeka 2 02.slp

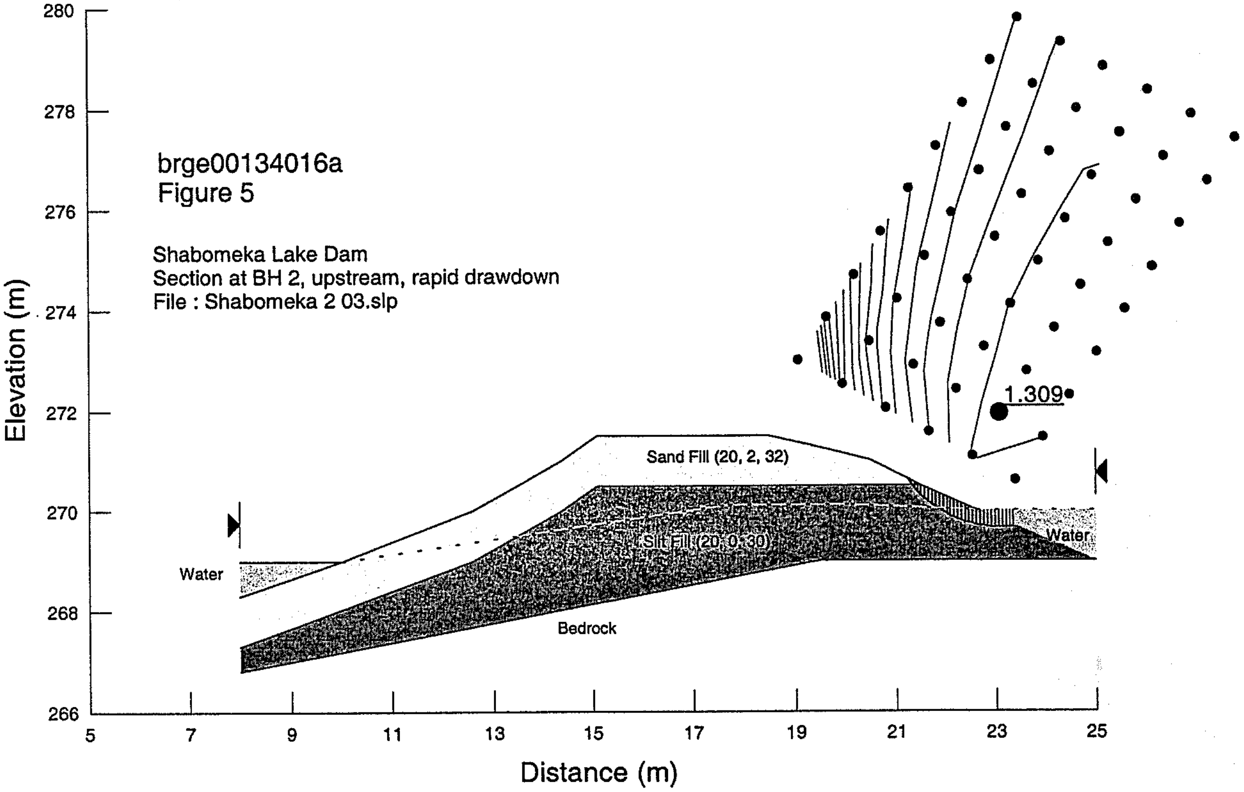


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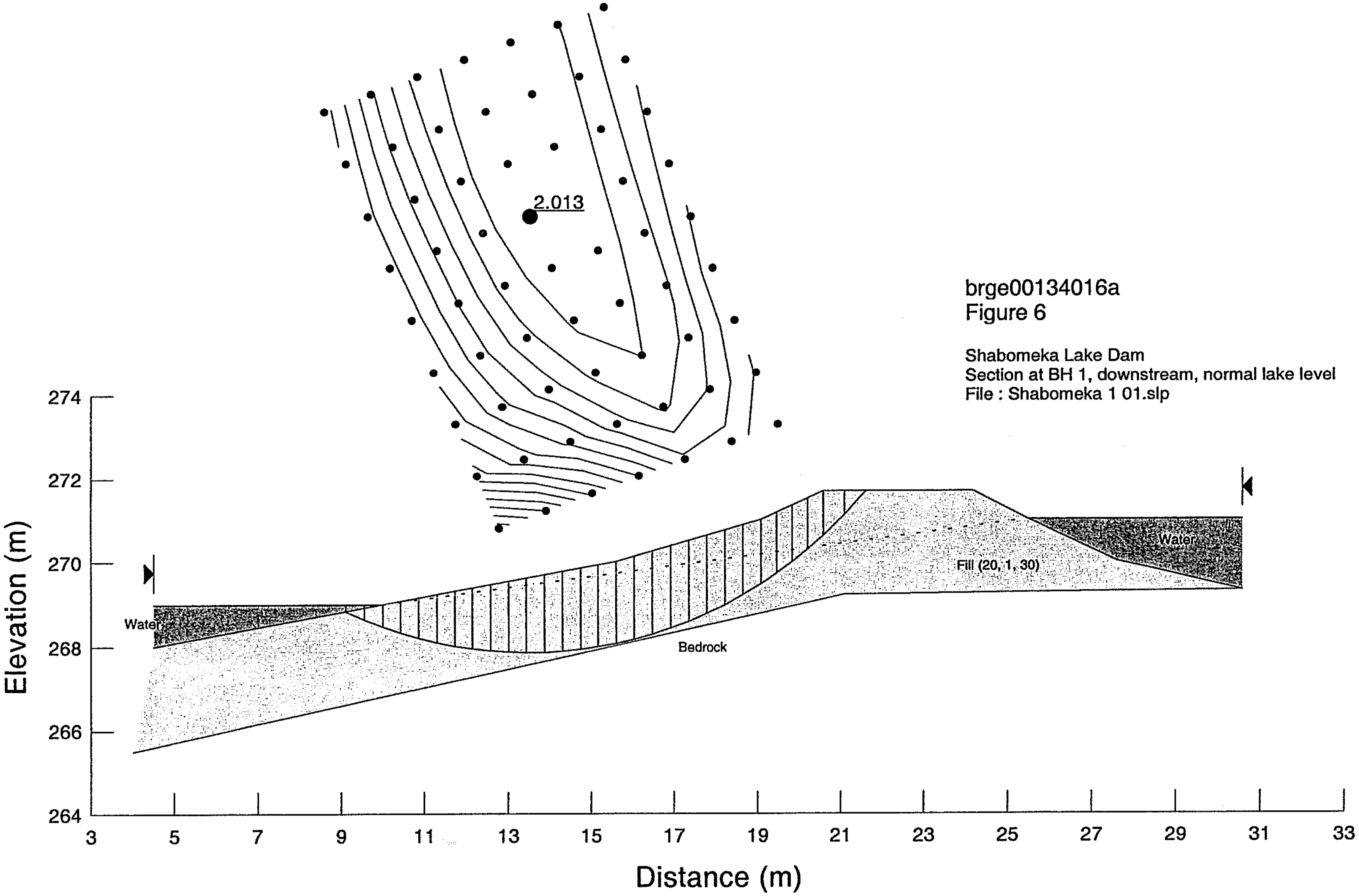
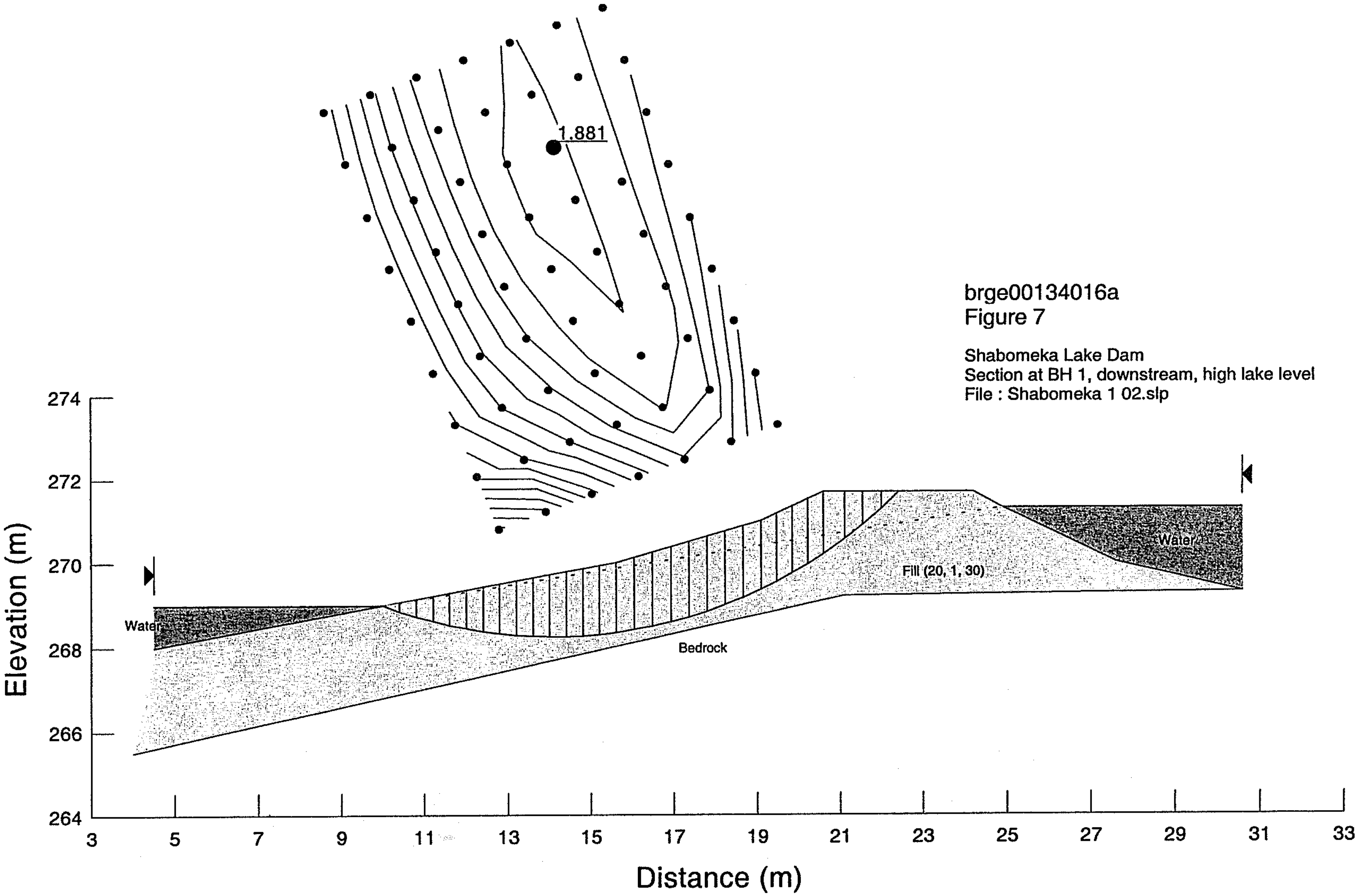
Figure 5

Shabomeka Lake Dam
Section at BH 2, upstream, rapid drawdown
File : Shabomeka 2 03.slp



brge00134016a
Figure 6

Shabomeka Lake Dam
Section at BH 1, downstream, normal lake level
File : Shabomeka 1 01.slp

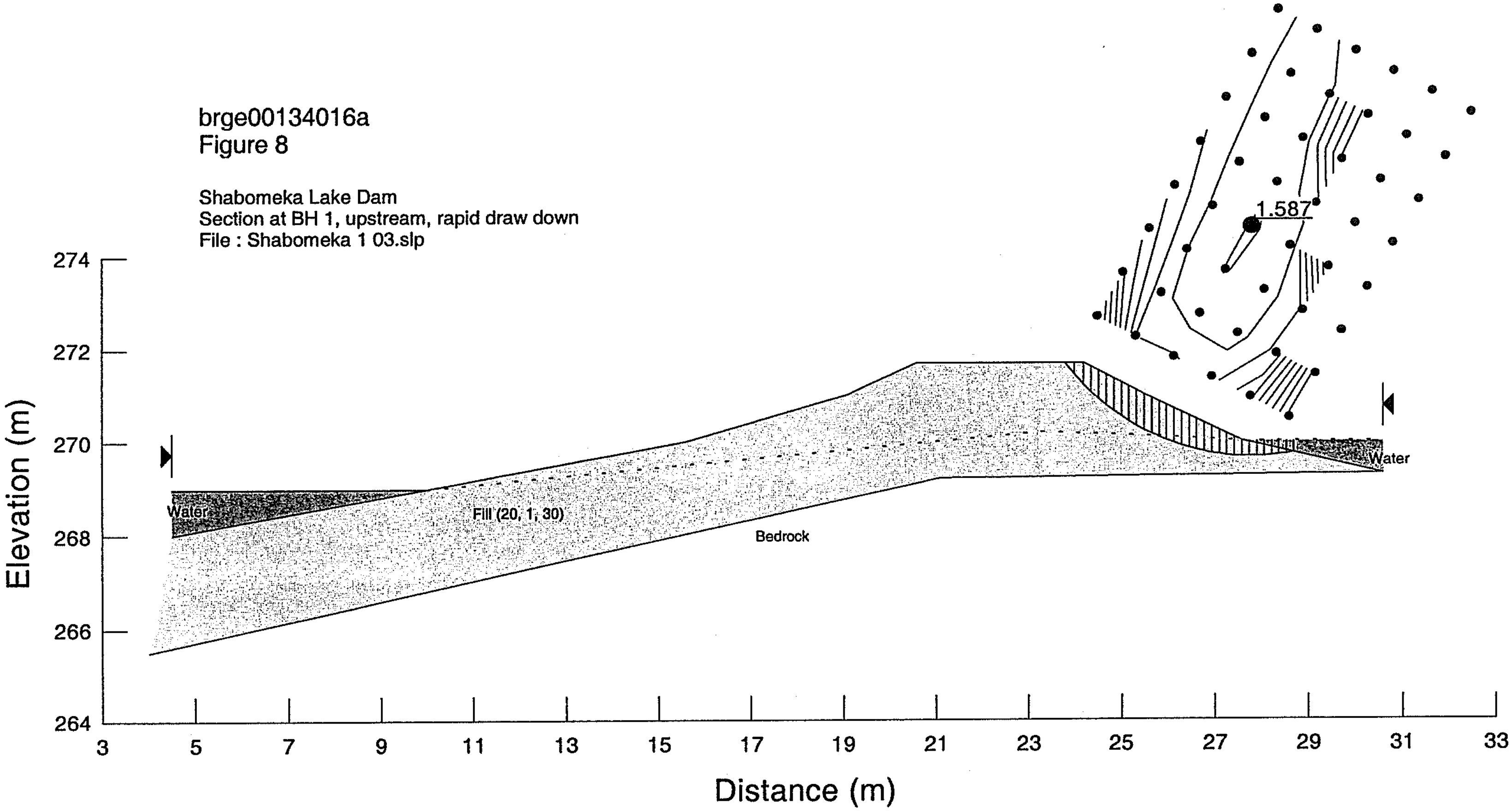


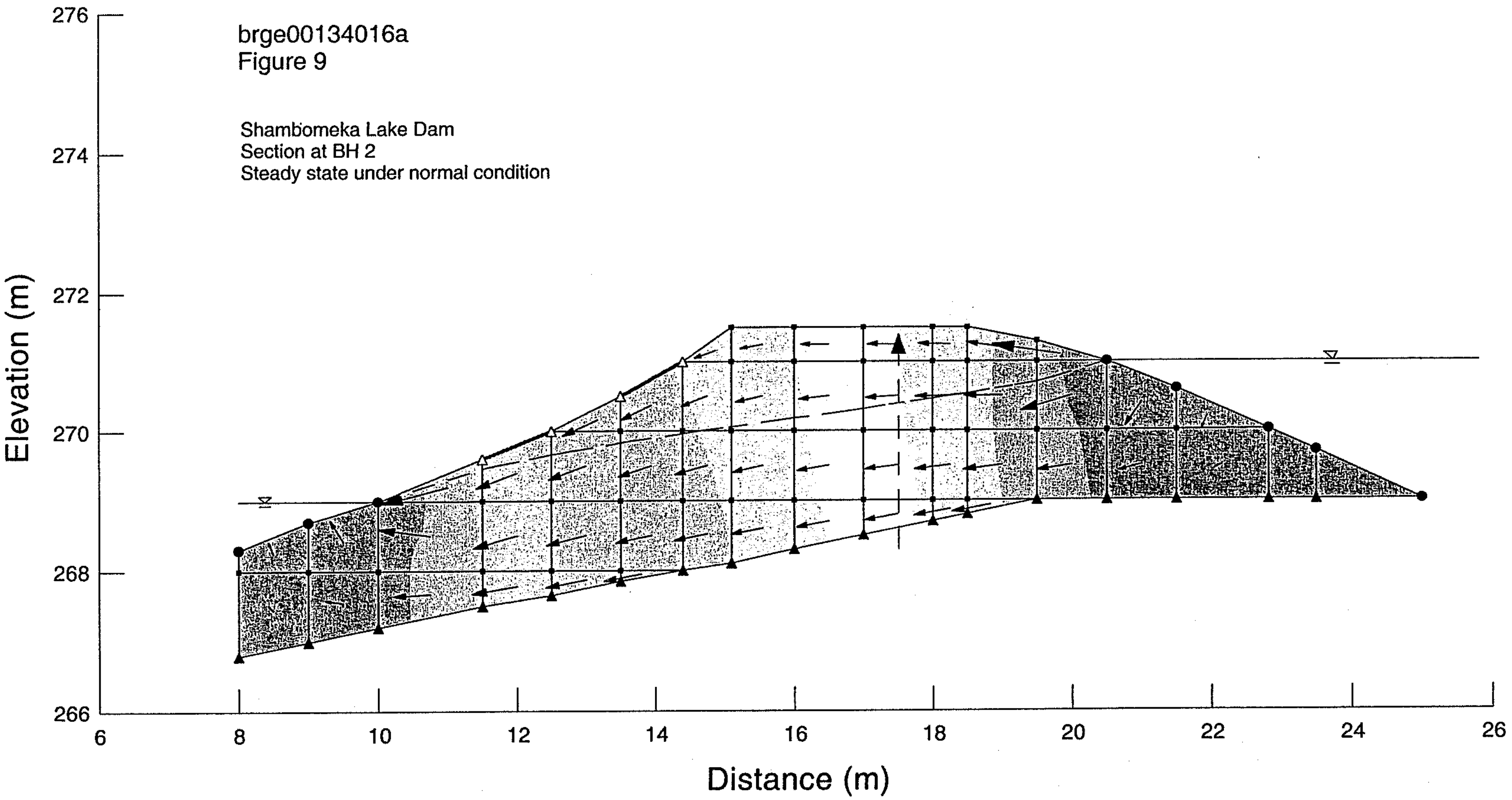
brge00134016a
Figure 7

Shabomeka Lake Dam
Section at BH 1, downstream, high lake level
File : Shabomeka 1 02.slp

brge00134016a
Figure 8

Shabomeka Lake Dam
Section at BH 1, upstream, rapid draw down
File : Shabomeka 1 03.slp





brge00134016a
Figure 9
Shambomeka Lake Dam
Section at BH 2
Steady state under normal condition
Elevation (m)
276
274
272
270
268
266
Distance (m)
6
8
10
12
14
16
18
20
22
24
26

Appendix C:
Chemical Tests Results

Client: Trow Brampton
Attention: James Ng
Project: BRGE00134016A
P.O.:
Sample Type: Soil
Date Received: May 11/04
Date Analysed: May 11/04
Date Reported: May 17/04

ENTECH
A Division of Agri-Service Lab Inc.
6820 Kitimat Rd., Unit #4
Mississauga, ONT L5N 5M3
TEL: (905) 821-1112
FAX: (905) 821-2095

Certificate of Analysis

Data Pertain To Specific Sample(s) Tested

PARAMETER	Method Detection Limit (µg/g)	CONTROL SAMPLE			SAMPLE DATA (µg/g)				
		Expected Conc. (µg/mL)	Found Conc. (µg/mL)	Recovery %	Blank	33095 BH2 SS2	33096 BH3 SS2	33096 BH3 SS2 Duplicate	
Sulphate (µg/g)	0.5	14.5	13.61	94	<0.5	4.7	2.6	2.7	
pH (units)	-	7.41	7.37	99	-	8.1	8.3	8.3	

Sample Disposal: 30 Days from the Reporting Date.
Analyst(s): SJ, PS
Method:
Sulphate - Extraction/Ion Chromatography (EPA 300.0)
pH - Extraction/Electrometric (EPA 9045)

Sanyal

Sam Sanyal, M.Sc., C. Chem.
Manager, Inorganic Analysis.

ENTECH
A Division of Agri-Service Lab Inc.
6820 Kitimat Rd., Unit #4
Mississauga, ONT L5N 5M3
TEL: (905) 821-1112
FAX: (905) 821-2095

Client: **Trow Brampton**
Attention: **James Ng**
Project: **BRGE00134016A**
P.O.:
Sample Type: **Water**
Date Sampled: **May 07/04**
Date Received: **May 11/04**
Date Analysed: **May 11 & May 12/04**
Date Reported: **May 17/04**

Certificate of Analysis

Data Pertain To Specific Sample(s) Tested

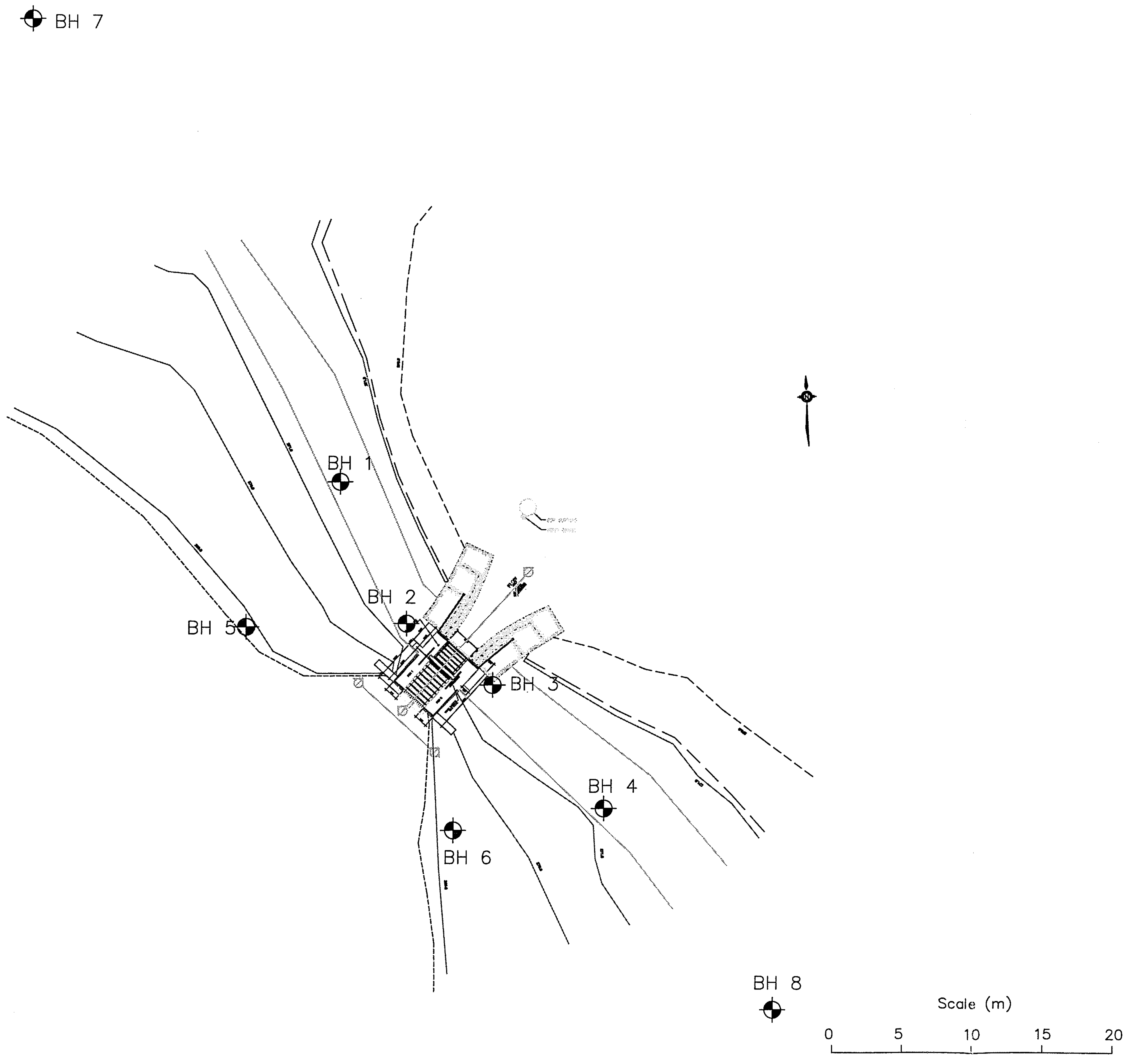
PARAMETER	**Method Detection Limit (µg/mL)**	**CONTROL SAMPLE**			**SAMPLE DATA (µg/mL)**			
		Expected Conc. (µg/mL)	**Found Conc. (µg/mL)**	**Recovery %**	**Blank**	**33097 No. 1**	**33098 No. 2**	**33098 No. 2 Duplicate**
Sulphate (µg/mL)	0.1	14.5	13.61	94	<0.1	5.3	5.2	5.3
pH (units)	-	7.41	7.37	99	-	7.4	7.7	7.7

Sample Disposal: 30 Days from the Reporting Date.
Analyst(s): PS, SJ
Method:
Anions - Ion Chromatography (EPA 300.0)
pH - Electrometric/pH-Meter (EPA 150.1)

Sanyal

Sam Sanyal, M.Sc., C. Chem.
Manager, Inorganic Analysis.

Appendix D:
Drawings



GENERAL NOTES:

[illegible]

No.	REVISION	DATE	APPD.
A	RELEASED FOR INFORMATION	JAN 28 02	

SEAL

Ministry of
Natural
Resources
Ontario

PROJECT
SHABOMEKA LAKE DAM

CLIENT
MISSISSIPPI VALLEY CONSERVATION

PROJECT NO.
brge00134016a

MVC FILE NO.

DRAWING TITLE
SITE PLAN

SCALE
1:300

DATE
MAY 2004

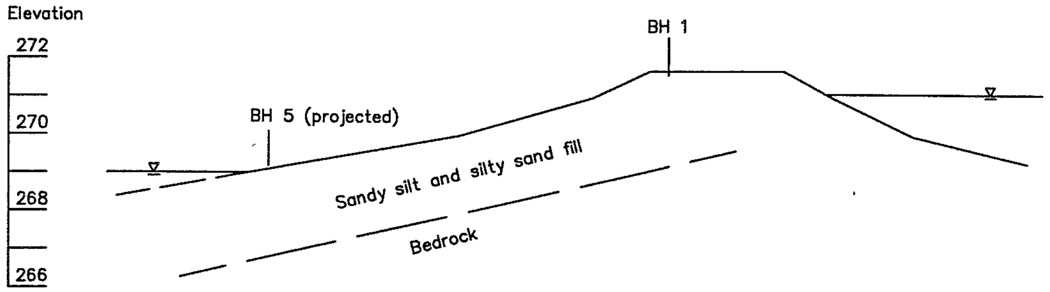
DRAWN BY
M.S.A.

REV. NO.
A

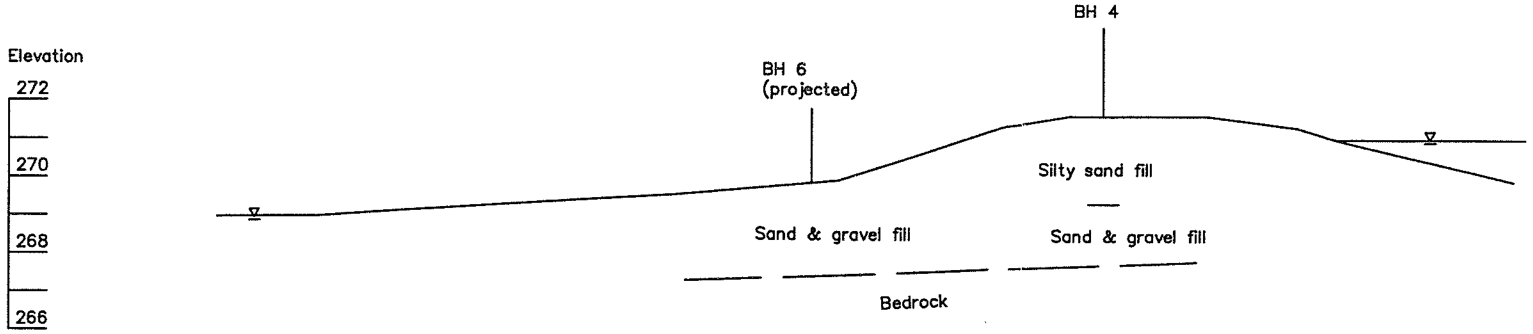
DESIGNED BY

DRAWING NO.
1

CHECKED BY
M.H.N.



SECTION AT BH 1



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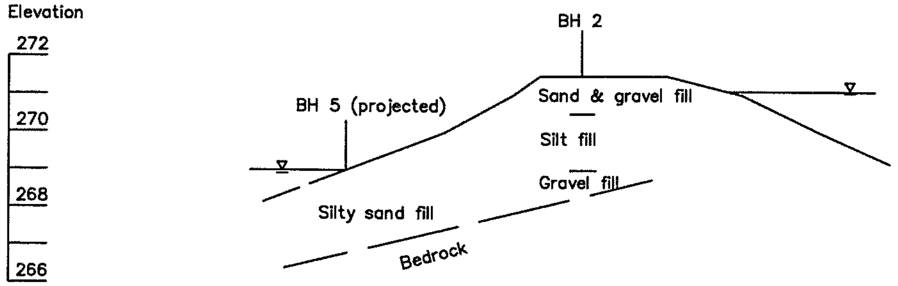

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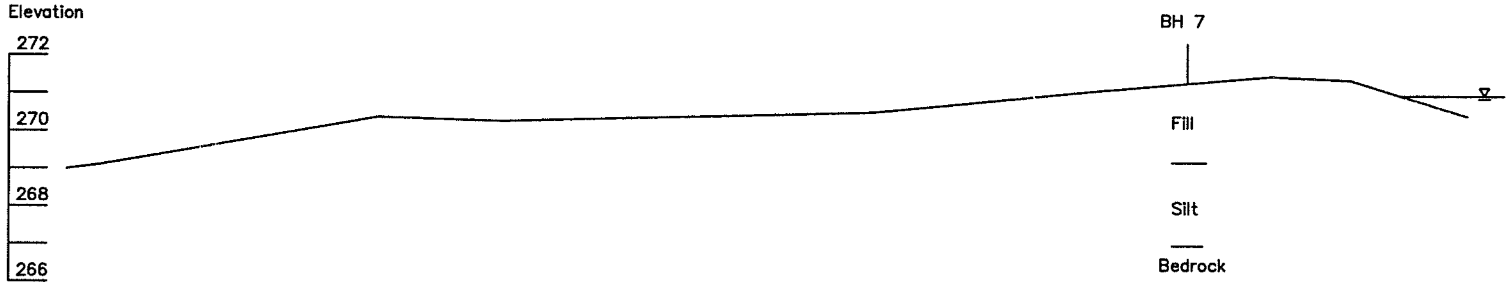

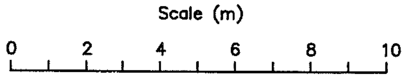

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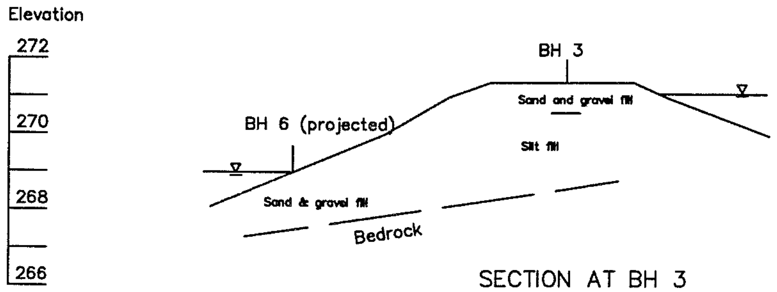
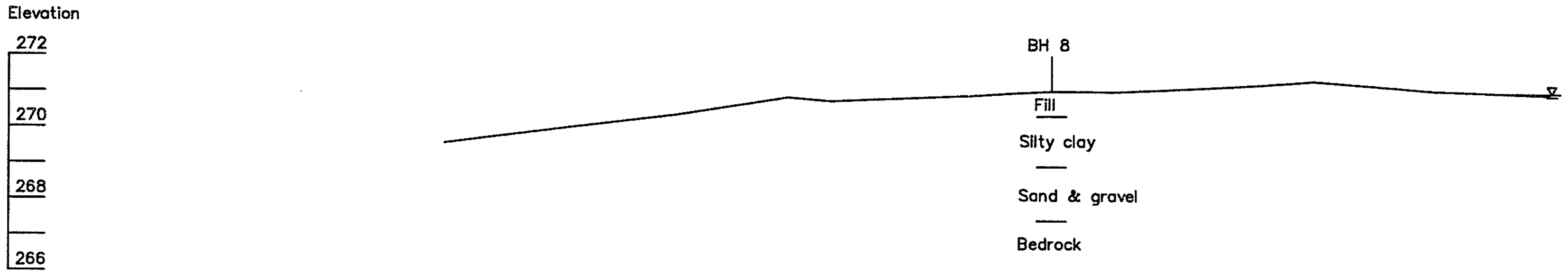


SECTION AT BH 3



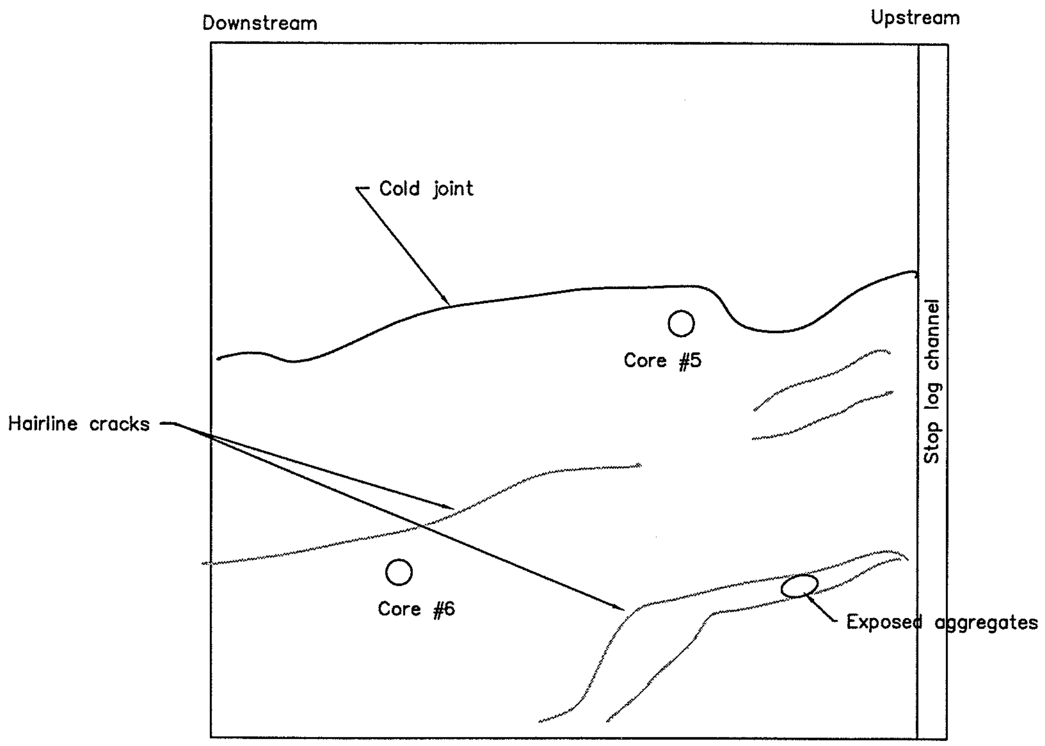
SECTION AT BH 8

Trow
Trow Associates Inc.

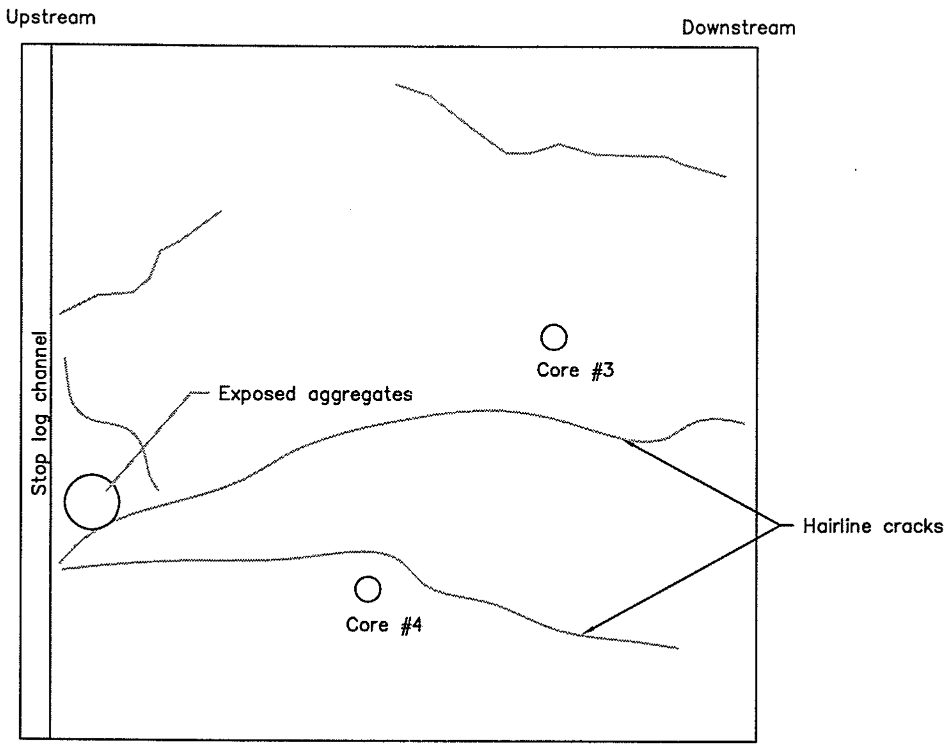
SHAMBOMEKA LAKE DAM
CROSS SECTIONS OF EMBANKMENT AT BOREHOLE LOCATIONS

Scale: 1:200

Project No. brge00134016a
Drawing No. 2
Date: June 2004

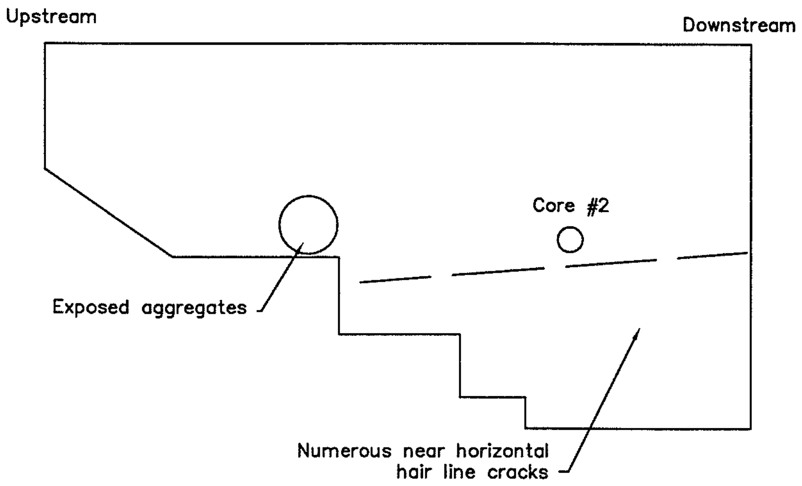


NORTH WEST WALL INTERIOR



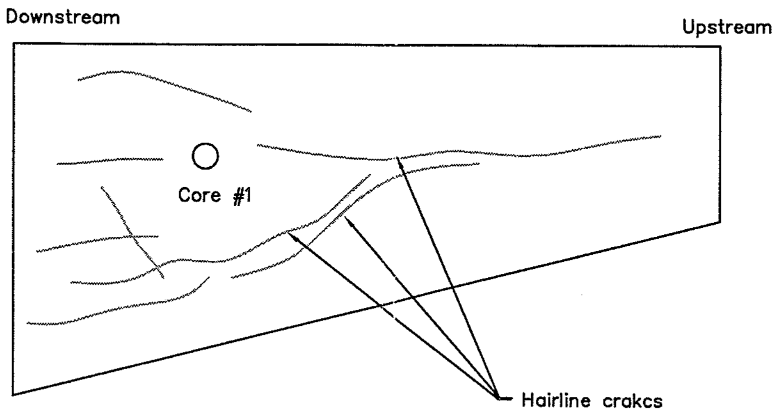
SOUTH EAST WALL INTERIOR

Ups



:racks

NORTH WEST WALL EXTERIOR



SOUTH EAST WALL EXTERIOR

Trow Associates Inc.

CONTROL STRUCTURE DEFECTS AND CORE LOCATIONS

Scale: 1:30

Project No. brge00134016a
Drawing No. 3
Date: June 2004

Shabomeka Lake Dam Rehabilitation Design Report

Mississippi Valley Conservation Authority
June 2021
Project: 159100826

Stantec