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Public Works and
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Canada

Parks Canada

AECOM

Project No.: R.051116.002

December 2013

Dam Safety Reviews: Port Severn Dams Main Dam, Lock 45, Upstream Shoreline Wall and Dam D

Dam Safety Review Report

Final

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December 12th, 2013

Shawn Filion, P.Eng.
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111 Water Street East
Cornwall, ON K6H 6S3

Dear Mr. Filion:

Project No: **60245720**

Regarding: **Dam Safety Reviews: Port Severn Dams**
Phase II – Dam Safety Review
Main Dam, Lock 45, Upstream Shoreline Wall and Dam D – Final Report
PWGSC Project Number R.051116.002
Standing Offer No. EQ715-101279/001/PWL

AECOM Canada Ltd. is pleased to provide Public Works and Government Services Canada (PWGSC) with six (6) hard copies and four (4) CD-ROMs of our Dam Safety Review Report for Main Dam, Lock 45, Upstream Shoreline Wall and Dam D at Port Severn.

The Phase II study was conducted in general conformance with Section RS2 Dam Safety Review of the Project Brief dated August 25, 2011 and our Phase II Project Proposal dated October 20, 2011.

The key tasks undertaken in completing this Dam Safety Review Report included the visual inspections and site investigations, stability analyses, classification of the dam in conformance with the PCA Directive for Dam Safety Program, determination of the appropriate Inflow Design Flood, evaluation of the hydraulic capacity of the control structure, review of the operational procedures, completion of an operator and public safety assessment, documentation of the results of our investigations and analyses, and recommendations for required improvements/action.

The report also summarizes available information, results of geotechnical and concrete field investigations and laboratory testing conducted to provide information required for the completion of this Dam Safety Review.

We express our appreciation to PWGSC and Parks Canada staff for providing valuable input and assistance throughout the course of the Phase II study. We are available to elaborate on any aspect of the report, and to assist in the implementation of the recommendations, at your request.

Sincerely,

AECOM Canada Ltd.

Annie Dumas, P.Eng.
Project Manager
annie.dumas@aecom.com

AD:md
Encl.
cc: Jacques Béland

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6	Yes	Public Works and Government Services Canada
3	Yes	AECOM Consultants Inc.

Revision Log

Revision #	Revised By	Date	Issue / Revision Description

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Executive Summary

AECOM has been mandated by PWGSC to undertake a Dam Safety Review for the Main Dam at Port Severn, Ontario. This dam is operated to maintain navigational water levels in the Gloucester Pool for Lock 45, the last lock on the Trent Severn Waterway. The Maximum Normal Operating Level (MNOL) of the reservoir is 180.50 m.

The Dam Safety Review consisted of inspections and review, dam break analyses, dam classification, stability analysis, public and operator safety, operation review and flood mapping. The site inspection was carried out in December 2011. As a result of the Dam Safety Review, Main Dam has been determined to have a **Significant** hazard classification. The selected design basis earthquake (DBE) is the 500-year earthquake. The selected inflow design flood (IDF) is the 1,000-year flood, which has a maximum water level of 181.35 m in Gloucester Pool. The water level under IDF conditions can be reduced to 180.57 m by operating all sluices at Main Dam, Dam E and Dam G. This is recommended to reduce flood damage in the reservoir, yet is currently difficult to attain given the complications with removing the bottom logs at Main Dam.

Currently, Port Severn Main Dam does not meet the regulatory requirements for the following reasons:

- The dam was shown to be unstable under ice loading scenarios only. It is structurally sound under all other loading scenarios.
- While Port Severn Main Dam can theoretically be operated to discharge the IDF, reliability in operation of certain sluices poses a risk.
- Improvements should be made to provide a safer working environment.

Deficiencies were identified and measures required to remediate the issues were recommended. These recommendations were prioritized into short, mid and long term time horizons and their costs were estimated. Preliminary costs for the recommendations amount to approximately $ 2,003,400 including engineering design, construction management and contingency.

The dam was shown to be unstable under ice loading conditions and using certain criteria based on the CDA 2007. It is recommended to install post-tensioned anchors to stabilize the dam. Until this is done, maintenance crews should break the forming ice cover on the entire length of the upstream face of the dam to reduce loading. Water agitators could also be considered, yet are quite expensive for a temporary solution.

AECOM has determined the dam can be operated to safely discharge the IDF flood, though it would cause upstream flooding. Constraints and deficiencies in operation were identified regarding the operation at the Main Dam. These cover a range of aspects of the dam operation and recommendations are made to improve operation capabilities and reduce risk of upstream flooding. In short, all stoplogs and lifting devices must be maintained operable at all times. In addition, staff must be fully trained in operation of stoplogs under fair and adverse conditions. While the hydraulic log lifter is in an acceptable working condition, it is recommended to replace it with a newer model as its inability to compress stoplogs forces the operators to perform hydraulic jacking, a hazardous operation.

At the maximum water level under the Main Dam's IDF conditions, water overtops the damaged Dam A, causing flooding inland. It is recommended to mend Dam A and coordinate with the municipal authorities the improved drainage of the lowlands. Associated costs are defined in the condition assessment report of Dam A.

Recommendations are put forth concerning occupational health and safety. These include the need of operators to tie-off to adequate engineered anchors, safe procedures for the use of the hydraulic jack to tighten the stoplogs, additional training on the OHS procedures and use of the equipment, appropriate lighting, upgrade of railings and more. It is also recommended to investigate alternative backup systems as the use of the manual winches is hazardous.

With regard to Public Safety, deficiencies were identified in the review process. These have been outlined in this report and also in a stand-alone Public Safety Review Report. It is recommended that PCA pursue the Risk Management Process and engage in Risk Treatment. Most notably, it is recommended that the configuration at the lock entrance be reviewed as strong currents make it difficult to navigate towards the lock.

Finally, it is recommended that an Emergency Preparedness Plan (EPP), an Emergency Response Plan (ERP) and an Operation, Maintenance and Surveillance (OMS) manual covering all Port Severn Dams be prepared and distributed to the concerned authorities. These will notably serve as reference documents for the authorities to communicate, coordinate, manage and implement the appropriate actions during a hazardous flood event.

Prioritized Recommendations and Cost Estimate

Item	Description	Total Estimated Price		
		Short Term	Medium Term	Long Term
1	**Remedial Works**	**$1,196,800**	**$27,000**	**$0**
1.1	Mobilization and demobilization	$100,000	$10,000	$0
1.2	Structural / Civil	$551,100	$17,000	$0
1.3	Mechanical	$4,500	$0	$0
1.4	Geotechnical	$541,200	$0	$0
2	**Operator Safety**	**$168,000**	**$0**	**$0**
3	**Public Safety**	**TBD**	**$0**	**$0**
4	**Operations**	**TBD**	**$0**	**$0**
5	**Maintenance, Surveillance, Planning**	**TBD**	**$0**	**$0**
6	**Engineering and Construction Management**	**$273,000**	**$5,000**	**$0**
Subtotal		$1,637,800	$32,000	$0
Contingency (20 %)		$327,600	$6,000	$0
Total with contingency (taxes are not included)		**$1,965,400**	**$38,000**	**$0**

TBD To be determined by PCA, through further cost estimations or by requesting proposals

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List of Abbreviations

AECOM	Consultant
AEP	Annual Exceedance Probability
ALARP	As Low As Reasonable Practicable
ASCE	American Society of Civil Engineers
ANCOLD	Australian National Committee on Large Dams
CA	Conservation Authority
CEATI	Center for Energy Advancement through Technical Innovation
CDA	Canadian Dam Association
DEM	Digital Elevation Model
DSR	Dam Safety Review
DV	Depth Velocity
DBE	Design Basis Earthquake
ERP	Emergency Response Plan
EPP	Emergency Preparedness Plan
FERC	Federal Energy Regulatory Commission
GIS	Geographic Information System
GSC	Geological Survey Canada
HPC	Hazard Potential Classification
ICOLD	International Conference on Large Dams
IDF	Inflow Design Flood
MNOL	Maximum Normal Operating Level
MNR	Ministry of Natural Resources
NRC	Natural Resources Canada
LOL	Loss of life
OBC	Ontario Building Code
OMS	Operation, Maintenance and Surveillance
OHS	Occupational Health and Safety
OHSA	Occupational Health and Safety Act
OPG	Ontario Power Generation
PAR	People at Risk
PCA	Parks Canada Agency
PGA	Peak Ground Acceleration
Port Severn Main Dam	Structure comprised of Main Dam, Dam D, Lock 45 and Upstream Shoreline Wall
Port Severn Dams	All water-retaining structures at Port Severn: Main Dam, Dam A, Dam B, Dam B1, Dam C, Dam E (Bayview), Dam F, Dam G (Little Chute) and Little Go Home Bay Dam
PWGSC	Public Works and Government Services Canada
PFD	Personal Flotation Device
PMF	Probable Maximum Flood
PPE	Personal Protective Equipment
PMP	Probable Maximum Precipitation
PMSA	Probable Maximum Snow Accumulation
RGC	Reference Ground Condition
RL	Ratio of wind speed over water to wind speed over land
SSARR	Streamflow Synthesis and Reservoir Regulation Model
SO	Standing Orders
TSW	Trent Severn Waterway
USBR	United States Bureau of Reclamation
UTM	Universal Transverse Mercator

1. Introduction

1.1 Purpose and Objectives

The owner of a dam is responsible for its safe management. Dam safety management is the management of risks associated with dams, including release of water as a result of structural failure, mis-operation, planned operation, or any other cause. Principle 1a of the Canadian Dam Association (CDA) Dam Safety Guidelines 2007 (Reference 8) cites that:

> *The public and environment shall be protected from the effects of dam failure, as well as release of any or all of the retained fluids behind a dam, such that the risks are kept as low as reasonably practicable.*

As such, the completion of initial and periodic dam safety reviews (DSR) are part of the responsible management of a dam. AECOM was mandated by Public Works and Government Services Canada (PWGSC) on behalf of Parks Canada (PCA) to carry out, as required, reviews of dam structure status and classification and dam safety review on the Port Severn Dams, which refers to all dams and water-retaining structures at Port Severn, Ontario.

The current DSR reviews the Port Severn Main Dam and its immediate associated structures. The structure known and referred to as the Port Severn Main Dam is made of the following dam components:

- Main Dam.
- Lock 45 and Upstream Shoreline Wall.
- Dam D.

Dam C, Dam E (Bayview) and Dam G (Little Chute) are the object of separate DSR reports. The water-retaining structures known as Dam A, Dam B, Dam B1, Dam F and Little Go Home Bay Dam are the object of another report. That report is not a full DSR, but presents the results of the inspection and conditions assessment, review of the status and confirmation of hazard classification.

1.2 Scope of the Dam Safety Review

The Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures (Reference 49) provides guidance to PCA for the management of its dams and water-retaining structures. When the PCA Directive does not specify or give sufficient guidance, good dam safety practices as presented by the CDA's Dam Safety Guidelines 2007 (Reference 8) shall be followed. The current DSR report respects guidelines from both documents. In case of conflicting guidelines, the PCA Directive takes precedence.

The DSR is defined as "a systematic review and evaluation of all aspects of design, construction, operation, maintenance, processes and other systems affecting a dam's safety, including the dam safety management system". The main objective of a DSR is to demonstrate whether the dam meets the dam safety requirements. Should it be demonstrated that the dam does not meet the requirements, the DSR must recommend actions to be taken for the dam to meet the safety requirements or otherwise maximize the security of the impacted population.

The necessity of a Dam Safety Review depends on the status of the water-retaining structure (i.e. is it a dam, according to PCA's definition?), and its hazard classification (i.e. what are the impacts of a failure?). A DSR, as its name implies, must be performed for hydraulic structures defined as "dams". This entails that a hydraulic structure must first be evaluated and categorized. The structures at Port Severn are assessed according to the PCA Directive (Reference 49). Classification of each structure is detailed in Section 1.5 - Dam Status Review.

According to the PCA Directive, a dam safety review shall be carried out for all dams, except when hazard classification is VERY LOW. A dam hazard classification is evaluated based on the expected consequences due to a dam failure. Also, the frequency of reviews depends on the hazard classification. Section 6 - Dam Classification Review fully details the classification of the Port Severn Dams.

The DSR must be sufficiently detailed to meet its objective. The level of detail may be modified based on previous assessments, complexity of the dam, surveillance records, external and internal hazards, operating history, performance and age and the need for public protection.

As part of the DSR, the following activities were performed:

- Review of historic documentation, plans and literature.
- Site inspection conducted in December 2011 to assess the operational procedures, safety conditions and the status of the dam components.
- Interviews conducted with the operators in December 2011 to gather data on operational procedures, safety conditions, recurring issues and the status of the dam components.
- Concrete core testing conducted in April 2012.
- Topographical surveys done in May 2012.
- Analysis of failure modes, inflow design floods.
- Analysis of reliability and functionality of discharge facilities.
- Review of the consequences of dam failure, including flood mapping and estimation of loss of life.
- Dam classification.
- Structural and stability analyses, based on different load combinations.
- Review of operational procedures.
- Review of occupational health and safety (OHS) and public safety around dams.
- Review of the overall effectiveness of safety management at the dam.
- Costing, scheduling and prioritization of remedial measures.
- Recommendations.

1.3 Site Location

The dams included in the study are all located in or near the town of Port Severn, on the Severn River at the outlet into Georgian Bay. Main Dam, Lock 45 and the Upstream Shoreline Wall, Dam D, Dam C, Dam E and Dam G are located in the town of Port Severn. Dam F is located between Dam E and Dam G. Access to these dams is possible via Port Severn Road. The upper deck of the Main Dam serves as a bridge for the Port Severn Road. Dam B and Dam B1 are located 400 m northeast of Main Dam in an open field located southeast of Kelly’s Road. Dam A is located in a small bay on Little Lake, 930 m to the northeast of Main Dam and is accessed via Baguley Road. Finally, Little Go Home Bay Dam is located in the western arm of Gloucester Pool about 9.7 km northwest of Main Dam on Baxter Lake.

The lake formed by the Port Severn water-retaining structures is known as the Gloucester Pool and Little Lake. The outflows from the Main Dam discharge into Georgian Bay.

Appendix A presents the general location plan of the Port Severn Dams. Their Universal Transverse Mercator (UTM) coordinates are presented in Table 1.1. The location of the structures specifically described in the current DSR (Main Dam, Lock45, the Upstream Shoreline Wall and the Dam D) is shown in Figure 1.1.

Table 1.1 UTM Locations of the Port Severn Dams

Site	UTM Location	
Port Severn Main Dam		
Main Dam	44°48'13.36"N	79°43'11.91"W
Lock 45	44°48'14.56"N	79°43'13.85"W
Upstream Shoreline Wall	44°48'15.45"N	79°43'14.48"W
Dam D	44°48'13.79"N	79°43'09.77"W
Other Port Severn Dams		
Bayview Dam E	44°48'12.53"N	79°43'20.74"W
Little Chute Dam G	44°48'12.80"N	79°43'47.30"W
Port Severn Water-Retaining Structures and Others		
Dam A	44°48'30.77"N	79°42'39.59"W
Dam B	44°48'21.15"N	79°42'58.24"W
Dam B1	44°48'21.90"N	79°43'01.00"W
Dam C	44°48'17.54"N	79°43'06.64"W
Dam F	44°48'10.00"N	79°43'27.60"W
Little Go Home Bay Dam	44°53'10.00"N	79°45'51.45"W

Figure 1.1 General Layout of the Port Severn Main Dam

1.4 Site Description

The Port Severn Main Dam rests on the line dividing the townships of Severn and Georgian Bay, at the outlet of the Severn River, where it flows into Georgian Bay. According to the 2011 census, the Township of Severn is home to more than 12,000 residents in 2011 (Reference 55). Growth projections indicate that the Township's population is increasing rapidly. Estimates for the period of 1996-2016 projected a rise of 5,200 residents, with 2,210 households being added. The population of the Township of Georgian Bay has risen from 1,991 in 2001 to 2,482 in 2011. The 5,173 private dwellings include a very high number of summer homes. Sections 6.6 and 6.7 give further information on the population and dwellings living close to the dam and at risk of flooding.

The Port Severn Main Dam is the northern gateway to the Trent-Severn Waterway (TSW) and Lock 45 is the smallest lock on the TSW. With the completion of the TSW in 1915, the region's economy shifted from lumber to tourism as Port Severn attracted boaters with its natural surroundings, historic character and position as entry to the TSW. Its 25.6 m (84 ft) length limits the size of vessel that can through-navigate the entire waterway. The lock is 3.76 m deep and PCA guarantees a 1.64 m draft (5 feet).

Port Severn Road, which runs atop the Main Dam, is a part of Simcoe County Road 5. Bridge 60, over Lock 45, is a metal, eight-panel bolt-connected pony truss swing bridge (Reference 35). The Trans-Canada Highway, also known as Highway 400, crosses the bay 460 m downstream of the dams.

The reservoir (Gloucester Pool and Little Lake) immediately upstream of the dam is surrounded by marinas, chalets, resorts and permanent houses. The body of water downstream of the dam is surrounded by more densely populated communities, with dwellings built along most of the shoreline. Many of the dwellings appear to be for seasonal use. There are no major industrial facilities within the vicinity of the Port Severn Dams. Marinas, restaurants, hotels, resorts, boat rentals and lakeshore chalets for rent are the most significant commercial activity. The Oak Bay Golf Club is located 1.5 km west of the dam, along Port Severn Road.

1.5 Dam Status Review

The application of standards to water-retaining structures depends on their status. For a PCA-owned water-retaining structure to be recognized as a dam, and thus be subject to the associated standards, it must meet the PCA Directive's definition (Reference 49):

> *A barrier constructed for the retention of water, water containing any other substance, fluid waste, or tailings, provided the barrier is capable of impounding at least 30,000 m³ of liquid and is at least 2.5 m high. Height is measured vertically to the top of the barrier: (i) from the natural bed of the stream or watercourse at the downstream toe of the barrier, in the case of a barrier across a stream or watercourse; or (ii) from the lowest elevation at the outside limit of the barrier, in the case of a barrier that is not across a stream or watercourse.*
>
> *In this Directive, the term dam includes appurtenances and systems incidental to, necessary for, or connected with the barrier.*
>
> *The definition may be expanded to include dams less than 2.5 m high or with an impoundment capacity of less than 30,000 m³ if the consequences of dam operation or failure are likely to be unacceptable to the public […].*

Table 1.2 presents the status, as understood by AECOM, of the structures of Port Severn. These structures (except for Dam B, Dam B1 and Dam F) retain the water of Little Lake and Gloucester Pool Reservoirs under normal conditions. Note that the maximum water level at Port Severn is 180.50 m.

Table 1.2 Structures of the Port Severn Dams – Status (from 2012 Surveys)

Port Severn Dams	Crest Elevation m	Bottom Elevation m	Height m	Dam	Water-Retaining Structure	Other
Port Severn Main Dam						
Main Dam	183.02	172.47	10.55	X		
Lock 45	181.31	173.57	7.74	X		
Upstream Shoreline Wall	181.31	178.57	2.74	X		
Dam D	181.39	179.43	1.96	X		
Other Port Severn Dams						
Dam E – Bayview	181.96	177.03	4.93	X		
Dam G – Little Chute	181.36	177.10	4.26	X		
Port Severn Water-Retaining Structures and Others						
Dam A	181.37	179.97	1.40		X	
Dam B	181.27	179.79	1.48			X
Dam B1	180.95	180.92	0.03			X
Dam C	181.42	179.04	2.38		X*	
Dam F	181.27	180.19	1.08			X
Little Go Home Bay Dam	180.62	179.47	1.15		X	

* Dam C is considered as a water-retaining structure according to the criteria used (height and impoundment capacity). Nonetheless, a DSR is performed since it is very close to the acceptability threshold for a dam and there may be consequences in case of failure.

Port Severn Main Dam (Main Dam, Dam D, Lock 45 and the Upstream Shoreline Wall), Dam E and Dam G are considered as dams and will be the object of a complete Dam Safety Review. Dam C is considered as a water-retaining structure according to the criteria used (< 2.5 m). A DSR is nonetheless performed since it is very close to the acceptability threshold for a dam and there may be consequences unacceptable to the public in case of failure.

Little Go Home Bay Dam and Dam A are considered as water-retaining structures since their height is less than 2.5 m. In addition the consequences of a failure are unlikely to be unacceptable to the public. It is also noted that there are significant outcrops of natural bedrock at several locations, which minimizes the impact to the public by reducing the effects of a dam break.

Dam B and Dam B1 are listed in the category “Other”. These structures are less than 2.5 m high and do not contribute to the impoundment of water in the reservoir. Their crests are at the same elevation or lower that the land on either sides of the dam. The consequences of a failure are unlikely to be unacceptable to the public.

Dam F is located between buildings on the reservoir side (north) and Port Severn Road (south), which is approximately 20 m away. The dam is listed in the category “Other” since it does not contribute to the impoundment of water in the reservoir. The surface water link between the reservoir and Dam F has been eliminated and the dam no longer impounds water at all. This is possibly due to the area being infilled for construction. Water is visible on either side of Dam F and this is due to the low land water table in this isolated spot. In the event of a large flood, water would overtop Dam F and would be held back by the road, located behind the dam (south), at higher elevation (± 181.85 m).

2. Background Information

Background information on the use and state of the dams has been acquired through documentation review and interview sessions with the operators. Historic information on events and operation procedures that have taken place is also collected through discussions with the operators.

2.1 General

The TSW is a National Historic Site of Canada. It runs 386 km from the Bay of Quinte on Lake Ontario at the City of Quinte West (Trenton) to Port Severn, located in the south of Georgian Bay (Lake Huron). The Ontario Waterways Unit of Parks Canada Agency manages and operates the Trent Severn Waterway National Historic site, including its 44 locks. It also owns and operates the 143 dams of the Trent River Watershed and Severn River Watershed which control the water levels in the waterway.

The Ontario Waterways Unit manages the Port Severn Dams with the intent of maintaining water levels for navigation. While flood management is not the main objective of the operations at the Port Severn Dams, they must nonetheless have the capacity to discharge the IDF and meet the dam safety requirements as per the PCA Directive and the CDA Guidelines (References 49 and 8). Therefore, while regular operation is performed to maintain navigation water levels, the staff must be proficient to operate the dam and the facilities must have the capacity to discharge floods and keep risks as low as reasonably practicable.

Management for navigation purposes also maintains favorable consistent water levels for lakeside residents and businesses of the Gloucester Pool and Little Lake. The crew operates the Port Severn Dams seasonally. Table 2.1 indicates the seasonal normal water levels.

- Date of fall drawdown initiation: after Thanksgiving.
- Date for achieving total drawdown: December 1st.
- Date for reaching normal operating level: 1 week before Victoria Day.
- Water levels do not vary much: fluctuation within 30 cm throughout the year.

Table 2.1 Normal Water Levels

Water Levels	Navigation Season (Summer)	Non-Navigation Season (Winter)
Maximum	180.50 m	180.50 m
Minimum	180.42 m	180.20 m

Note: Winter levels normally vary between 180.20 m and 180.30 m, but can occasionally reach 180.50 m.

2.2 History

2.2.1 Investigations

Construction of the TSW began in late 18th century with the building of small dams and water powered mills at numerous locations through south-central Ontario. Since construction in 1916, the Port Severn Main Dam has been the object of few investigations, testing and sampling programs. As part of the current mandate, AECOM proceeded to a visual inspection of civil, geotechnical and hydro-mechanical facilities, observations of operational procedures and safety review on December 5th to 7th 2011. This is detailed in Section 4 - Observations, Inspection and Document Review. Also, Geo-Logic Inc. performed concrete core testing on Dams C, D, E, G, and Lock 45 (see Appendix F). The cores were sampled on April 20th, 2012.

Below is a list of the other known investigations and reports:

- November 1992 – DBA Engineering Ltd. (Reference 24) – Geotechnical investigations. The tests included concrete core testing on Main Dam.
- December 1985 – Peto MacCallum Ltd. (Reference 51) – Geotechnical investigations.

Since their construction, the Port Severn Dams have been the object of minor repairs (Reference 24). Works recorded throughout the years and made available for the current study, are as follows:

- 1985: Refacing deteriorated areas of the lock chamber, including entrance walls, the Upstream Shoreline Wall and the dam side of the lock wall (References 5 and 51).
- 1985: Construction of a 30 m pier at each of the upper and lower entrances (Reference 51).
- 1985: Construction of a new lock control building (References 51).

2.2.2 Water Levels and Operation

In April 1960, the Swift Rapids station recorded a discharge of 275 m^3/s. This remains the highest mean daily discharge recorded at this station since its implementation in 1953. In 1948, the water level in Gloucester Pool reached a historical peak of 180.72 m.

2.3 Description of Dam Components

2.3.1 Main Dam

The Main Dam is located on the TSW, adjacent to Georgian Bay in Port Severn, Ontario. Figure 2.1 shows an upstream view of the structures. Because it is an upstream view, the left bank (east) is on the right of the picture and the right bank (west), on the left (by convention the left and right banks of a river are designated when looking downstream). Drawing 005 of Appendix A includes details of the Main Dam.

The left abutment of the Main Dam (right side of the picture) sits on bedrock while the right side of the dam joins the left wall of Lock 45. The existing dam has a width of 71.32 m. The project brief mentions a height of 8.99 m from top of deck to bedrock, though it has been surveyed at 10.55 m during the site inspection (crest elevation at 183.02 m). It includes an upper deck and contains nine stoplog sluices to control water levels, separated by piers.

The dam deck is made of reinforced concrete. The downstream section of the dam deck acts as a bridge of the Port Severn Road. The upstream section acts as an operation area on which three parallel steel rails run along the dam deck to convey the hydraulic log lifter and the manual winches.

All eight piers are 1.83 m wide and made of concrete. The upstream noses of the piers are rounded to reduce hydraulic drag. Each sluice has two sets of gains built into the piers to accommodate the stoplogs inserted to control discharge through the dam.

By convention, sluice numbering starts at the lock. Sluice 1 is the rightmost (west) sluice. Each of the nine sluices is 6.10 m wide. These sluices sit on concrete sills. Sluices 2 and 3 can hold up to 12 logs, while all others are limited to 10. Sluices 1 to 6 are located over the river bed. Sluices 7 to 9 are located over non-submerged ground. Furthermore, an electricity distribution pole sits in front of sluice 9. This suggests that the operation of sluices 7 to 9 could cause damage over the exposed surface.

The stoplogs are made from 305 mm (12”) high by 356 mm (14”) wide Douglas Fir wood and are 6.63 m (21’-9”) long (Photo 22 of Appendix B1). Older logs are treated but newer ones are not. At each end of the stoplogs there is a recessed steel handling ring, referred to as a “dee” which is hooked for log removal.

Figure 2.1 Upstream View of Port Severn Main Dam

2.3.2 Lock 45 and Upstream Shoreline Wall

Lock 45 is located at the right end of the Main Dam. Due to the 25.6 m length of the lock, the size of vessels which can navigate through the waterway is limited. The lock is 3.76 m deep and PCA guarantees a 1.64 m draft (5 feet). Lock 45 and the Upstream Shoreline Wall are built on bedrock. The left wall of the lock extends downstream approximately 41 m to protect boats from the back current of the Main Dam’s sluices. The crest elevation of both lock and wall is 181.31 m. The valves and gates are manually operated.

The original left wall of the lock was repaired in 1985. The top portion of the wall was removed and replaced with new concrete in addition to the lock walls being refaced. The left wall is solid mass concrete next to both sets of gates. At the entrance and along the chamber, the concrete wall has hollow sections that were filled.

Swing Bridge 60 allows vehicles and pedestrians to cross over the lock. This bridge consists of a series of concrete t-beam spans that share a common substructure with the dam and there is a pony truss swing span over the canal (Reference 35). The pier on which the bridge rotates is located on the right shore. The original swing span was demolished and replaced with a modern bridge, constructed to retain the general appearance of the heritage bridge.

2.3.3 Dam D

Dam D is located at the east end of the Main Dam and is a 70 m long concrete face embankment dam. Initially considered as 2.75 m tall, it has been surveyed at 1.96 m during the site inspection (crest elevation at 181.39 m). Dam D is founded on bedrock. It has no mechanized parts and does not require operation.

2.4 Regular Operation

2.4.1 Main Dam

The Main Dam is operated primarily to maintain the levels upstream within the acceptable range for navigation. As stated in the PCA Directive and CDA Guidelines, the owner of the dam is also responsible for its safe management of floods. In the case of a significant flood event, the dam operator must be able to operate the spillway to discharge flows sufficiently to reduce risks to a level as low as reasonably practicable (ALARP).

Main Dam is the main water control structure at Port Severn. At Main Dam, only sluices 2 to 6 are ever used, with sluices 2 and 3 being the most frequently operated. Dam E and Dam G can also contribute to the water management at Port Severn.

Discharge through the sluices is controlled by adding or removing stoplogs. In each sluice, the stoplogs are inserted in the gains on top of each other to block the water flow. When removed from the sluices, stoplogs are stored in the operation area of the dam deck. Detailed instructions on the use of the equipment for dam operation are available in Section 7 of the TSW Standing Orders (SO, Reference 50).

Each sluice boasts two sets of gains. The downstream gains are the operation gains. Dam operation is done by inserting and removing logs from the operation gains. The upstream gains are the maintenance gains. They remain free of logs. Logs are only inserted in the maintenance gains when maintenance must be done on the operation gains.

Stoplogs in sluices 2 to 8 can be handled with the hydraulic log lifter (Photo 26 of App. B1). The log lifter travels along the dam deck rails. When it is positioned over a sluice, the cover is removed. From the deck, the operators hook the guided hooks (Photo 27 of App. B1) of the log lifter onto the log dees (i.e. recessed steel handling rings). The logs are lifted and lowered into place. The log lifter can reach neither sluice 1 nor sluice 9 because of obstacles (fences).

Manual winches serve as a back-up when the hydraulic log lifter is unusable. They are also used to handle logs at sluices 1 and 9 that cannot be reached by the log lifter. They are operated from the deck. To hoist a log, a manual winch is placed on either side of a sluice. Winches hook into the dees, allowing the stoplogs to be lifted and lowered.

If the dee is broken or if submerged logs are too difficult to hook, logging tongs are used to grab the logs. Tongs can replace guided hooks on the log lifter (Photo 36 of App. B1) and manual winches. Photo 37 of App. B1 shows the operators trying to grab a submerged log with tongs. The tongs, chained to the winches, are guided by the operators using pike poles (Photo 37 of App. B1). Tongs are also used to correct the alignment in the gains when the logs get jammed.

When lowering logs into the sluices with the lifter, downward pressure must be applied to the log to obtain a tight seal between the logs and with the sill. When using the winches or if a gap remains between the logs after having applied pressure with the log lifter, jacking is performed to properly seal the sluice. Jacking consists of applying a downward force on the top log through the use of a jack, wedged between the log and the deck (Photo 32 of App. B1). To position the jack, an operator must partly enter the sluice.

The removal of debris and breaking of ice are also part of the regular operation performed by the operators. To perform this operation, the operators remove the sluice's cover and use long poles or tiger torches.

2.4.2 Lock 45

The lock is operated to allow passage of boats. Both the upstream and downstream gates are manually operated. Detailed instructions on the operation of the upstream and downstream lock gates are available in the TSW SO 5.2 – Operation of a Manual Lock and TSW SO 5.2 – Operation of a Manual Lock, respectively (Reference 50).

2.5 Geology and Geotechnical Characteristics

2.5.1 Geology

The Port Severn Dams are located in the Grenville Province dominated by grey, migmatitic tonalitic and granodioritic ortho gneiss (Reference 44). Available data from geotechnical investigations carried out in 1992 at the Main Dam (Reference 24) identifies the bedrock as a banded black migmatite gneiss with some pink coloration. This bedrock displays numerous quartz veins, and feldspar, biotite and hornblende are often noticed. The bedrock is unweathered, of high strength with bedding joints at close to moderate intervals.

Additional investigations conducted in April 2012 on the concrete structures of the lock and on Dam C, Dam D, Dam E and Dam G confirm the presence of migmatite gneiss as the bedrock under these structures.

2.5.2 Geotechnical Conditions

There is little information regarding previous geotechnical investigations available for the Port Severn Dams. A geotechnical investigation was carried out around the lock in November 1985 by Peto MacCallum Ltd (Reference 51).

The field work for this geotechnical investigation consisted of four boreholes. Two of these boreholes were drilled over water from a barge, and the two others were drilled on the west side of the lock structure, with one borehole drilled near the entrance of the lock house and the other one drilled on the west side of the lock house.

Information from those boreholes indicates that the overburden thickness around the lock varies from approximately 0.9 m to approximately 2.10 m. The overburden material is identified as loose to compact silt and till. This has been confirmed by visual observation of the embankments around the Port Seven Dams during the site inspection conducted in December 2011.

2.6 Seismicity

According to Earthquakes Canada, Port Severn is located in a low relative hazard zone and therefore has a relatively low likelihood of experiencing strong earthquakes. This can be seen on a simplified seismic hazard map showing the relative seismic hazard across Canada (Figure 2.2). Although the map on Figure 2.2 shows a relative seismic hazard for spectral acceleration at 0.2 second period, the distribution for Peak Ground Acceleration is the same, meaning that a region with high relative hazard on Figure 2.2 will also have a high PGA. The mean PGA (Peak Ground Acceleration) for the Port Severn region is around 0.020 g for a 10 % probability of exceedance in 50 years (0.0021/year) as calculated specifically for the Port Severn Dams by the Geological Survey of Canada

(Reference 34). A PGA of 0.020 g is low, given that regions of high seismic hazard like La Malbaie,QC or Victoria,BC have PGA values of 0.52 g and 0.34 g for the same probability (Reference 43).

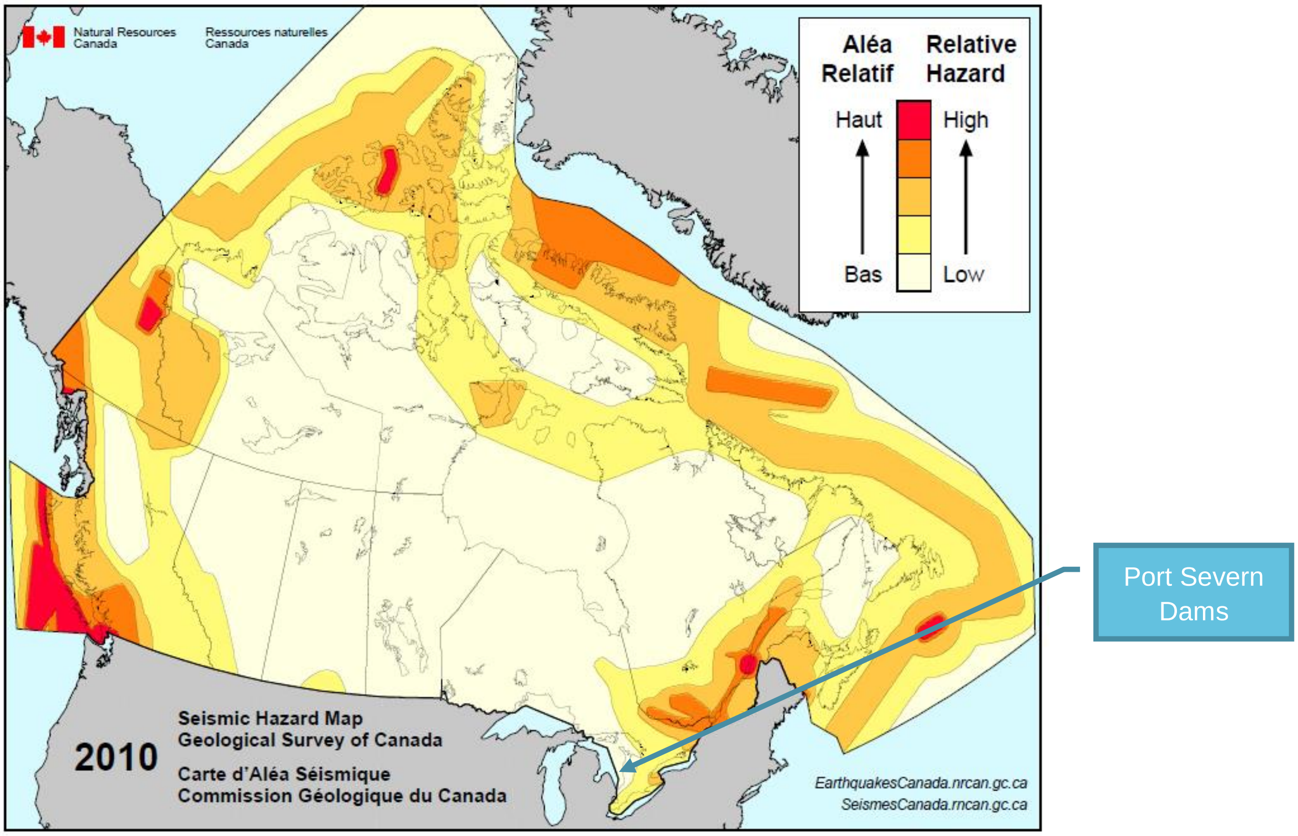

Figure 2.2 Simplified Seismic Hazard Map of Canada (Reference 43)

3. Drawings

Appendix A includes all the drawings for the Main Dam, Lock 45, the Upstream Shoreline Wall and Dam D. Drawings were done to represent the site layout as well as the detailed structures. The drawings produced and presented are based on information obtained from the following sources:

- Previous drawings, supplied by PCA.
- Topographical surveys performed as part of the current mandate.
- Observations from the site inspection of December 2011, which is detailed in Section 4 - Observations, Inspection and Document Review.
- Photographs taken during the site inspection or supplied by PCA.
- The digital elevation model (DEM), tile 104 at a resolution of 10 m, UTM zone 17, NAD 83 CNT, excerpted from the NRVIS Provincial Digital Elevation Model Data, available from the Ontario Ministry of Natural Resources.
- Orthophotos, resolution 30 cm, supplied by PCA.
- Nautical charts, published by the Canadian Hydrographic Service.

More specifically, the available drawings were supplied by PCA in the form of .tiff files. They are the following:

- Plan of Small Dams at Port Severn As-Build, T11-27309, (1918).
- Plan of Dam and Highway Bridge, T11-27304, (1913).
- Amended Plan of Dam and Highway Bridge, T11-27303, (1914).
- Final Plan of Dam and Highway Bridge, T11-27308, (1918).
- Layout Plan of Lock and Dam at Port Severn, T20-20204, (1918).
- Plan showing Topographical Details Port Severn Locks, T20-24302, (1968).
- Plan showing Topographical Details Port Severn Locks, T41-21301, (1968).
- Plan showing Topographical Details Port Severn Locks, T41-21302, (1968).
- No Title T41-36801, (1984).
- No Title T41-36802, (1984).
- Topographical Plan, T41-42003, (1984).

The drawings produced as part of the current mandate are:

- 001: General Location Plan.
- 002: Location Plan – Upstream Shoreline Wall, Lock 45, Main Dam, Dam D and Dam C.
- 003: Location Plan – Dam G.
- 004: Location Plan – Dam E.
- 005: Main Dam – Plan, Elevation, Sections and Detail.
- 006: Main Dam – Sections.
- 007: Lock 45 and Upstream Shoreline Wall – Plan, Elevation and Details.
- 008: Lock 45 and Upstream Shoreline Wall – Sections.
- 009: Dams A, B and B1 – Plan and Elevation.
- 010: Dams C, D, F and Little Chute Dam G – Plan, Elevations and Sections.
- 011: Bayview Dam E and Little Go Home Bay Dam – Plan, Elevations and Sections.
- 012: Little Chute Dam G – Plan.
- 013: Little Chute Dam G – Plan, Elevations and Sections.

Drawings related to the Port Severn Main Dam (Main Dam, Lock 45, Upstream Shoreline Wall and Dam D) can be found in Appendix A.

4. Observations, Inspection and Document Review

The current section focuses primarily on observations made during the site inspection and information gathered from documentation review, interviews or otherwise. Analysis and comments based upon this information are available in Section 7 - Dam Safety Analysis. The current section is divided in the following sub-sections:

- Documents reviewed: a list of the technical documentation made available and reviewed
- Inspection program: a summary of how the inspection was performed
- Current state of the dams: a listing of the observations, concerning physical assets (equipment, structures, land), stemming from the inspection of the facilities, document review and interviews
- Current operation procedures: a listing of the observations, concerning operations, stemming from the supervision of operation and testing, document review and interviews
- Maintenance: a listing of the maintenance activities that are performed at the Port Severn Dams, according to the operators interviewed and documentation reviewed
- Safety: a listing of observations concerning occupational health and safety and public safety
- Review of existing policies, Operation Maintenance and Surveillance (OMS) plan, Emergency Preparedness Plan (EPP) and Emergency Response Plan (ERP). In the case of Port Severn, these did not exist.
- Deficiencies: a summary of all deficiencies identified during observations, inspection and document review

4.1 Documents Reviewed

To complete the DSR, a literature review was performed. Documents reviewed include historic records of the structures, data sets, guidelines, standards and other reports.

The following is a listing of the documents reviewed to assess the condition of the dams.

- AECOM, Severn River Watershed Hydro-Technical Study, Flood Flows Estimation Study Report, June 2012 (Reference 3): This study provides hydrographs and peak flows for several flood conditions, including the PMF.
- Genivar – Establishing Mitigating Measures for High Risk Dams in Ontario – Northern Sector – November 2011 (Reference 28): This study presents the site inspection observations for dams located within the Trent-Severn waterway and identifies mitigation measures for the dams considered at high risk including a class D cost estimate.
- Ecoplans Limited – Trent-Severn Water Management Study – Water Management Program – May 2007 (Reference 25): This study outlines the background on the way Parks Canada manages water flows and levels on the Trent-Severn waterway.
- DBA Engineering Ltd. – Geotechnical Investigations Main Dam – November 1992 (Reference 24): This study presents the results of a geotechnical investigation carried out to determine the quality of the concrete in the Main Dam to evaluate the nature of the bedrock foundation and to assess the water tightness of the dam and bedrock.

The following is a list of the guidelines and standards that were reviewed and respected throughout the studies.

- PCA, Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures, 2009 (Reference 49).
- Canadian Dam Association (CDA), Dam Safety Guidelines, 2007 (Reference 8).
- Canadian Dam Association (CDA), Technical Bulletins:
  - Inundation, Consequences, and Classification for Dam Safety, 2007 (Reference 9).
  - Seismic Hazard Considerations for Dam Safety, 2007 (Reference 10).

  - Surveillance of Dam Facilities, 2007 (Reference 11).
  - Geotechnical Considerations for Dam Safety, 2007 (Reference 12).
  - Dam Safety Analysis and Assessment, 2007 (Reference 13).
  - Public Safety and Security around Dams, 2007 (Reference 14).
  - Structural Considerations for Dam Safety, 2007 (Reference 15).
  - Hydrotechnical Considerations for Dam Safety, 2007 (Reference 16).
  - Flow Control Equipment for Dam Safety, 2007 (Reference 17).
- Government of Canada, Department of Justice, Canada Labour Code, R.S.C., 1985, c. L-2, 1985 (Reference 30).
- Government of Ontario, Occupational Health and Safety Act (OHSA), 1990, amended in 2011 (Reference 31).
- Natural Resources Canada (NRC), National Building Code of Canada, 2010 (Reference 42).
- Parks Canada Agency (PCA), Trent-Severn Waterway Operational Standing Orders, 2010 (Reference 50).
- United States Bureau of Reclamation (USBR), A Procedure for Estimating Loss of Life Caused by Dam Failure, DSO-99-06, 1999 (Reference 62).
- Federal Energy Regulatory Commission (FERC), Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter 2 – Selecting and Accommodating Inflow Design Floods for Dams, 1993 (Reference 27).
- Center for Energy Advancement through Technical Innovation (CEATI), Static Ice Loads on Hydroelectric Structures – Summary Report, Ice Load Design Guide and Ice Load Prediction Computer Program, 2003 (Reference 18).

For a complete listing of all documents considered in the DSR, see the References section.

4.2 Site Inspection

The site inspection was conducted on December 5th to 7th, 2011 and covered nine structures: Port Severn Main Dam (including Main Dam, Lock 45, the Upstream Shoreline Wall, and Dam D), Dam A, Dam B, Dam B1, Dam C, Dam E (Bayview Dam), Dam F, Dam G (Little Chute) and Little Go Home Bay Dam. Other site visits include a second public safety assessment on May 2nd, 2012 and a second flow test on November 5th, 2012. The current DSR reviews the status of the:

- Main Dam.
- Lock 45 and Upstream Shoreline Wall.
- Dam D.

The other associated structures in Port-Severn are reviewed in separate DSR studies.

The team consisted of experienced engineers, an Occupational Health and Safety Act (OHSA) auditor and a public safety specialist. During the visit, information was gathered through observation and by conducting interviews with operators and staff. Data gathered includes:

- Structural, mechanical, and geotechnical integrity of the works.
- Assessment of the hydraulic conditions in the upstream and downstream reaches.
- Operational procedures.
- Maintenance of the facilities.
- Occupational health and safety aspects.
- Public safety.

The site inspection was conducted to assess the condition of each dam and water-retaining structure. It also included operational testing of the lifting devices and associated equipment. Inspections were conducted by making observations from structures accessible by foot, such as the dam decks and the upstream and downstream banks. A systematic examination of each surface was performed to detect and locate, if present, cracking, deterioration, seepage, and other visible damage. Notable specifics concerning the inspection program include:

- The under-deck was accessed by removing the wooden covers on the stoplog gain openings, and standing on top of the stoplogs.
- An underwater camera mounted on a pole was used to inspect surfaces of the dams and lock walls which are below the water level.
- Geotechnical components were visually inspected to detect anomalies such as piping, settlement, slope movement or deterioration of embankments.
- The visual inspection of hydro-mechanical components aimed at verifying the condition, reliability, and functionality of the spillway stoplogs along with their lifting equipment and power supply. Only visible parts were inspected. Functional tests were conducted on spillway and lock gates.
- Observations regarding public safety as well as occupational health and safety aspects were noted.
- Testing of the operational procedures was done in accordance with the Flow Control Equipment for Dam Safety CDA Technical Bulletin 2007 (Reference 17).
- The water level during the site inspection was 180.25 m. Winter settings vary between 180.20 and 180.50 m though it is preferable to maintain them below 180.30 m to keep ice levels lower.

The current report describes the main observations and findings from the inspection, including photographs in Appendix B. In addition, sketches inserted at the beginning of Appendix B locate significant observations on the works and refer to pertinent photographs.

Part of the inspection program included discussions with the operators. A list of questions was asked to the operators during the start-up meeting and during the site inspection. Recorded answers are added to the observations and used to assess the operational procedures and the safety review. Question and answers are available in Appendix H.

4.3 State of the Dams

The physical observations, pertaining to the structural, mechanical, and geotechnical components of the works, are separated for each structure. Observations on the operational procedures, operator and public safety, and maintenance program for all the structures are regrouped.

4.3.1 Main Dam

Photos presenting the observations that were made by each discipline are presented in Appendix B1 – Pictures of Main Dam. Photos 1 to 3 show the upstream and downstream views of the Main Dam.

4.3.1.1 Structure and Civil Works

A visual inspection of the Main Dam was performed. Inspection purpose was to assess the general condition of the dam. The visual inspection was done from the deck of the dam as well as from the right and left banks. Under-deck was inspected at sluices 1, 2, 3, 7, 8 and 9. Under-deck at sluices 4, 5, and 6 could not be inspected since these were partially open.

An underwater camera was used to inspect the upstream face of the dam between pillars 2 and 3, at the nose of pier 2 and on the right gain of sluice 2. Pillars (and gains) of sluice 2 were chosen because that sluice is one of the most frequently used and also because the hydraulic log lifter was located over sluice 3. Structural state of the other piers is expected to be no worse than in the inspected area since this area is the most solicited.

The visual inspection and local measurements showed that the geometry and dimensions are in agreement with the original drawings.

Several anomalies have been identified and are described hereafter:

- Old repairs of the joints between the deck slabs at the piers' centerline location with grout are deteriorated (Photo 4 of App. B1, pier 5).
- Cracks can be seen on the deck at the corner of the gains (Photo 5 of App. B1, sluice 3).
- Moss is present along the dam deck rails at some locations (Photo 6 of App. B1, sluice 6).
- Observations noted at each sluice are the following:
  - Photo 7 of App. B1 shows the downstream part of the right wall of sluice 1 which is fairly damaged.
  - At sluice 2, the under-deck concrete is considered to be in good condition. However, a few stirrups are apparent and rusted but no delaminating was observed (Photo 8 of App. B1). At the time of construction, concrete covering of these stirrups was not sufficient.
  - Reinforcement bars of the beams supporting the deck at sluice 3 are apparent and rusted (Photo 9 of App. B1).
  - According to the dam operators, sluices 4 and 5 are probably in a similar condition as sluice 3, since these three are the most frequently opened sluices. Sluice 6 is not frequently opened, and is thus in better condition than sluices 3, 4 and 5.
  - The under-deck concrete at sluices 7, 8, and 9 is in better condition (Photo 10 of App. B1, sluice 7).
  - The gains show some deterioration. On some bays, the top ends of the embedded steel angles are delaminated from the concrete, making it difficult to insert the stoplogs (Photos 23 and 24 of App. B1).
- Pieces of concrete on the piers in the drawdown zone have broken away and large cracks are present at some locations (Photo 2 of App. B1 showing the upstream general view of the piers and Photo 11 of App. B1 showing piers 7 and 8).
- A large crack (approximate width of 5 mm) was observed between the noses of the piers and the deck, at the joint at pier centreline. Photo 12 of App. B1 shows the top of the nose of pier 2 and Photo 13 of App. B1 shows the top of nose of pier 7.
- Two very large cracks (approximate width of 25 mm) were observed on either ends of the Main Dam: one at the intersection of Dam D (Photo 14 of App. B1), and a second in the wall common to the lock (Photo 7 of App. B2).
- Submerged concrete surfaces are covered with zebra mussels (Photos 15 and 21 of App. B1).
- The concrete on the left side of pier 2 is deteriorated at some locations and some aggregates are exposed (Photo 16 of App. B1).
- On the left side of pier 2, the construction joint at the level of the embedded part of the gain is open (Photo 17 of App. B1).
- On the left side of pier 2, a crack or an open construction joint near the embedded part of the gain is also noted (Photo 18 of App. B1).
- The concrete on the right face of pier 3 is partially damaged under the beams of the deck (Photo 19 of App. B1).
- There is a large crack below the crest of the nose downstream of pier 3 (Photo 20 of App. B1).

4.3.1.2 Mechanical Works

The following observations were noted concerning the mechanical equipment at the Main Dam:

- Gears, winches, frames and wheels of the log lifter and manual winches are in good condition.
- The hydraulic log lifter is maintained in acceptable working condition but shows some surface rust and wearing.
- The hydraulic log lifter is nearing the end of its life cycle. The life cycle of an Atlas Polar log lifter is estimated by their sales staff at 40 years, depending on use and maintenance.
- The hydraulic log lifter's sideway adjusting rails (mechanical parts of the log lifter, not to be confused with dam deck rails) are badly damaged (Photo 30 of App. B1).
- The painting has deteriorated. Minor rust has appeared.
- In operation, the diesel engine is a little bit noisy with some undesired exhaust fumes. These conditions could be due to the age of the unit.
- The diesel engine is refueled on the dam deck, potentially creating an environmental risk associated with spilled fuel entering the river. No spill protection is used. Operators mention that only a few drops are every spilled and that they are immediately wiped off.
- It has been demonstrated that the log lifter does not have sufficient power to push the stoplogs downward.
- Fences prevent the log lifter from reaching sluices 1 and 9, and therefore logs in the end sluices must be added or removed using the backup manual winches.

The following observations on the stoplogs were noted:

- At some of the sluices, there is significant water leakage between stoplogs.
- Some stoplogs are covered with zebra mussels (Photo 21 of App. B1).
- Not all the stoplogs are in the same condition. Some stoplogs are new (Photo 22 of App. B1) while some are old and should be replaced (Photo 40 of App. B1). It is estimated that approximate percentages for each age and condition category are as follows:
  - Very old (in service prior to 1950-60) and in poor condition: 30 %.
  - Old (between 1950 and 1980) and in fair condition: 25 %.
  - Relatively recent (in service since 1980) and in good condition: 45 %.

4.3.1.3 Flow Control Equipment

Observations and testing of the flow control equipment are discussed in Section 4.4.2 - Main Dam – Flow Control Equipment Testing.

4.3.1.4 Geotechnical Aspects

The Main Dam is constructed on a rock foundation. The site inspection at this dam indicates that:

- No erosion has been observed on the Main Dam or lock abutments at the contact of the concrete structure and rock foundation.
- No seepage has been observed around and at the downstream side of the dam.
- No ground settlement or signs of piping conditions have been observed at the Main Dam.
- There is no excessive vegetation around this dam or nearby structures that would indicate excessive seepage or moisture.

4.3.1.5 Hydrotechnical Aspects

The flow at some sluices on the left side of the Main Dam is impeded by the presence of rock outcrops. Elevations were recorded during the surveying campaign. Photos 1, 38 and 39 of App. B1 show the view of the downstream and upstream obstacles that impede flow. It can be seen that:

- Exposed bedrock directly downstream of sluices 7, 8 and 9 is an obstacle to flow.
- Though submerged, the high rocks located upstream of sluice 9 are a possible impediment to flow.

Dam discharge capacity and the effects of the obstacles on the flow are discussed in Section 7.1 - Dam Hydraulic Capacity.

4.3.2 Lock and Upstream Shoreline Wall

Photos presenting the observations that were made by each discipline are presented in Appendix B2 – Pictures of Lock 45. Photo 1 of App. B1 shows the general view of the structures, including Lock 45.

4.3.2.1 Structure and Civil Works

Visual inspection was performed from the walkways. An underwater camera inspection was performed on the left upstream wall common to the intake of sluice 1 of the Main Dam. The following observations were noted:

- A large crack (5 mm) is noted at the top of the stairs (Photo 2 of App. B2). It causes leakage in the lock chamber (Photo 3 of App. B2) leading to standing water on the steps, algae growth and, in wintertime, icy patches.
- The horizontal construction joint (located approximately 1 m below water level, which was 180.25 m during the inspection) and the vertical construction joint (located at the top of the stairs) are open and may be the cause of the leak in the wall (Photos 3 to 6 of App. B2), discharging into the lock chamber.
- A large crack is located at the end of the downstream wall on the reservoir side (Photo 7 of App. B2). The crack starts at the crest of the wall and continues below the water surface.
- The large crack located at the top of the stairs causing leakage in the lock chamber is located at the vertical construction joint, which is open on the entire length that was inspected with the underwater camera (Photos 8 and 9 of App. B2).
- The horizontal construction joint located approximately 300 mm below water level is deteriorated between the downstream end and the opening mechanism of the lock gate (Photo 10 of App. B2).
- All construction joints are slightly open. At some locations, zebra mussels are growing in large numbers around openings.

The concrete walls of the lock are in good condition except for the left upstream wall, which is common to sluice 1 of the Main Dam.

A portion of the upstream shoreline wall appears to have been rebuilt or repaired above the water surface. This is located between the lock canal and boarding dock and is in good condition (Photos 12 to 14 of App. B2).

The remaining portion of the wall above the water surface shows a few minor cracks on the right end of the crest but is also in good condition (Photos 15 to 17 of App. B2). The wall is founded on bedrock above the water level at the right end.

4.3.2.2 Mechanical Works

Lock 45 and the Upstream Shoreline Wall do not include any flow control equipment which plays a role in passing the IDF. Since the lock is not included in the assessment of the dam's capacity to pass the IDF, and as directed by PWGSC during the preparation of the Phase II Project Proposal, a mechanical inspection of the Lock 45 operating equipment was not performed by AECOM as part of this dam safety review.

A functional test, involving the opening and closing of the west upstream gate as well as opening and closing the valve in the west downstream gate, was performed. It was demonstrated that the installation is in an acceptable working condition.

4.3.2.3 Geotechnical Aspects

The site inspection at this structure indicates that:

- No erosion has been observed on the lock abutments at the contact of the concrete structure and rock foundation.
- No seepage has been observed around the lock and the upstream shoreline wall.
- Sinkholes have been observed near the Upstream Shoreline Wall. These sinkholes have been filled recently with sand and gravel (Photos 22 and 23 of App. B2).

4.3.3 Dam D

Photos presenting the observations that were made by each discipline are presented in Appendix B3 – Pictures of Dam D. Photo 1 of App. B1 shows the structures, including a partially obstructed view of Dam D (on the right, behind the trees).

4.3.3.1 Structural and Civil Works

Visual inspection was performed from atop the dam. The underwater camera inspection was performed over a distance of 9 m along the wall, on the most damaged part of the wall, between Main Dam and the boat, and all along the dam for spot checks. The following observations were noted:

- Some minor cracks have been observed at the top of the wall (Photo 1 of App. B3).
- There is a significant degradation of the concrete in the drawdown zone. Notably, a piece of concrete, with an average thickness of 300 mm, has broken away from the structure in the drawdown zone near Main Dam (Photos 2 and 4 of App. B3). The thickness of the deterioration could not be assessed without performing core drilling.
- A large crack is also present at the junction of Dam D with the Main Dam (Photo 3 of App. B3).
- The submerged concrete surface was covered with algae, making any observations of the concrete condition impossible (Photo 5 of App. B3).
- A drainage pipe goes through the base of the dam (Photo 6 of App. B3).
- Presence of small cracks with minor efflorescence on the top part of the dam (Photo 7 of App. B3).

4.3.3.2 Geotechnical Aspects

The site inspection at this structure indicates that:

- There are two sinkholes (Photos 8 to 10 of App. B3) on the downstream side of Dam D. The sinkholes are located 1.5 m and 0.3 m away from the concrete wall and both have a diameter of approximately 0.2 m. They seem to result from erosion occurring due to the deterioration of the concrete face wall (Photos 11 and 12 of App. B3), which allows underground seepage and transportation of soil particles.
- The compactness of the embankment on the downstream side of the concrete wall is loose. This implies that other sinkholes may appear with time.

4.4 Current Operation Procedures

At the Port Severn Main Dam, two structures can be operated: the Main Dam and Lock 45. The Main Dam is operated to manage water level and can also be operated to discharge floods if necessary. Lock 45 is solely operated to allow the passage of boats. Dam E and Dam G are also structures, part of the Port Severn Dams that can be operated to manage water levels. The operation procedures at these dams are discussed in their specific DSR reports.

The information on current operation procedures is obtained from observations, interviews and review of the TSW SO.

4.4.1 General Operation at the Port Severn Dams

During the site visit, the staff answered questions about their experience and ability to operate the dams under normal conditions and when facing specific issues.

4.4.1.1 Staff Capabilities

At the time of staff interviews, the operation crew was made of six operators. By July 2013, the staff had been reduced to four experienced operators, working all year long, and one part-time employee available during the summer months (four months). As such, two teams of two would be made to operate the dams. During significant floods, a third team could be made up of the seasonal employee and the Sector Manager.

The four persons operation crew is based at the Washago shop and has multiple dams at six sites / reservoirs to operate under its responsibility. These are, from Georgian Bay to Lake Simcoe:

- Port Severn Dams and Lock 45.
- White’s Portage Dam (including Six Mile Lake Main Dam, Six Mile Lake Ravin Dam, Hungry Bay Dam and Crooked Bay Dam).
- Big Chute and Lock 44, Pretty Channel Dams and Lost Channel Dam.
- Swift Rapids Dams and Lock 43.
- Washago Dams and Lock 42.
- St. John Creek Dam (not on the TSW and requires minimal operation).

During a flood, the crew would be expected to operate all six dam sites under their responsibility, as required. The shop in Washago is a 1-hour drive away from the Main Dam. Also, at Port Severn, the lock master’s building and visitor center is located on the right bank beside Lock 45.

The following information about staff capabilities applies to all the dams at Port Severn. It was gathered during the interviews. Personnel changes that have happened since then (from six full-time employees to four full-time and one seasonal) are not reflected:

- Experience: among the maintenance crew, two have more than 25 years' experience, one has between 10 and 15 years, and one has 5 years. Among the lock operators, three have more than 25 years' experience and nine have between 5 and 10 years.
- During navigation season (May to October): Two lock operators who are trained to operate the dam are on site throughout the day at the Port Severn Dam. In addition, five to six maintenance employees are on duty and based at the Washago shop.
- During freshet: The lock is shut down and two operators are assigned to the Port Severn Dam.
- During winter (October to May): The five or six maintenance employees based in Washago can operate the dam.
- Operators manage water levels by adding or removing one log per sluice. The movement of more must be declared to the Operations Supervisor.

4.4.1.2 Winter Conditions

- Ice jams have occurred, especially in springtime. They occur when a quick melt generates ice blocks, which hit the pier noses.
- Average ice thickness over the past years is measured at 24 to 30 inches (60 to 75 cm).
- Normally the ice cover forms at the beginning of December, and stays in place up to late March, sometimes to mid-April. Ice cover may form as early as November and as late as January.
- Ice or frazil ice needs to be chipped away from the stoplogs before they can be removed. Once this is done, the same number of stoplogs as in normal conditions can be removed. When the stoplog gains are frozen, a pike pole is used to break the ice.
- Frazil ice is a problem. When frazil ice is present it is usually necessary to use the manual winches instead of the log lifter, since it is not possible to feel the recessed steel handling rings at both ends of the logs.
- The dam deck is maintained in a clear and open condition especially in the winter season.
- Snow and ice accumulation on the deck is removed on a regular basis (Photo 34 of App. B1).
- De-icing is performed using steel tools. No hot steam generating machine is available for de-icing. Tiger torches or similar tools are used effectively to melt ice when necessary.

4.4.1.3 Sluice Obstruction

- Ice jams happen, especially in springtime. They occur when a quick melt generates ice blocks, which hit the pier noses. Photo 35 of App. B1 shows an iced sluice.
- Trees and other debris also sometimes get jammed in the bays.

4.4.2 Main Dam – Flow Control Equipment Testing

At the Main Dam, flow control equipment testing was performed to evaluate the capability of the staff to operate the facilities. This further complements the information gathered by observing the TSW staff perform regular operations, by interviewing the operators and through document review.

An operation test was performed during the site inspection of December 2011 by removing and inserting some stoplogs in the control bays. The following issues were encountered:

- The bottom logs have never been removed by any of the operators. It is unknown whether they can be removed or not.

- TSW staff was unable to perform an operation test on the date of the site inspection since this process is labor intensive.

A second visit was carried out in November 2012 to conduct a full flow test. Nine stoplogs were already removed, the test aimed at removing the last three. After 3 hours of attempts using the log lifter and the manual winches, operators were unable to remove any of the bottom logs.

The bottom logs are under high head and water velocities. This causes the logs to bounce or “chatter”. It is also impossible for the operators to see the deepest logs. This makes it very difficult to hook the log lifter or manual winches into the dees.

4.4.2.1 Sluices Operation

- Sluices 7-8-9 have not been opened in 27 years.
- Sluices 7-8-9 sit on land such that opening them would flood property and lead to soil erosion.
- A Hydro-One post sits immediately downstream of sluice 9 such that opening may damage the distribution line.
- Sluice 1 is rarely opened since it creates disturbances to the navigation leaving the lock.
- Stoplogs that are not often used are old and have deteriorated. Removal of the stoplogs in these sluices may require the use of tongs/winches and the total time to fully open the sluices may increase. Log breaks add an extra 30 to 45 minutes for removal. This can be a hazard when urgent reservoir discharging is required. Damaged stoplogs include:
  - The bottom four or five logs of sluices 2, 3, 4, 5 and 6.
  - All logs of sluices 1, 7, 8 and 9.
- Submerged logs are difficult to remove as they bounce or “chatter”. Problematic logs refer to logs 9 to 12 in sluices 2 and 3, logs 9 and 10 in sluices 1, 3 to 9, and log 9 in Dam E.
- According to operators, removal of the bottom three stoplogs would be possible but cause them to break (therefore, it was not done during the full flow test).

4.4.2.2 Hydraulic Log Lifter

- The operator utilized the log lifter to lift and set the logs in place.
  - The hydraulic log lifter is suitable to add and remove logs in sluices 2 to 8.
  - The hydraulic log lifter cannot be used over sluices 1 and 9.
- The embedded steel angle (edge protection of gains) causes interference when inserting the stoplogs in the operation gains.
- When stoplogs get jammed in the gains or when dees are broken, logging tongs are used. Photos 37 and 43 of App. B1 respectively show the regular hooking onto a log dee and the use of tongs when the dee is broken.
  - The use of tongs also delays (unknown time) the removal of logs.
- Use of the hydraulic log lifter to push the logs into position and seal the gaps, as per TSW SO 7.7, is insufficient.
  - Operators regularly resort to using a hydraulic jack, a method developed for use when using the manual winches, as per TSW SO 7.7.
  - It is placed on the top stoplog and a steel cross beam is placed underneath the deck at each end of the sluiceway. Operators on the deck pump the jacks (Photo 32 of App. B1).
- Using the log lifter, logs in good condition can be removed at a rate of ± 5 minutes per log (removal of the first eight logs took 30 minutes; bottom logs could not be removed).
  - Moving and setting the winches over a sluice takes about 10 minutes.
  - Removal of lower level logs may be slower, depending on the weather condition, the water level and the stoplogs condition.

4.4.2.3 Backup Winches

Two manual winches are used as a back-up for the log lifter. One unit is required at each end of the sluiceway. The manual log lifters which comprise the backup lifting system seem to be in acceptable working condition. TSW staff mentions the following issues regarding the use of the manual winches:

- The winches were used no later than summer 2012. They are notably used during the summer season when maintenance is carried out on the log lifter.
- A minimum of two operators is needed to change logs with the manual winches, though more operators would accelerate the work.
- The winches have to be hand cranked.
- The operators attach the lifting hooks to long pike poles in order to insert the hooks in the log dees.
- It is not unusual for TSW staff to encounter difficulty hooking the logs when they are submerged and not visible.
- The manual method requires the use of peaveys to handle and to clear the logs on the dam deck and then for replacing them in the gains.
- When stoplogs get jammed in the gains or when dees are broken, logging tongs are used.
  - The use of tongs also delays (unknown time) the removal of logs.
- Operators use the hydraulic jack, as per TSW SO 7.7, to seal the gaps between logs.
  - It is placed on the top stoplog and a steel cross beam is placed underneath the deck at each end of the sluiceway. Operators on the deck manually pump the jacks.
- Since the purchase of the log lifter, the manual winches have become the backup system at the Main Dam. Manual winches are still the primary equipment at Dam E and Dam G. Back-up for the manual winches at Dam E and Dam G is transported by truck.
  - Moving and setting the winches over a sluice takes about 10 minutes at Main Dam.
  - Moving and setting the back-up manual winches to Dam E or Dam G takes approximately 1 hour.
- As apparent on Photo 41 of App. B1, the hydraulic log lifter and the manual winches cannot cross each other on the rails.
  - To bring the manual winches to sluice 8 and 9, they must be lifted from the rails using a pole as a lever.
  - This adds approximately 30 minutes.
- Use of the manual winches is physically demanding and hazardous, especially in adverse weather conditions.
- Using the back-up winches (manual), logs in good conditions can be removed at a rate of ± 30 minutes per log.
  - TSW staff advises that it could take an entire day to remove all the stoplogs from just one sluice, due to the foregoing operational difficulties.

4.4.3 Lock 45 and Upstream Shoreline Wall Operation

Lock 45 is not operated to discharge floods or manage water level. The lock gates are solely operated to allow the passage of boats. Operation of the lock gates was observed mainly to identify safety concerns and verify compliance to the TSW SO. As such, most observations concerning the operation of Lock 45 are found in Section 4.6 – Safety.

A functional test was performed, involving the opening and closing of the west upstream gate as well as opening and closing the valve in the west downstream gate, and demonstrated that the operation is done efficiently and safely.

Operators have explained that, as a safety precaution, the waterway is closed when flows reach 90 m^3/s.

4.4.4 Dam D Operation

There are no movable parts or equipment at Dam D. Dam D does not require any operation.

4.5 Maintenance

Maintenance activities include all tasks performed with the objectives of maintaining the work site fully operational, prolonging the facilities' lifecycle and keeping the grounds clean, safe and secure. Operators are said to participate in regular surveillance of the facilities. However, there are no indications of record keeping.

4.5.1 Main Dam Maintenance

The Lockmaster and Damkeeper are responsible for the operation of the dam and ensuring its safety, security and good appearance. The SO 8.5 – Custodial Care of Control Dams specifies the minimum standards of custodial care. The operators at Port Severn are said to follow the SO. No deficiency has been noted.

Tasks are done at different periodic intervals and presented below. It is unclear whether a log book of maintenance activities is kept.

It is sometimes required to climb on the log lifter for maintenance activities, such as accessing the hydraulic lines running to the top of the telescopic rams. A ladder is used to do so.

There are no spare parts and backup equipment available on site.

4.5.1.1 Dam Deck

Maintenance of the dam deck aims at keeping the surface safe for the workers to perform their duties. According to the operators:

- The dam deck is maintained in a clear and open condition especially in the winter season.
- Snow and ice accumulation on the deck is removed on a regular basis (Photo 34 of App. B1).

4.5.1.2 Stoplogs

Maintenance of the stoplogs means replacing broken or badly damaged logs.

- Typically about six stoplogs per year break for the entire sector (approximately 1% of the sector's logs). Causes for failure include:
  - Wear induced by the occurrence of "chattering" (vibration of the stoplogs).
  - Damage caused by the use of logging tongs.
  - Loads.
- Beyond log failure, many more logs must be replaced as their poor condition (usually broken dees) makes for difficult operation and poses a risk to flood discharge capacity.
- When a log splits the removal operation causes a delay of about 30 to 45 minutes.
- The maintenance gains are shut for removal of damaged stoplogs from the operation gains.
- It is believed that there are still original logs in bays 7, 8, and 9.

40 new stoplogs were received in 2011 for the whole sector, which is exceptional: normally 24 stoplogs per year are received. This is not enough considering that the life expectancy is shorter for new stoplogs than it was 40 years ago. It is estimated that at least 30 new logs per year are necessary to replace the older damaged logs.

4.5.1.3 Log Lifter

Maintenance at the log lifter aims at keeping the equipment in good functioning order and consequently prolonging its lifecycle. It includes the verification of all fluid levels and possible leaks and the visual inspection of all components. It is important that the unit never runs out of fuel because it may induce damage to the carburetor. Concerning refuelling:

- The fuel level is checked every time the log lifter is used. A dip stick is used as there is no fuel gauge.
- The fuel reservoir contains about 3 to 3.5 gallons.
- A 3-gallon fuel tank is stored at the opposite end of the unit and a 5-gallon tank is stored in the building adjacent to the dam.
- The unit is refuelled from its current location on the deck simply by pouring fuel from the tank into the reservoir.
- While there are risks of accidental spills, staff have indicated that they are very rare and limited in volume to a few drops that are wiped off with a cloth.

The following maintenance activities are done on a daily basis:

- Checking the oil and other fluids.
- Start-up.

The following maintenance activities are done on a monthly basis:

- Visual inspection of hydraulic lines, telescopic rams, and hydraulic controls.
- Oil changes (not necessarily every month).
- Wear and damage inspection.

4.5.2 Lock 45 and Upstream Shoreline Wall Maintenance

Lock 45 is not operated to control flow or manage water level. The lock gates are solely operated to allow the passage of boats. Maintenance of the lock gates was therefore observed only to identify safety concerns and verify compliance to the TSW SO. As such, most observations concerning the operation of Lock 45 are found in Section 4.6 – Safety.

The following maintenance activities are done at irregular intervals, when required:

- Cleaning the steps at the left upstream wall of Lock 45, removing standing water, algae and ice, caused by the large cracking.

4.5.3 Dam D Maintenance

PCA staff do not perform any regular maintenance at Dam D. Landscaping (cutting the grass) is sometimes being performed by private citizens.

4.6 Safety

4.6.1 Occupational Health Safety (OHS)

During the site visit, an OHSA auditor was present and made observations. TSW staff was required to perform operations while OHSA expert studied their methods, as relates to public safety and occupational health and safety aspects. Because of time constraints not all the activities could be fully performed. Activities that could not be performed were nonetheless explained and re-enacted. These include de-icing and tonging. Observations made by the OHSA expert are cited in this section.

4.6.1.1 Personal Protective Equipment

Personal protective equipment made available to and used by the operators includes:

- Life jackets or inflatable personal flotation devices (PFD).
- Full body harness.
- Lanyards.
- Self-retracting life lines.
- Beam straps.
- Hard hats.
- Safety boots.

All the required personal protective equipment (PPE) is available. However, due to the absence of engineered anchors, use of PPE to prevent falls does not respect standards.

4.6.1.2 Security Installations and Facilities Safety Equipment

Safety equipment available on-site includes:

- Life rings.
- Pike poles.
- Permanent ladder in the lock.
- Railings at the Main Dam.

The following observations were made concerning the security installations and facilities safety equipment:

- Tie-offs: there are no engineered anchors or tie-off locations on the dam or on the log lifter. Operators therefore are not properly protected against falls while working over and around open gains.
- There are no railings along Lock 45 to protect the staff when manoeuvring the locks.
- The handrails at the entrance, near the visitors' building, do not meet the safety requirements of NBCC 2005 and OBC 2006 (Photo 11 of App. B2).
- Night time stoplog manipulation is potentially hazardous as there is no lighting on the deck. Lights on the hydraulic log lifter are inadequate.
- Operator control on log lifter not clearly marked.
- Operator compartment safety gate missing.
- There are no rails or rolling strips on which to secure the removed logs, as per SO 7.1.

4.6.1.3 Training

Operators indicated that it was between five to seven years since a formal fall arrest safety training program had been attended.

No training on rescue or on work under extreme weather conditions or during flood events has been dispensed.

4.6.1.4 Operation

During the operation of the sluices and demonstration of other operations, the following observations were made:

- Safety rules are not diligently observed. When removing the gain covers or working above the open gains, operators did not wear the safety harness and were not properly tied-off.
- When going into the gain area, operators tie off to the log lifter or fence rail since there are no proper tie-off locations or engineered anchors. These are not appropriate anchoring points.
- It was observed that the process of jacking the logs with the hydraulic jack is causing damage to the under-deck. The hydraulic jack is used either with the manual winches or because the log lifter cannot apply sufficient force.
- When opening the lock gates the operator uses a large turnkey located on the side of the lock. When the operator operates the key he has to walk in circles coming very close to the edge of the lock. No fall arrest safety protection was noted.
- It is sometimes required to climb on the log lifter for maintenance activities, such as accessing the hydraulic lines running to the top of the telescopic rams. A ladder is used to do so.

Operators indicate that when log jams occur, the jacks and pry bars are used to free them. This process was not demonstrated.

The log lifter is a mechanized tool. Use of the log lifter reduces risk of injury due to physical exertion, repetitive handling or heaving lifting. However, improper, inattentive or otherwise unsafe use of mechanical equipment can lead to serious injury, such as crushed limbs, severe cuts or from impacts. Caution should be exercised and the log lifter must only be operated by trained operators.

The winches must be hand cranked and the lifting hooks manually engaged in the log dees using the pikes. Operators indicate that they regularly encounter difficulty hooking the logs when they are submerged and not visible. When cranking the winches (lifting stoplogs), a safety stop will block the cogs should the operator inadvertently release the handle or similar accident happen. There is no similar safety measure for when the logs are lowered, which poses a risk. Manual log lifting is anticipated to be physically demanding and hazardous, especially in adverse weather conditions. Minor injuries or physical exertion may occur.

4.6.1.5 Surfaces

Surfaces must be kept free of debris and safe to walk and work on. The following were noted:

- The large crack (5 mm) at the top of the stairs (Photo 2 of App. B2) causes leakage in the lock chamber (Photo 3 of App. B2). This is problematic because standing water on the steps lead to algae growth, creating a slipping and falling hazard to workers. In wintertime, the water freezes on the steps and creates a similar hazard.
- Operators have mentioned that they keep the surfaces free of snow and ice during the winter and spread salt and/or abrasives.

4.6.1.6 Station Logbook

During interviews, the dam operators have indicated that they follow the safety protocols and procedures as detailed in the standing orders. SO 9.1 – Station Logbook establishes the policy and procedures for recording events at each lock and bridge station. While no deficiency has been noted, there was no indication of annual safety inspection.

4.6.2 Public Safety

During the site inspections realised by AECOM Public Safety Specialists, in December 2011, May 2012 and April 2013 public safety aspects were considered. Observations on Lock 45, Main Dam and Dam D as well as Dam C, Dam E, and Dam G were noted.

The Port Severn site is in the centre of a well-visited tourist area and national park and is designated as a Historic Site. The setting and available facilities are attractive and hence public exposure to the hazards is high, particularly in the tourist and navigation season.

Development around the area seems to have proceeded unchecked over the years, resulting in several facilities, public and private, surrounding and abutting the structures: e.g. marina and boat storage at Dam D and Dam G, boat docks jutting out of Dam D, use of the main dam as a public walkway, residences and developments downstream of the Main Dam and Dam G within what is likely the flood plain and the zone of influence of the dams. These situations increase the public exposure on an ongoing basis.

Given their age and construction era, hazards to the public were not systematically “designed out” of these structures and sites. Later attempts at mitigating some risks (e.g. guard rails, guard wires, fences) do not meet current codes or do not entirely address the existing safety issues. From conversations on site, it appeared that existing safety mitigations have been added in a reactive mode, when incidents happen, rather than in a studied and planned approach. From boaters’ perspective, different signs and safety booms exist but are not sufficient or adequate as per CDA Guidelines to fully ensure their security.

Before dam operations are performed, operators must verify that no one is immediately upstream or downstream of the dam. Only visual observation is performed. Operators mention that increase in flow is not rapid since stoplogs may only be removed one at a time, at intervals of about 3 minutes.

The detailed findings and observations regarding public safety are presented in the Public Safety around Dams Risk Assessment Report (Reference 3). The final version was submitted to PCA in August 2013.

4.7 Review of Existing Policies, OMS, EPP and ERP

As of March 2013, there are no existing Operation Maintenance and Surveillance manuals (OMS), Emergency Preparedness Plan (EPP) or Emergency Response Plan (ERP). All dams must have an OMS manual. EPP and ERP are required if lives are at risk or if implementation of emergency procedures can reduce the potential consequences of dam failure.

4.8 Summary of Deficiencies

From the anomalies noted during the site visit, through interviews and from document review, deficiencies have been identified. The following section summarizes all the deficiencies.

4.8.1 Main Dam

The following deficiencies on the structure were noted:

- Concrete deterioration
  - Some reinforcement bars of the beams supporting the deck are apparent and rusted.
  - Pieces of concrete on the piers in the drawdown zone have broken away.

  - Concrete is damaged mainly at nose of piers, underside of deck and at joints on top of the deck.
- Cracking and open joints
  - Large cracks are present at some locations on the piers in the drawdown zone
  - Two very large cracks (approximate width of 25 mm) were observed on either ends of the Main Dam: one at the intersection of Dam D, and a second in the wall common to the lock.
  - A large crack (approximate width of 5 mm) was observed between the noses of the piers and the deck, at the joint at pier centreline.
- Gains deterioration
  - At some gains, the top ends of the steel armouring angles are delaminated from the concrete, making it difficult to insert the stoplogs.
  - Damage at the gains hinders insertion of logs.

The following deficiencies on the equipment were noted:

- Log lifter
  - Sideway adjusting rails on the log lifter are damaged.
  - The log lifter, in its current setup on the dam deck rails, is unable to exert sufficient force to seal the gaps between logs.
  - The log lifter appears reliable to operate the top eight logs during flood events, but not the lower logs.
- Dam deck rails
  - Rail bed bolting and dam deck rails are rusted and damaged causing rail to shift.
  - The log lifter cannot reach sluices 1 and 9.
  - Refuelling of the log lifter poses an environmental risk associated with spilled fuel entering the river. No spill protection is used.
- Logs
  - 30 % of the logs are in poor condition. Dees are missing on many logs.

Other deficiencies noted include:

- Exposed bedrock outcrop immediately downstream of sluices 7, 8 and 9 impedes flow if the sluices are open.
- A house and a Hydro-One post lie in front of sluice 9.

4.8.2 Lock 45 and Upstream Shoreline Wall

The following deficiencies on the structures were noted:

- Cracking and open joints
  - A large vertical crack (5 mm) is noted at the top of the stairs, causing leakage.
  - The horizontal construction joint (below water level) and the vertical construction joint (located at the top of the stairs) are open and may contribute to the leak.
  - A large vertical crack is noted on the left wall immediately upstream of sluice 1.
  - A large piece of the left wall bounded by cracks seems to be unstable and has detached from the wall.
- Deteriorated concrete
  - Concrete surfaces in the drawdown zone of the Upstream Shoreline Wall have deteriorated.

The following deficiencies on the operator walkway were noted:

- When the gates are open, the walkway extension for public passage at the upstream gates hinders the operation of the gate mechanism handle.

- Two wooden protective covers are missing from the downstream face of the west gate above the water level.

The following deficiencies concerning soil stability were noted:

- Presence of sinkholes at the Upstream Shoreline Wall, though recently filled with sand and gravel, suggests that water seeps through the degraded wall.

4.8.3 Dam D

The following deficiencies on the structure were noted:

- Concrete deterioration
  - There is a significant degradation of the concrete in the drawdown zone, allowing underground seepage and transportation of soil particles.
- Cracking and open joints
  - A large crack is present at the junction of Dam D with the Main Dam

The following deficiencies pertaining to soil stability were noted:

- The compactness of the embankment on the downstream side of the concrete wall is loose.
- Two sinkholes resulting from underground erosion due to deterioration of the upstream concrete wall were identified downstream of the dam.

4.8.4 Operation Procedures

As pertaining to the operation procedures, the following were found lacking:

- Operating procedures under wintery conditions is not covered in the SO.
- Use of a hydraulic jack to seal the gaps between the logs in lieu of the log lifter is not a favored option. This method is usually reserved for when using the backup equipment. It also damages the under-deck since there are no trusses specifically tailored to this activity present. In this case, it is necessary because the log lifter does not exert sufficient force on the logs.
- The bottom logs (log 9 and below) cannot be relied upon for gate opening given that the operators were unable to remove these during the flow tests.
- Insufficient staff: There are no experienced backup operators.

4.8.5 Maintenance

As pertaining to maintenance, the following deficiencies were noted:

- There are no spare parts available on site.
- The maintenance of the logs is not adequate: bottom logs and logs in less solicited sluices are not adequately maintained.
- Annual rate of replacement of the logs (number of new logs to number of broken logs) is not sufficient.
- Because of their degraded state and difficulties to hook onto their dees, the bottom logs are not regularly removed. Operation of each log and position rotation is part of proper stoplog maintenance.
- It is unclear whether a complete log of events is kept.

4.8.6 Safety

4.8.6.1 OHS

As pertaining to the OHS, the following were found not to meet requirements:

- Inadequate equipment and facilities
    - Operator compartment safety gate missing.
    - Capacity and data plate missing.
    - Operator control not clearly marked.
    - Night time stoplog manipulation is potentially hazardous as there is no lighting on the deck.
    - There are no railings along Lock 45 to protect the staff when manoeuvring the locks.
    - There are no engineered anchors or tie-off locations on the dam or on the log lifter.
    - Leakage at the top of the stairs at Lock 45 makes surfaces slippery and thus creates a potential falling hazard for operators and the public.
- Unsafe behaviour
    - Proper safety equipment is not worn diligently: fall-arrest equipment, life jackets, hard hats are not always worn when required.
    - Operators tie-off to fence posts or other non-adequate structures since there are no adequate tie-off locations.
- No indication of annual safety inspection.
- Safe working procedures under wintery conditions are not indicated in the SO.
- Safety training is not given frequently enough.

4.8.6.2 Public Safety

As pertaining to the public safety, there were many deficiencies that were observed and need to be addressed. The deficiencies are mainly related to signage, safety booms, fencing/railing, pedestrian access, public education, ladders for helping escaping the danger zones, unsecured mechanical equipment, lack of life rings and operational procedures

The detailed deficiencies and recommended corrective measure regarding public safety are presented in the Public Safety around Dams Risk Assessment Report (Reference 3).

5. Hydro-Technical Study

5.1 Description of the Watershed

The Ontario Waterways Unit of Parks Canada Agency owns and operates the TSW, which is a 386 km waterway that extends from the Bay of Quinte on Lake Ontario at the City of Quinte West (Trenton) in the south, to the Georgian Bay (Lake Huron) at Port Severn in the north. From Lake Ontario the waterway rises from an elevation of ±75.6 m to an elevation of ±256.3 m at Balsam Lake and then descends to an elevation of ±176.8 meters at Georgian Bay.

The waterway traverses two major watersheds, the Trent River Watershed and the Severn River Watershed. Port Severn is located within the Severn River Watershed. The Severn River Watershed drains an area of over 6,000 km². Included in this watershed are the Canal Lake – Talbot River system, the Holland River, the Lake Simcoe-Couchiching basin, the Black River and the channels of the Severn River below the hamlet of Washago (Reference 3).

Of the 143 dams and other retaining structures, owned and operated by the Ontario Waterways Unit to manage water levels and flows in the TSW, 43 are located in the Severn River Watershed.

The principal tributary of the Severn River is the Black River which flows into the Severn River a short distance from the outlet of Lake Simcoe-Couchiching. Downstream of the confluence of the Black and Severn rivers lies the Lower Severn sub-basin, which has a drainage area of 824 km².

The waterway, including its tributary lakes and rivers, is an important economic, environmental and recreational resource used by thousands of boaters, shoreline residents, businesses and vacationers every year. It also provides water for power generation, municipal water supplies and agriculture and supports a tremendous variety of fish and wildlife.

Appendix C shows the general plan of the Severn River, Lake Simcoe and Black River Watersheds, accompanied by its Streamflow Synthesis and Reservoir Regulation Model (SSARR) flow chart. Both are excerpted from the Severn River Watershed Hydro-Technical Study – Flood Flows Estimate (Reference 3).

5.2 Statistical Flows and Assessment

The Hydro-Technical Study aims at determining the flood flows and hydrographs at the Port Severn Dams. The flow estimates from this study are here presented. For full methodology and detailed results, please refer to the complete document (Reference 3).

The Flood Flows Estimation Study focuses on the estimation of flood flows ranging from the flood having a return period of 2 years to the probable maximum flood (PMF) and is intended to support dam safety reviews for selected dams within the Severn River Watershed.

Six study nodes were selected for the hydro-technical study of the Severn River Watershed. The node 10 represents the Port Severn Dams. Results from the statistical analysis were computed using the available flow records within the watershed, for return periods up to 100 years.

The PMF was assessed using the SSARR (Streamflow Synthesis and Reservoir Regulation Model, U.S. Army Corps of Engineers) model for both the summer-fall and the spring floods. For the spring PMF, two scenarios were analyzed, the first one based on the spring probable maximum precipitation (PMP) combined with the melting of the 100-year snow accumulation, and the second scenario based on the melting of the Probable Maximum Snow Accumulation (PMSA) combined with the 100-year spring rainfall. The leading PMF scenario for Port Severn corresponds to the spring scenario 1 (spring PMP combined with the melting of the 100-year snow accumulation).

Because estimating an extreme event such as the 1,000-year flood based on considerations of probability (statistical analysis) may induce uncertainties, the flood flows having return periods of 500, 1,000 and 10,000 years were estimated based on interpolation between the 100-year flood and the PMF.

Both statistical peak flows and PMF hydrographs already include the routing capacity effect of the upstream reservoir system, and hence further flow reduction due to routing should not be considered.

Table 5.1 and Figure 5.1 present a summary of flood flows estimated for the six study nodes.

Table 5.1 Flood Flows for the Severn River Watershed

Flood	Extreme Flood Flows (m^3/s) at Study Node					
	2 Talbot River Dam	3 Washago	4 Black River	7 Swift Rapids	9 [(1)] White's Portage	10 Port Severn
Drainage area (km^2)	*308*	*3,707*	*1,544*	*5,856*	*5,969*	***6,132***
Statistical and Extreme Flood Flows (m^3/s)						
PMF	*350*	500	1,520	2,350	2,370	**2,510**
10,000 years	231	300	760	1,058	1,067	**1,130**
1,000 years	144	185	380	470	474	**502**
500 years	123	165	304	376	379	**402**
100 years	83	138	210	269	274	**288**
50 years	70	129	200	263	268	**281**
25 years	64	121	188	255	260	**273**
20 years	60	118	184	252	257	**269**
10 years	52	107	171	241	246	**258**
5 years	43	95	156	227	231	**243**
2 years	30	74	128	195	199	**208**
Other Flows for Dam Safety Review Purposes						
1/3 between the 1000-year flood and the PMF	213	290	760	1,097	1,106	**1,171**
2/3 between the 1000-year flood and the PMF	281	395	1,140	1,723	1,738	**1,841**

(1) Including Big Chute Generating Station outflow

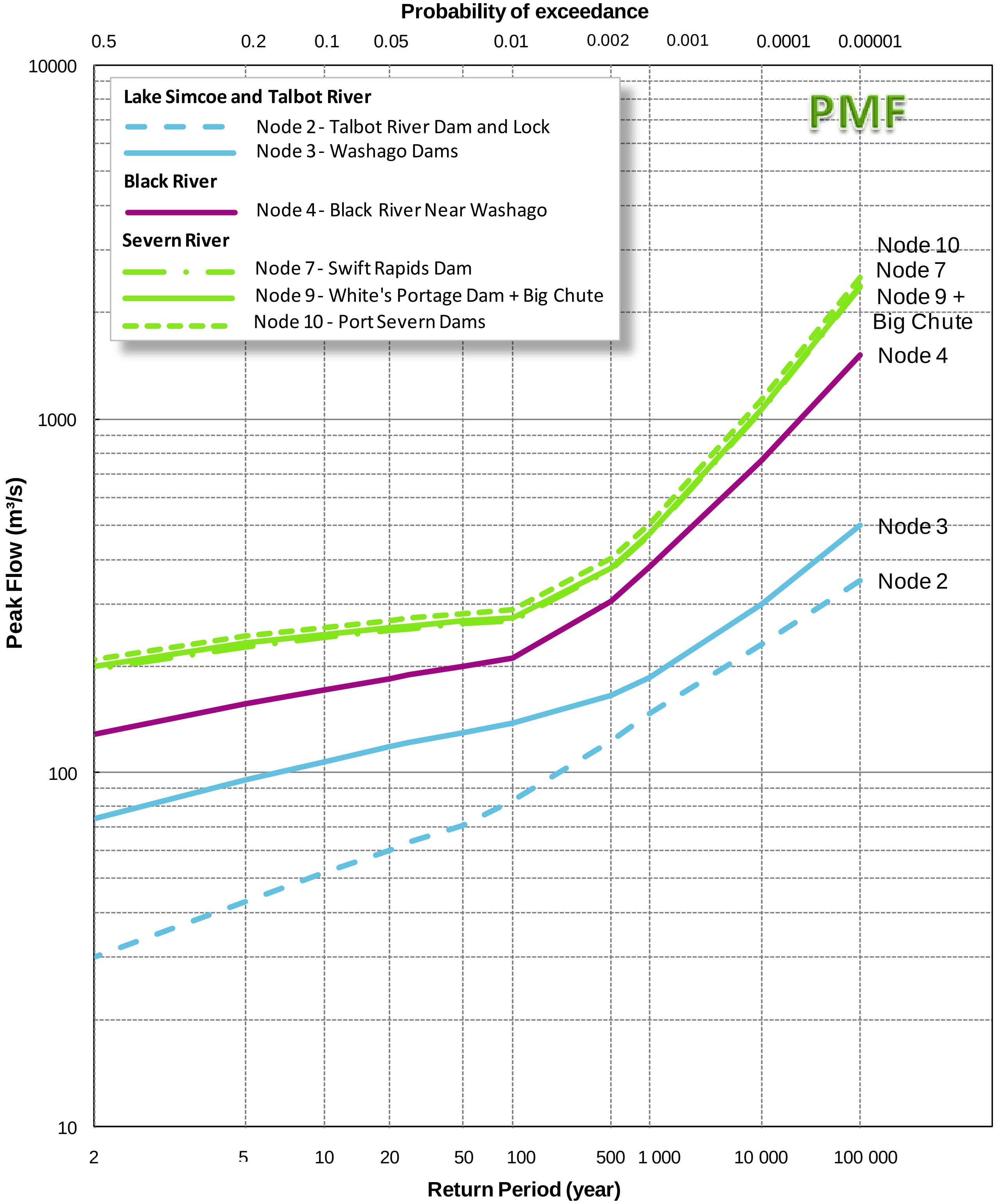

Figure 5.1 Summary of Flood Flows for all Study Nodes

It should be noted that a hydrological study has a limited 'shelf-life' as climatological conditions are constantly changing, as described in the Hydrometeorological Conditions Study Report for the Severn River Watershed (Reference 3). It is recommended to update the Hydro-Technical Study as frequently as possible, when new information becomes available; for example after the occurrence of a very large rainfall or flood, or when new advances on climate modelling become available.

5.3 Storage Curve

Gloucester Pool and Little Lake form a reservoir impounded by the Port Severn retaining structures. The storage curve describes the relationship between the volume of stored water and the elevation of the water surface. The live storage is the storage that can be discharged by operating the dam, while the dead storage corresponds to the storage below the sill of the dam sluices. The storage curve of Gloucester Pool was calculated in ArcGIS using the 3D Analyst Tools and the DEM, including bathymetry of the head pond. For a series of water levels, the volume of water impounded is the volume contained between the DEM surface and the water level. Points of water level to volume are then used to draw the storage curve.

Figure 5.2 shows the live storage capacity of Gloucester Pool. At MNOL (180.50 m) the live storage is approximately 9.3 Mm³ ($9.3x10^6$ m³).

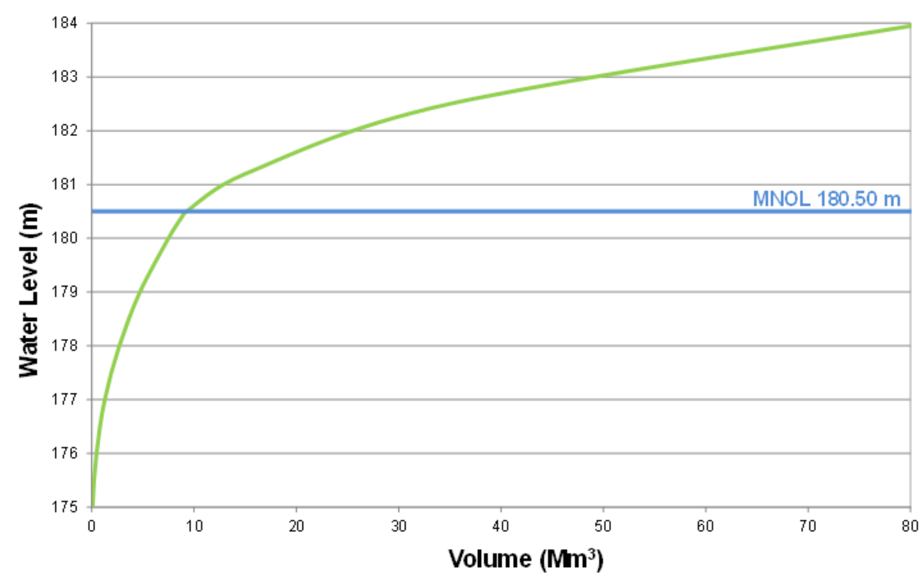

Figure 5.2 Storage Curve – Gloucester Pool

5.4 Hydrologic Conditions for Dam Break Analysis

This section describes the hydrological conditions which are assumed for the dam break analyses. Two hydrological cases are assumed for the dam break – a Sunny Day scenario and a Flood scenario.

The Sunny Day failure is a sudden dam failure that occurs during normal operations. It may be caused by internal erosion, piping, earthquakes, improper operation leading to overtopping, or another event. The Flood scenario is a dam failure resulting from a natural flood.

The flows considered here are the natural inflows/outflows discharging from Gloucester Pool / Little Lake through Port Severn. Given that all dams at Port Severn are close, the natural inflows are the same for all. The flows discharging through a given dam depend on the operation management.

The hydrological assumptions for both cases follow.

5.4.1 Sunny Day Scenario

Natural inflows of 41 m³/s are assumed for the Sunny Day scenario. This corresponds to the average flow during the navigation season for the period from 1954 to 2009. The value is based upon hydrometric data at Swift Rapids station and transposed to Port Severn.

5.4.2 Flood Scenarios

Hydrographs for floods having return periods which range from 100 years to 10,000 years and the PMF are taken from the Hydro-Technical Study (Reference 3). Table 5.2 and Figure 5.3 present the peak inflows used for the dam break analysis.

Table 5.2 Summary of Peak Natural Inflows

Flood	Peak Natural Inflows (m³/s) at Port Severn			
	100 years	1,000 years	10,000 years	PMF
Flood Flow	288	502	1,130	2,510

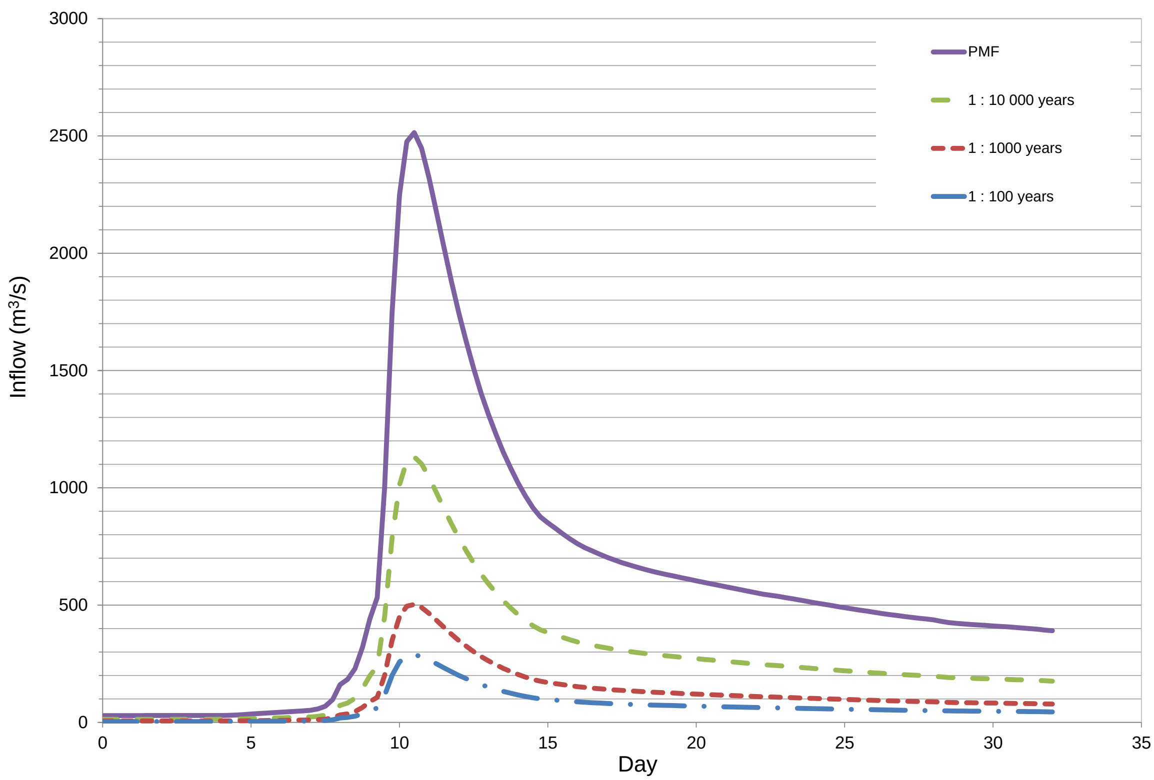

Figure 5.3 Hydrographs for Flood Events

5.4.3 Downstream Water Levels

The Georgian Bay is located downstream of the Port Severn Dams and forms part of the boundary conditions for the flood routing calculation. The water level of the Georgian Bay is defined from a statistical analysis of the recorded historical water level data.

The water level data was obtained from the Canadian Hydrographic Service (Reference 19). This provided the water level data needed to define the water levels for different return periods in the Georgian Bay. The Midland 11445 station was used as the reference location. 93 years of data were used, a period from 1918 to 2011.

HYFRAN software was used to statistically define the Georgian Bay water level according to all relevant return periods. The failure hydrologic conditions, downstream hydrologic conditions and the associated Georgian Bay water level are shown in Table 5.3.

Table 5.3 Georgian Bay Water Levels

Failure Hydrologic Conditions	Downstream Hydrologic Conditions	Georgian Bay Water Level (m)
Sunny Day	Average water level summer-autumn	176.48
100-year Flood	20-year water level	177.24
1,000-year Flood	100-year water level	177.51
10,000-year Flood	1,000-year water level	177.81
PMF	1,000-year water level	177.81

A sensitivity analysis concerning the water level in the Georgian Bay was carried out in order to determine the effect on the backwater simulations, especially in the downstream vicinity of the Main Dam following a dam breach. The sensitivity analysis shows that this has no effect on the people at risk and dam classification. Therefore, the values shown in Table 5.3 have been used and are reflected in the inundation mapping.

Results show that the influence of the downstream water level on the backwater simulations is very low (less than 10 cm in maximum water levels) for cross sections located downstream of the Main Dam. Thus, the selection of the hydrologic condition of the Georgian Bay (downstream water level at the downstream end of the model) has no effect on the evaluation of the consequences following a breach in the Main Dam.

5.5 Maximum Daily Incremental Discharge

The maximum daily incremental discharge is defined as the maximum increase in discharge which can be expected to occur over a period of 24 hours, for a flood event of a given recurrence interval. It is determined from the historical daily increments recorded at Severn River over Swift Rapids. This data set covers the period from 1954 to 2009. The approach is as follows:

- For each year, identify the maximum daily incremental discharge (difference in daily discharge between two consecutive days) and assemble a data set (56 values).
- Perform a statistical analysis, with Gumbel distribution, of the historic data to evaluate the maximum daily incremental discharge for different return periods.
- Adjust for the watershed by prorating as per the drainage area method.

Before proceeding to computing all maximum daily incremental discharge, it is important to reiterate that the accuracy of flood probability estimates based upon statistical analysis of flood data deteriorates for probabilities rarer than those directly defined by the period of systematic records. The period covered by available annual flood data is usually a few decades, and the variability is often high. Even if climate and runoff responses are assumed to be stationary, uncertainties with respect to basic data reliability, long-term representativeness of the record period, and appropriate statistical distributions for fitting and extrapolation, mean that the confidence limits on a 1,000-year or a 10,000-year estimate are usually so wide that its reliability is very low. The theoretical limit of validity of a statistical analysis corresponds to a return period of approximately twice the sample size. Since 56 years of data are available, the 100-year peak flood flows fall inside the validity range, while all floods of larger recurrence intervals have to be extrapolated.

The maximum daily incremental discharge for the return periods beyond the theoretical validity range must be extrapolated. The maximum daily incremental discharges directly extrapolated resulted in values deemed too low by the engineers. To be more conservative in the estimations, it has rather been decided to extrapolate the relation between the relation between the max daily incremental discharges and the peak flows, and then compute the incremental discharges by that same ratio. Figure 5.4 shows the peak flows, as calculated and presented in the Hydro-Technical Study (Reference 3), and the maximum daily incremental discharges.

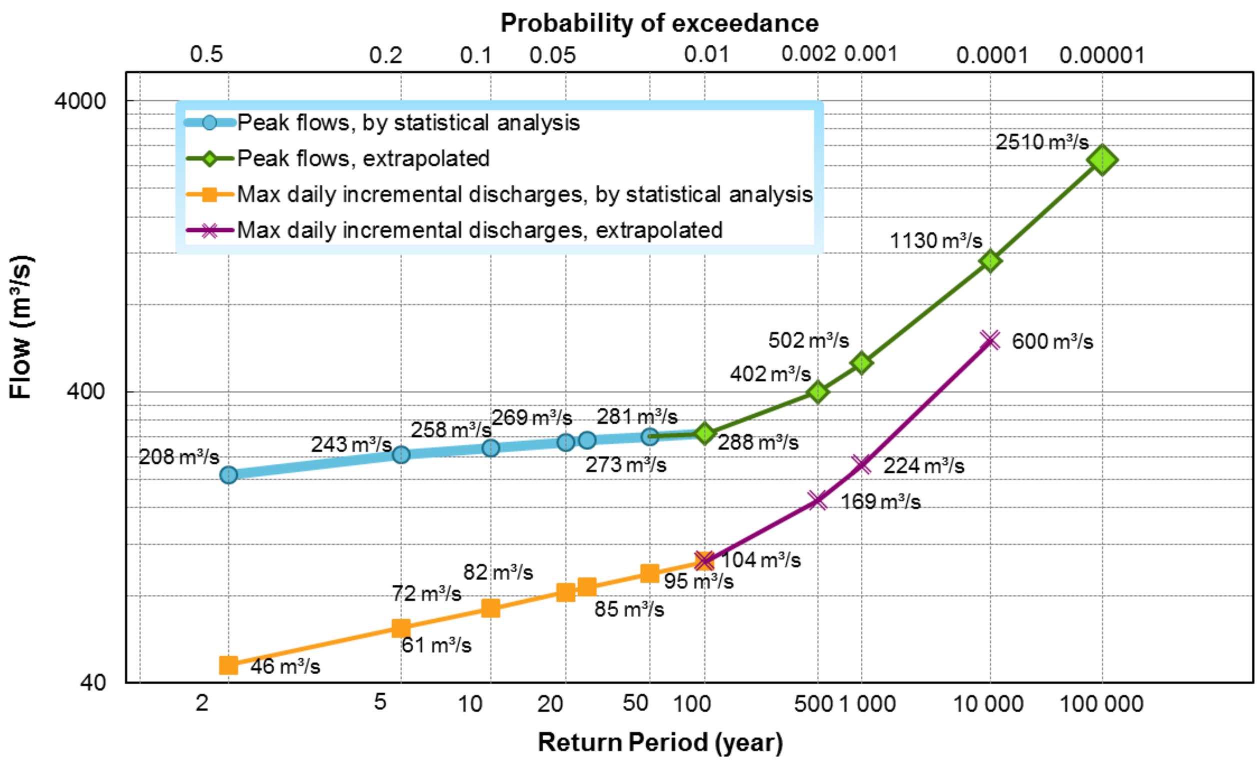

Figure 5.4 Statistical and Extreme Flood Flows – Port Severn Dams

6. Dam Classification Review

This section presents the Dam Classification Review related to the Main Dam at Port Severn. Dam classification is necessary to define, among others, the:

- Inflow design flood (IDF).
- Design basis earthquake (DBE).

Dam classification for water-retaining structures owned by PCA falls under its 2009 Directive for Dam Safety Program (Reference 49). Where not specified in the Directive, good dam safety practice as described in the CDA Guidelines (Reference 8) is used. In cases where there is a potential life safety hazard, the Directive considers the expected loss of life as the main factor for dam hazard classification.

To determine Loss of Life estimates, the USBR – A Procedure for Estimating Loss of Life Caused by Dam Failure (Reference 62) is used.

This section outlines the seven step procedure detailed in the USBR Procedure to obtain dam hazard classification. The dam hazard classification is then used to select the design criteria. An overview of the methodology and the raw data used for calculation are first presented. The workings and results for each step are then presented sequentially.

As a part of the USBR procedure, results are subject to the engineering judgement as a way to refine the estimate based on site specific uncertainty. The USBR procedure carries a few limitations and the assumptions made to overcome them are discussed.

6.1 Review of Preliminary Assigned Dam Classification

Based on the Parks Canada Directive for Dam Safety Program of Parks Canada Dams and Water-Retaining Structures, the Main Dam has been assigned a preliminary dam classification of High A, as outlined in the Project Brief. The current section reviews the dam classification.

6.2 Methodology

6.2.1 Approach to Dam Classification

The dam classification is a function of losses, categorised as follows:

- Life Safety hazard, expressed as expected loss of life (LOL).
- Environmental, cultural and heritage losses.
- Infrastructure and economic losses (property losses).

Dam classification is done as per the CDA guidelines (2007) and PCA Directive (2009). Loss of life is the primary criteria used for dam classification. The estimation of loss of life is performed following a seven-step procedure by USBR titled “A Procedure for Estimating Loss of Life Caused by Dam Failure” (DSO-99-06, Reference 62). As per the USBR procedure, an empirical-based equation or a criteria based selection must be used.

The seven step procedure defined in the USBR document is followed to classify the dam’s hazard level. Figure 6.1 shows the flow chart of tasks that have to be performed as part of the seven step procedure. All potential dam breach scenarios are studied. The worst case scenario is then used to evaluate the consequences and thus the dam classification.

From this flooding scenario, loss of life is estimated. Different methods to estimate loss of life exist. Methods considered in the estimation are reviewed and evaluated based on the specific situation at Port Severn. Following the choice of the estimation method, evacuation is analysed and using engineering judgment, potential Loss of Life estimation is confirmed.

Limitations of the procedure and the relevance to the Port Severn Dams are then discussed. Assumptions and literature research are made in order to overcome the limitations. Of particular interest is the necessity to determine the incremental consequences of dam failure, while the USBR procedure is only relevant for total consequences.

Once the Dam Classification is confirmed, it is used to select the design criteria for inflow design flood (IDF) and earthquakes.

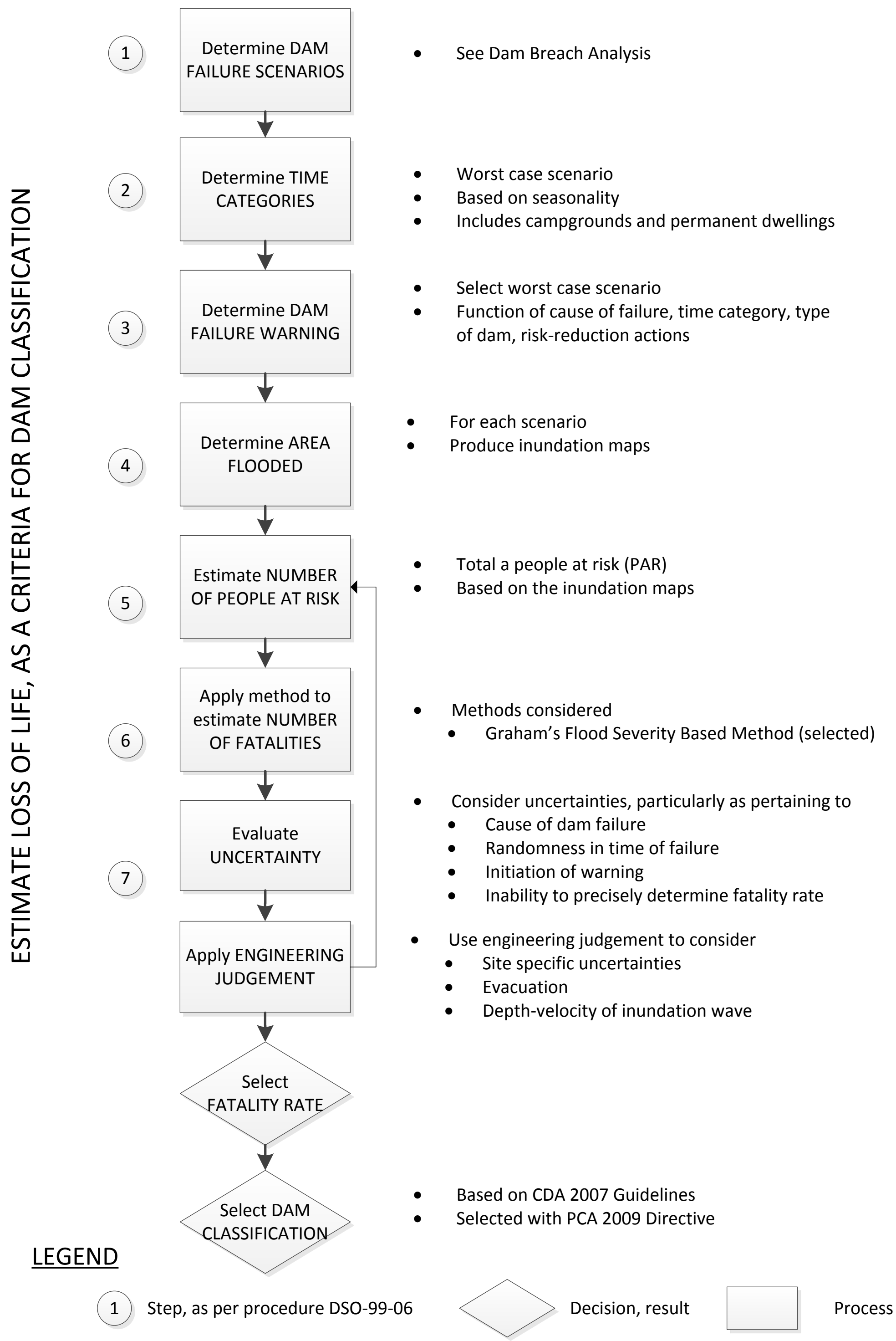

Figure 6.1 Dam Classification Flow Chart

6.2.2 Approach to Estimating Loss of Life

As per PCA 2009 Directive, the USBR procedure is used to evaluate the loss of life. The seven step approach and the results obtained are detailed in this section. Before undertaking the work, it must be underlined that, according to the procedure:

> *It is important to determine the incremental consequence of dam failure. […] For a dam failure caused by a major flood, the incremental consequence would be the additional loss caused by the dam failure over and above the loss that would have occurred if the dam and reservoir had passed the reservoir inflow without failing.*

The above statement is to reiterate that the dam owner shouldn't assume responsibility for damage that is an act of God and not caused by the failure of the owner's property. This is significant because the USBR method estimates the total loss of life, while the dam classification should be done using the incremental loss of life. The current memorandum proceeds with the estimation of the total loss of life, as per USBR Procedure DSO-99-06. Results from the procedure will then be subject to engineering judgement to obtain the incremental loss of life estimate (Section 6.10 – Apply Engineering Judgement).

6.2.3 Data

Raw data used in all the tasks performed for the dam classification cover:

- DEM of the area (Gloucester Pool to Georgian Bay), corrected based on topographic surveys carried out for this project.
- Bathymetry of Gloucester Pool and Little Lake.
- Mapping of the area downstream of the dams, including population count.
- Hydrology of the area, including flood hydrographs for various return periods and for the PMF (Table 5.1, Table 5.2 and Figure 5.3).
- Water levels in Georgian Bay (Table 5.3).
- Plans and geometry of the water-retaining structures (see Appendix A).
- Results of the site inspection (see Section 4 – Observations, Inspection and Document Review).
- CDA Dam Safety Guidelines (Reference 8).
- PCA Directive for Dam Safety Program (Reference 49).
- USBR A Procedure for Estimating Loss of Life Caused by Dam Failure (Reference 62).

6.3 Dam Break Analysis and Inundation Mapping

As part of the dam classification review, dam failure scenarios are studied. The dam break analysis for each scenario is comprised of a series of activities, as outlined in Figure 6.2. Each activity requires that the proper data be used and that adequate hypotheses be put forward. Consequently, the results of these activities serve as the input data and hypotheses for the dam classification.

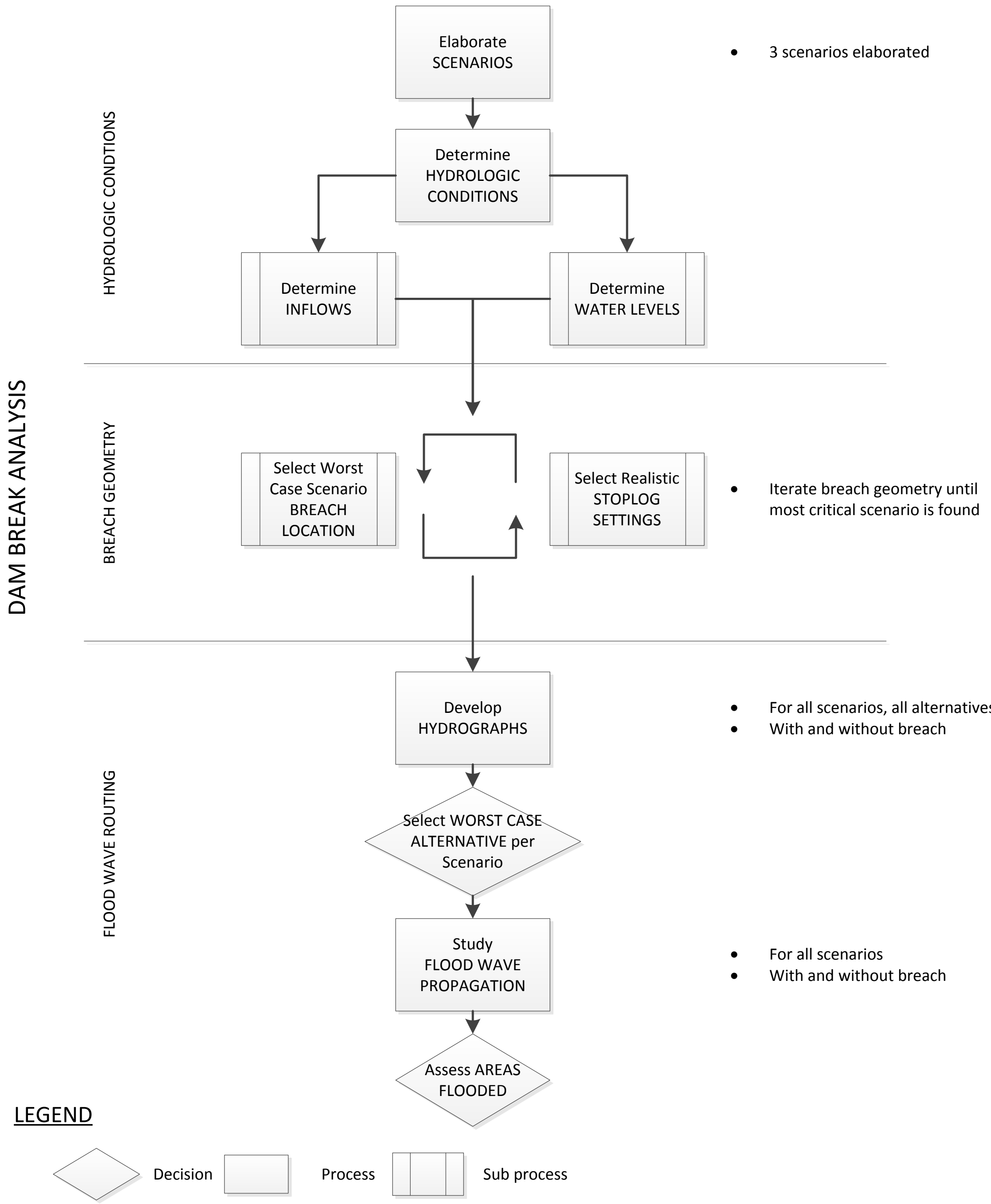

Figure 6.2 Flow Chart – Dam Failure Scenarios and Flooding

6.3.1 Hydrologic Conditions

The hydrologic conditions considered for the dam break analysis are described in Section 5.4 – Hydrologic Conditions. They are summarized in Table 6.1.

Table 6.1 Hydrologic Conditions for the Considered Dam Break Scenarios

Scenarios	Downstream Hydrologic Conditions	Peak Outflow (m^3/s) at Port Severn	Georgian Bay Water Level (m)
Sunny Day	Average water level summer-autumn	41	176.48
100-year Flood	20-year water level	288	177.24
1,000-year Flood	100-year water level	502	177.51
10,000-year Flood	1,000-year water level	1,130	177.81
PMF	1,000-year water level	2,510	177.81

6.3.2 Breach Geometry

Analysis of the dam-breach geometry is used to determine the discharge from a hypothetical breach of the dam. The resultant flood wave immediately downstream from the dam is used to estimate downstream damage and loss of life. For all scenarios, the development time of the dam breach is assumed to be instantaneous, which represents a generally used breach time for concrete structures. The location of each breach was selected in order to obtain maximum breach flow.

In the event of a lock failure, the breach will always pass a lesser flow than the scenarios studied in the dam break analysis and would not impact additional houses than those impacted by a breach in the Main Dam. Therefore, a failure at the lock is not the critical case with respect to the incremental flood analysis.

The topography and length of soil on the downstream side of Dam D makes it an improbable scenario to consider a dam break at Dam D. In the case of an overtopping at Dam D, the erosion of the soil on the downstream side would require a significant amount of time considering the flat slope in the area.

Therefore, the breach scenarios are considered to all occur on the Main Dam. However, location varies among flood scenarios depending on the number of open sluices to pass the flood.

6.3.2.1 Sunny Day Scenario

Figure 6.3 shows the dam breach location for the Sunny Day scenario.

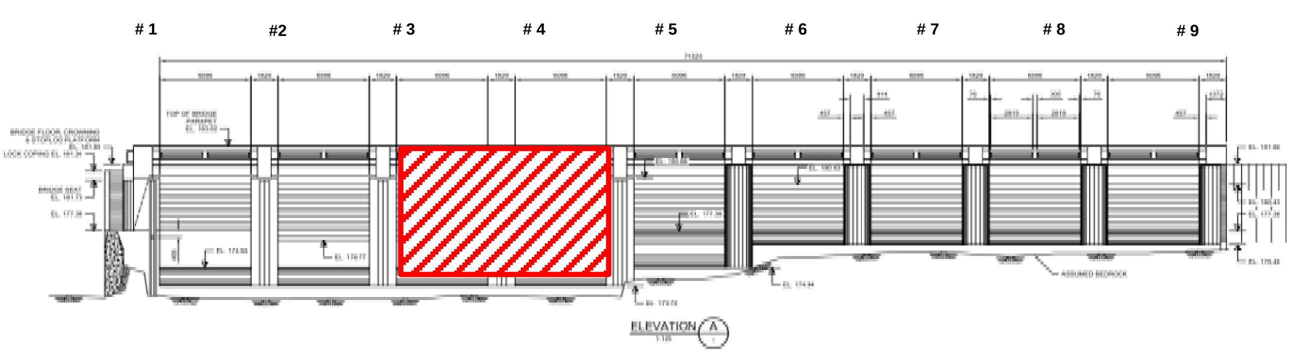

Figure 6.3 Dam Breach Location for the Main Dam, Sunny Day Scenario

The worst case scenario is the failure of sluices 3 and 4, including the centre pillar. The breach is initiated at a time where the upstream reservoir is at its maximum operating water level, creating a worst case flood wave for the hydrologic condition.

6.3.2.2 Flood Scenarios

Figure 6.4 shows the dam breach location for all flood scenarios. Note that the breach location differs from that of the Sunny Day scenario. This is because during floods, sluices 2 to 6 are open and therefore, this location results in a larger breach discharge than any other combination.

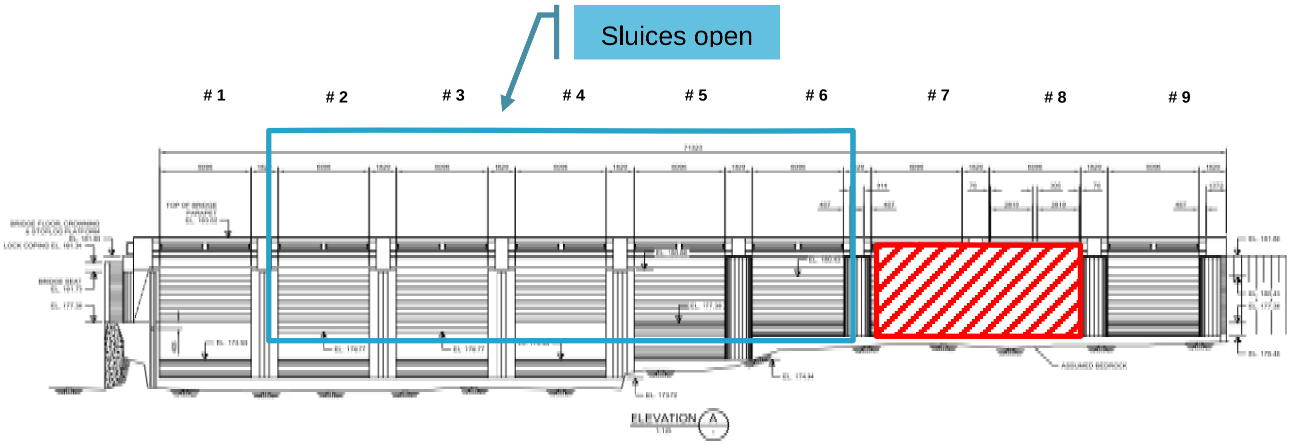

Figure 6.4 Dam Breach Location for the Main Dam, 100-year Flood Scenario

6.3.3 Stoplog Settings in Realistic Scenarios

The stoplog settings, corresponding to a realistic scenario for the Sunny Day and the flood scenarios, are presented in this section. Figure 6.5 shows the combined discharge capacity of the Port Severn Dams for all flood scenarios.

6.3.3.1 Sunny Day Scenario

- Main Dam: All stoplogs in place, except for sluices 2 and 3 which each have seven stoplogs in place. This setting allow to pass the Sunny Day flow of 41 m^3/s (mean discharge during the navigation season) while maintaining the water level between 180.4 m and 180.5 m, corresponding to the navigation season operating range (Maximum operating water level 180.5 m).
- Dam E and Dam G: maximum number of stoplogs in place.

6.3.3.2 Flood Scenarios

- Severe combination, to assess maximum water level.
- 1:100 return period:
  - Main Dam: sluices 2 to 6 open. 9 out of 12 stoplogs removed (from sluice 2 and 3) and sluice 4 to 6 fully open. Sluices 1, 7, 8 and 9 closed.
  - Dam E and Dam G: maximum number of stoplogs in place.
- 1:1000, 1:10000 and PMF:
  - Main Dam: sluices 2 to 6 open and sluices 1, 7, 8 and 9 closed. If any of the deepest stoplogs cannot be removed from sluices 2 to 6, twice the amount of stoplogs should be removed elsewhere (sluices 1, 7, 8 or 9 at the Main Dam). However, this scenario results in a smaller breach geometry.
  - Dam E and Dam G: maximum number of stoplogs in place.
- Overflow capacity is taken into account above all stoplogs and structures; when water level reaches the under deck elevation (soffit), the flow is pressurized in the sluices.

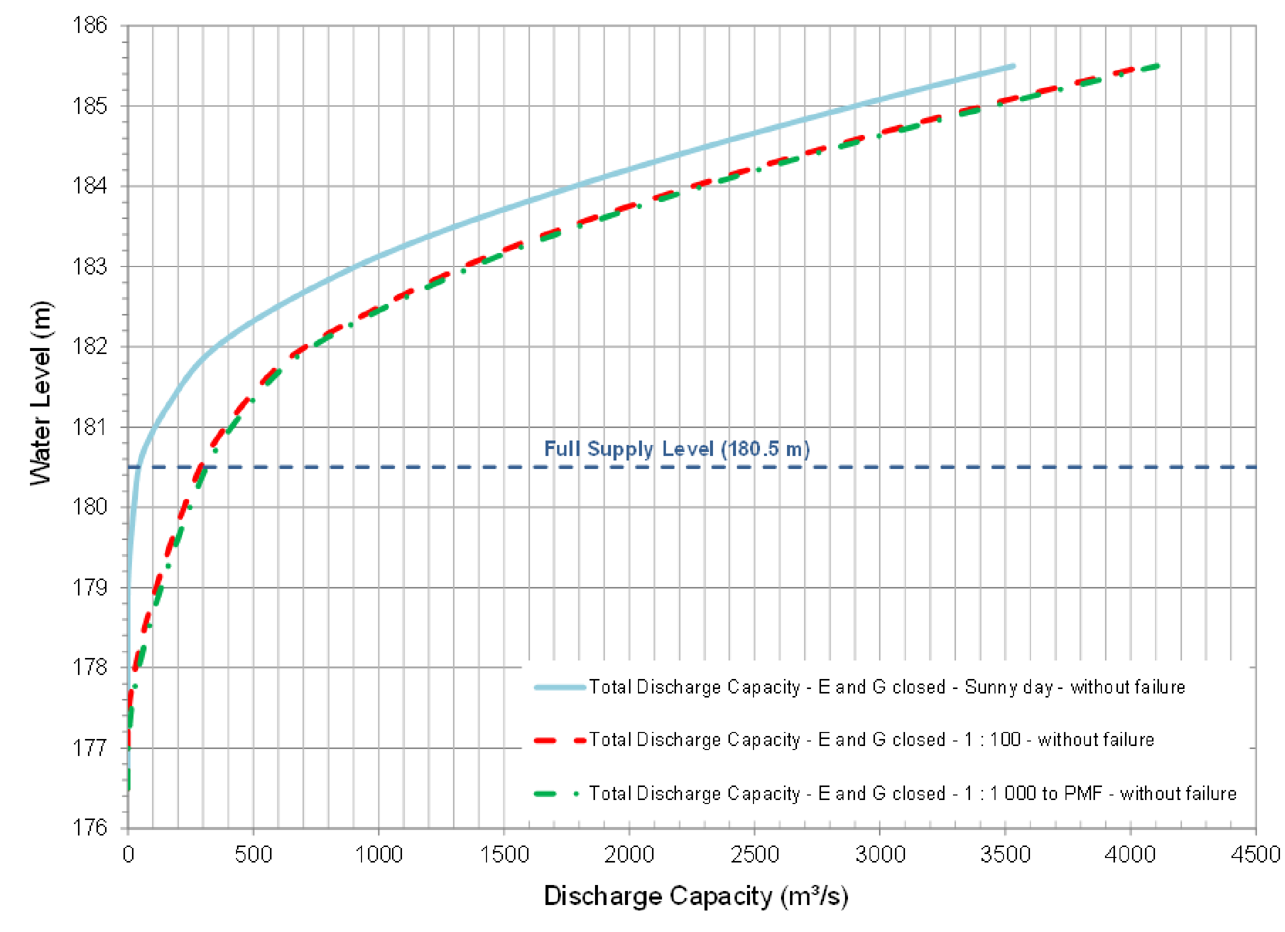

Figure 6.5 Discharge Capacity for Flood Scenarios

6.3.4 Flood Wave Routing

6.3.4.1 Dam Breach Hydrographs

The dam breach hydrographs represent the effect of dam break under different river flow scenarios. These hydrographs are estimated based on the river flows, the storage curve and the discharge capacity for the different structures. The dam break hydrograph set is presented for the Main Dam on Figure 6.6.

The discharge capacity has been calculated by considering all main flow conditions and hydraulic effects.

When the water level is under the dam crest soffit elevation, the flow conditions are open channel flow. When the water level reaches the dam crest soffit elevation, the flow conditions are considered pressurized under the dam bridge. When the water level overtops the dam, the flow is split in two parts. The flow conditions over the dam are considered as open channel and flow conditions located under the dam bridge are considered as pressurized.

Flow resistance around the piers and abutments as well as the submergence effect have been taken into account in the determination of the discharge capacity.

The discharge curve is smoothed in the transient area between open channel flow conditions and pressurized flow conditions.

Detailed graphs for each flood and dam are shown in Appendix D2.

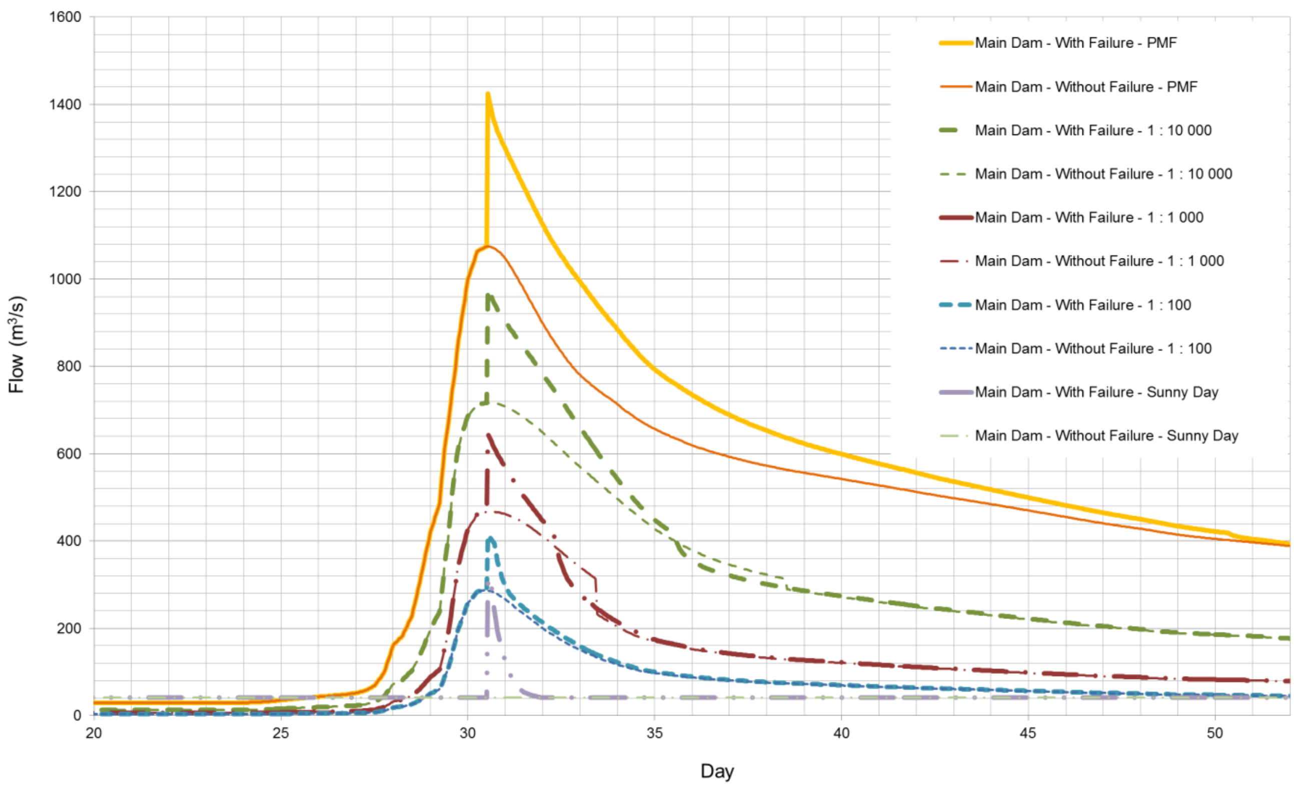

Figure 6.6 Main Dam Breach Hydrograph

From the foregoing hydrographs, Table 6.2 presents the maximum outflow at all dams.

Table 6.2 Summary of Maximum Outflow at All Dams during Flood Events

Structure	Flood Return Period			
	100-yr	1,000-yr	10,000-yr	PMF
Maximum natural outflow (m^3/s)	288	502	1,130	2,510
Upstream water level (m)	180.50	181.35	182.65	184.20
Main Dam maximum (m^3/s)	288	435	717	1,075
Dam G maximum (m^3/s)	0	4	179	767
Little Go Home Bay maximum (m^3/s)	0	23	106	248
Dam E maximum (m^3/s)	0.2	9	40	131
Dam A maximum (m^3/s)	0	0.2	90	294

6.3.4.2 Flood Wave Routing Analysis and Water Levels

Flood routing calculations were completed with HEC-RAS software. For each return period, the water level has been calculated with and without dam failure.

Table 6.3 presents the results of the flood routing calculations with HEC-RAS. The value *Dh* represents the increment that will be used to define the consequences.

Cross section position values represent the downstream location where the calculation was made. In this case, 0.00 km is located just downstream from the dam and -10.98 km is the furthest downstream point, 10.98 km downstream. These numbers are used in the HEC-RAS simulations for description of each cross section. Note that the calculation was performed to kilometric point -46.50 km, representing a point 46.5 km from the Main Dam in the Georgian Bay. The values are not presented as the effects of the flood and dam break are negligible after 10.98 km.

Figure 6.7 presents the location of the cross-sections or zones used in the model for the Main Dam.

Figure 6.7 Location Map for Cross-Sections at Main Dam

Table 6.3 Downstream Water Levels Computed – Main Dam

X-Sections (Figure 6.7)	Sunny Day			100-year Flood			1,000-year Flood			10,000-year Flood			PMF		
	Water Level (m)		Dh(m)	Water Level (m)		Dh(m)	Water Level (m)		Dh(m)	Water Level (m)		Dh(m)	Water Level (m)		Dh(m)
(km)	W/O F	W F		W/O F	W F		W/O F	W F		W/O F	W F		W/O F	W F	(m)
MAIN DAM															
0.00	176.49	177.00	0.51	177.34	177.44	0.10	177.66	177.79	0.13	178.04	178.21	0.17	178.44	178.65	0.21
-0.09	176.50	177.06	0.56	177.38	177.53	0.15	177.76	177.95	0.19	178.21	178.50	0.29	178.74	179.12	0.38
-0.20	176.50	177.05	0.55	177.38	177.52	0.14	177.74	177.94	0.20	178.18	178.47	0.29	178.67	179.04	0.37
-0.26	176.49	177.00	0.51	177.36	177.48	0.12	177.71	177.88	0.17	178.13	178.40	0.27	178.59	178.94	0.35
-0.39	176.48	176.57	0.09	177.24	177.24	0.00	177.51	177.51	0.00	177.81	177.81	0.00	177.85	178.14	0.29
HIGHWAY BRIDGE															
-0.46	176.48	176.56	0.08	177.24	177.25	0.01	177.51	177.51	0.00	177.81	177.81	0.00	177.81	177.81	0.00
-0.69	176.48	176.57	0.09	177.26	177.28	0.02	177.54	177.57	0.03	177.86	177.90	0.04	177.93	178.00	0.07
-1.20	176.48	176.53	0.05	177.25	177.27	0.02	177.53	177.55	0.02	177.84	177.87	0.03	177.89	177.94	0.05
-1.74	176.48	176.52	0.04	177.25	177.26	0.01	177.52	177.54	0.02	177.83	177.86	0.03	177.87	177.91	0.04
-3.64	176.48	176.51	0.03	177.24	177.25	0.01	177.52	177.53	0.01	177.82	177.84	0.02	177.85	177.87	0.02
-5.18	176.48	176.50	0.02	177.24	177.25	0.01	177.52	177.53	0.01	177.82	177.84	0.02	177.84	177.87	0.03
-6.98	176.48	176.50	0.02	177.24	177.25	0.01	177.51	177.52	0.01	177.82	177.83	0.01	177.83	177.85	0.02
-10.96	176.48	176.49	0.01	177.24	177.25	0.01	177.51	177.52	0.01	177.81	177.83	0.02	177.82	177.84	0.02
W/O F: without dam failure – W F : with dam failure															

6.4 Determine Time Categories

Time categories for the Loss of Life calculations assume the worst case scenario for people at risk downstream. This is a function of the time of day, because occupancy rates will change over the course of a day and season.

For the dam failure scenarios, there are two options to consider, a day scenario and night scenario. Both of these scenarios assume that it is summer, in the navigation season. This implies that there will be transient people making use of boats, the lock and fishing during the day time. The scenarios are summarized in Table 6.4.

Table 6.4 Time Category Scenarios

Scenario	% of Permanent People in Homes	Transient People Downstream	Comments
Night	100 %	0	A night scenario considers that 100 % of the people at risk are at home
Day	50 %	12	A day scenario considers 50 % of the people at risk are at home and there are 12 transient persons downstream. The transient people in this case are assumed to be fisherman and boaters.

6.5 Determine Dam Failure Warning Initiation

First, it is important to define what constitutes a warning in the procedure used here. According to Graham (Reference 33), the definition for "No Warning" is:

> *No Warning means that there is no warning issued by the media or official sources in the particular area prior to the flood water arrival; only the possible sight or sound of approaching flooding serves as a warning.*

According to the same text, the presence of rising waters downstream does not constitute a warning if it is not accompanied by an official warning.

Therefore, the warning time would be "No Warning".

6.6 Determine Area Flooded

The houses considered in the analysis have been identified from the orthophotos supplied by PCA. Buildings were classified as homes, commercial, mixed, vacant lots and boatsheds, with varying levels of day and night time population densities. These densities are shown in Table 6.5.

Table 6.5 Building Population Densities

ID	Permanent Night Density (ρ)	Permanent Day Density (ρ)
Boatshed	0	1
Commercial	0	2
Vacant*	2.63†	1.3
Mixed	2	2
Houses	2.63†	1.3

* Assumption that a home has been built on the vacant lot since DSR.

† The low density dwelling for the adjusted 20 year average is 2.63 people per house (Reference 63).

Population at risk (PAR) is therefore obtained by multiplying the number of buildings by the relative density. This is discussed in detail in Section 6.7- Estimate People at Risk.

6.7 Estimate People at Risk

The PAR is defined by USBR as the "*Number of people occupying the dam failure floodplain prior to the issuance of any warning.*" This does not take into account the depth of flooding they are subject to and the risk posed by the flooding, nor their ability to evacuate.

The Graham empirical-based equation makes use of the total population at risk (total PAR), meaning the amount of people at risk from both the naturally occurring flooding and the additional breach induced flooding (Reference 33). The Graham equation does not allow differentiating between total and incremental PAR. Incremental PAR is discussed in Section 6.10.1 - Incremental Loss of Life versus Loss of Life for a Reduced PAR. Another factor which affects the final PAR considered is the ability of the population to evacuate under certain conditions. This is further detailed in Section 6.10.2 – Evacuation.

Table 6.6 summarizes the number of buildings and population at risk in the flooded zones with dam failure for all scenarios (Sunny Day and flood events) at the Main Dam. Appendix D1 presents the flood depths for all buildings downstream of Main Dam and Appendix I presents the inundation mapping.

Table 6.6 Buildings and People Exposed to Flood for All Scenarios with Dam Failure at Main Dam – Based on Total PAR

ID	Perm. Pop. Densities†		Sunny Day			100-year Flood			1,000-year Flood			10,000-year Flood			PMF		
			Units	PAR		Units	PAR		Units	PAR		Units	PAR		Units	PAR	
	Night	Day		Night	Day		Night	Day		Night	Day		Night	Day		Night	Day
Perm. Pop.																	
Boatshed	0	1	0	0	0	0	0	0	12	0	12	12	0	12	12	0	12
Commercial	0	2	0	0	0	0	0	0	1	0	2	2	0	4	2	0	4
Vacant	2.63	1.3	0	0	0	0	0	0	0	0	0	1	3	1	1	3	1
Mixed	2	2	0	0	0	0	0	0	1	2	2	1	2	2	2	4	4
Houses	2.63	1.3	0	0	0	0	0	0	91	239	118	108	284	140	115	302	150
Subtotal				0	0		0	0		241	134		289	160		309	171
Transient People*				0	12		0	12		0	0		0	0		0	0
Total PAR				**0**	**12**		**0**	**12**		**241**	**134**		**289**	**160**		**309**	**171**
Total Buildings			**0**			**0**			**105**			**124**			**132**		

* Transient people are added to people at risk in buildings only for the day scenarios for Sunny Day and 100-year flood as no transient use is expected during large flood events. 0/12 corresponds to 0 transient people during the night and 6 during the day.

† Density is expressed in people per building.

6.8 Estimate Potential Loss of Life with Empirical Methods

In its sixth step, the procedure uses empirical based methods to estimate the fatality rate. Graham's Flood Severity Based Method (Reference 33) is currently preferred and the correct procedure when referring to the USBR procedure.

The procedure considers a series of factors to evaluate loss of life, with the following being the most important in estimating the number of fatalities:

- Number of people occupying the dam failure flood plain.
- Amount of warning provided by official sources.
- Severity of the flooding.
- The understanding of the flood severity.

6.8.1 Applying the Graham Method

It is important to note that the Graham method, and thus the entire procedure, is based on the use of a data set of historic dam failures in the US and abroad. It compiles and sorts the failures according to a series of factors. Dam classification is performed by associating the potential event (e.g. dam failure at Port Severn) to past historical events grouped according to studied factors, including total number of fatalities.

As such, the USBR procedure will estimate the total PAR and LOL associated with the failure event and does not provide any guidance to estimate the incremental LOL (LOL directly attributed to the failure, excluding LOL that would have occurred with the earthquake or flood without dam failure). Also, it does not allow for any reduction of the PAR to account for possible evacuation prior to the dam failure.

The Graham Flood Severity Based Method uses categories of flood severity and warning time to give an estimated failure rate, which represents the fraction of people at risk expected to die. Table 6.7 shows the recommended fatality rates using the Graham method, as presented in the USBR Procedure DSO-99-06.

Table 6.7 Recommended Fatality Rates for Estimating Loss of Life Resulting from Dam Failure (Reference 62)

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of PAR expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	*Use the value shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.*	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

6.8.1.1 Flood Severity

There are several methods to calculate the flood severity. For comparative purposes, two approaches are proposed in the USBR procedure and have been used in this analysis. These are the Graham Flood Based Severity approach and the DV (Depth Velocity) parameter approach. Flood severity should usually be evaluated based on expected impacts to structures. Should this prove difficult to do, particularly to differentiate between Medium and Low, water depth and the DV parameter can be used.

The calculation of the DV parameter is the same, whether the cause of the flooding is natural or following a dam break.

Table 6.8 Criteria for Evaluation of Flood Severity (Reference 62)

Flood Severity	Impacts on Structures	Water Depth	DV Parameter
HIGH	• Flooding from near instantaneous failure of concrete dam, or earthfill dam liquefied in minutes • Area swept clean by the flood wave such that little or no evidence of human habitation remains	Flood waves ultimate height reached in only minutes	None specified
MEDIUM	• Homes are destroyed • Trees or mangled homes remain for people to seek refuge in	> 10 ft (3 m)	> 50 ft^2/s (4.6 m^2/s)
LOW	• No buildings are washed off their foundation	< 10 ft (3 m)	< 50 ft^2/s (4.6 m^2/s)

For the specific case of Port Severn, incremental water depths do not exceed 2.41 m. This suggests that the buildings and structures flooded are not washed off their foundation.

To confirm this, the DV approach was used. The formula for calculating the parameter DV is as follows:

$$DV = \frac{Q_{df} - Q_{2.33}}{W_{df}}$$

Where,

- Q_{df} is the discharge at the site caused by dam failure,
- $Q_{2.33}$ is the mean annual discharge at the site
- W_{df} is the maximum width of flooding caused by a dam failure.

Table 6.9 shows the calculation and results of the analysis. According to Graham, low severity flooding should be assumed when DV is less than 50 ft^2/s (4.6 m^2/s).

Table 6.9 Calculation of DV – Depth – Velocity Parameter

	Scenarios	DV Calculation			
		DV	Q_{df}	$Q_{2.33}$	W_{df}
		m^2/s	m^3/s	m^3/s	m
Main Dam	Scenario 1 → Sunny Day	1.29	318	60	200
	Scenario 1 → 100-year	1.77	414	60	200
	Scenario 1 → 1,000-year	1.95	645	60	300
	Scenario 1 → 10,000-year	3.07	982	60	300
	Scenario 1 → PMF	3.90	1,425	60	350
Max		**3.9**			
Average		**2.4**			
Min		**1.3**			

The maximum value calculated for the DV parameter at Main Dam is 3.9 m^2/s (Table 6.9), which is considered to be in the mid to upper range for Low Severity. Note that for all other cases apart from the PMF the value is less, in the order of 1-3 m^2/s.

Therefore the Low Severity classification will be assumed.

6.8.1.2 Flood Severity Understanding

This does not apply when there is no warning. See DSO-99-06, page 27 (Reference 62).

6.8.2 Results

To differentiate incremental LOL from the total LOL, engineering judgment is used and further discussed in Section 6.10 – Apply Engineering Judgement. However, based on the USBR Procedure and from Table 6.7, the fatality rate is in the range of 0.0-0.02. For a dam failure under PMF conditions, there is a PAR which ranges from 171 to 309. This gives a total loss of life of 0 to 6.2 people.

6.9 Evaluate Uncertainty

The Procedure DSO-99-06 details how to estimate the loss of life based on a series of factor, yet it does insist that the engineer must rely on his or her judgment to evaluate uncertainties. Main uncertainties include:

- Cause of dam failure.
- Time of day at which failure would occur.
- Warning initiation.
- Inability to precisely determine fatality rate.

Ways to estimate the first three uncertainties are discussed in previous sections. However, the engineer should always use judgement rather than blindly follow the table.

The uncertainty on warning initiation is fairly significant. According to Procedure DSO-99-06, the presence of rising waters downstream does not constitute a warning if it is not accompanied by an official warning. However, since a flash flood is not likely to occur at Port Severn due to the large drainage area (more than 6,000 km^2) and the large routing capacity in upstream reservoirs, it should be considered that under large flood conditions, some parameters would change the flood severity understanding and/or authorities initiative to evacuate the population already flooded some time before the dam failure.

The most significant uncertainty and the one that requires the most discretion is the inability to determine fatality rate. Most notably, the use of the Graham method does not provide any assumption regarding evacuation.

Considering the reasons outlined above, steps 5 and 6 of the Graham method are revised using the engineering judgement in Section 6.10 – Apply Engineering Judgement to reduce the PAR.

6.10 Apply Engineering Judgement

The application of USBR's procedure DSO-99-06 results in a loss of life estimation of 0 to 6.2 for a failure under PMF conditions. While this has taken into account many factors, there remain site-specific issues that the procedure does not address. Moreover, since the procedure clearly states the importance of determining the incremental consequences of dam failure, judgement will also be applied in that objective. The following section addresses the evaluation of the incremental PAR most notably through the capacity to evacuate people at risk.

Figure 6.8 shows the engineering judgment iterative process, extracted from the Dam Classification Flow Chart shown on Figure 6.1.

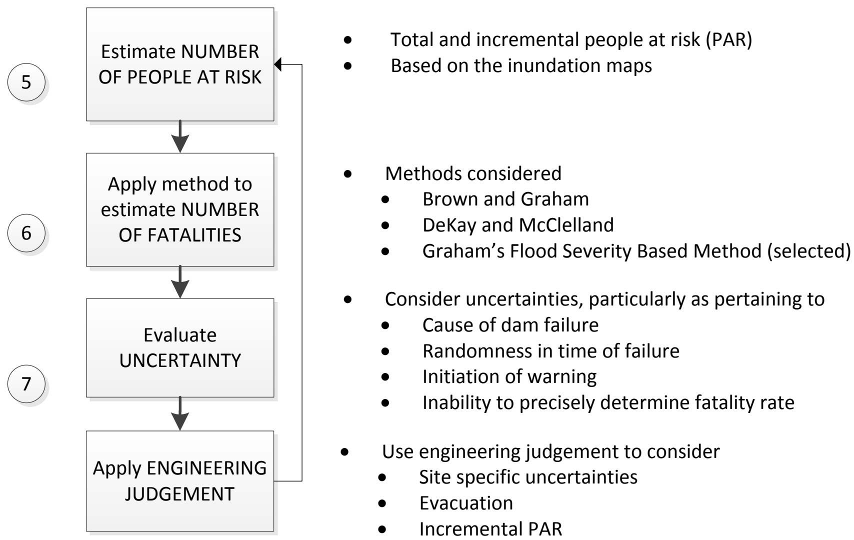

Figure 6.8 Dam Classification Flow Chart – Engineering Judgment

6.10.1 Incremental Loss of Life versus Loss of Life for a Reduced PAR

Little documentation exists on the evaluation of the incremental PAR. A study by Barton Maher (Reference 6) discusses the calculation of incremental people at risk using the USBR (Graham) method. The necessity of this conversion is the fact that the CDA uses the incremental effect of dam failure in the Dam Safety assessment; however, the Graham Method uses total PAR for calculation of loss of life.

The document states that the probability of loss of life for a flood without dam failure is much lower than a flood with dam failure (in the range of 0.001 – 0.0001 x PAR).

To put this in context, the background to this document must be explained. The Guidelines presented by Barton Maher regarding Acceptable Flood Capacity for Dams are based on a range of ANCOLD and other guidelines as listed below:

- Guidelines for Failure Impact Assessment of Water Dams, (ANCOLD, June 2010).
- Selection of Acceptable Flood Capacity for Dams (ANCOLD, 2000).
- Assessment of the Consequences of Dam Failure (ANCOLD, 2000).
- Risk Assessment (ANCOLD, 2003).
- Guide to Flood Estimation (AR&R 1999, Nathan, R. J. and Weinmann, P.E).

Following the process for loss of life from the most recent text, Guidelines for Failure Impact Assessment of Water Dams, it can be understood that the total PAR as estimated based on the USBR Procedure can be reduced, but should not be reduced to PAR in the incremental flooded area due to the dam failure, as it is the case for the estimation of economic losses due to the failure, considering some people flooded by a small amount may remain in the flooded area. Therefore, PAR should include more people than those in the incremental flooded area to remain conservative.

6.10.2 Evacuation

Table 6.6 shows the number of properties affected by the flood and how the consequences increase rapidly with increasing flood return period. This is intuitive, but requires justification regarding the people at risk. During the large floods, the time taken for the water level to rise is likely observable or audible to the public.

A large storm, required to generate flooding of this magnitude, will clearly be visible outside the occupant's homes. Therefore, during a large flood, for example the 10,000-year flood, some people in the obvious flood zone are assumed to be making plans to move to higher ground.

For the large floods, people at risk from the dam break are likely to be overestimated by formula presented in DSO-99-06, which does not take into account the fact that people will move if they can observe the onset of a large flood. Also, it does not take into account evacuation from authorities.

In summary, at the time of the dam break, less people are expected to be present than if there was no observable flooding danger prior to the break. Three main arguments support the idea that evacuation will occur under unusual or extreme events:

- Reasonable time to consider evacuation following rising water depth.
- Anticipated evacuation from authorities (following the activation of the CA or MNR ERP).
- Overtopping of the dams and embankments.

The Conservation Authority (CA) and Ministry of Natural Resources (MNR) Districts (where no CA exists) maintain a local flood notification/warning system, monitor local conditions, such as ice jam potential, local heavy rainfall/snow melt events and issue local flood messages.

Some jurisdictions such as CAs will also issue a bulletin regarding water or flood safety. These messages are intended for the users of river/lake systems and flood plain occupants.

It is important to note that MNR has no power to forcibly evacuate persons should they not wish to leave. The Emergency Response Plan of the MNR (Reference 48) advises staff as follows:

> *MNR staff should use the following strategy with individuals refusing to evacuate:*
> - *Strongly encourage residents to evacuate.*
> - *Make sure they understand the risk associated with not evacuating.*
> - *Advise them that they are responsible for the consequences associated with staying.*
> - *Document your advisory, e.g. names, date and time.*
> - *Maintain a registry of those people staying and their locations as well as those evacuating.*
> - *Solicit assistance from other agencies which have more influence and authority, e.g. OPP, local police.*

The above responses attempt to encourage evacuation and those that refuse are made to claim responsibility. Therefore, it is reasonable to assume that the evacuation can play a real role in reducing the dam break PAR.

6.10.2.1 Evacuation Indicators: Depth

For all scenarios except the Sunny Day scenario, natural flooding will be felt by the downstream population first. Dwellings flooded are an important cause for alarm. This should prompt the authorities, as well as the residents, to begin evacuation. Furthermore, dwellings flooded by reasonable depths can still be easily evacuated. This should significantly lower the PAR due to a dam failure.

Nonetheless, it is assumed that warning for evacuation will be given to all dwellings flooded by more than a certain depth of water during a significant flood event. For the purpose of the loss of life evaluation, this depth is assumed to be 50 cm of water. This depth is a buffer to compensate for:

- The fact that the average house elevation differs from the actual. For example, a flooded home on a sloping section will observe different water levels around the property. The average ground floor elevation is assumed to be at 50 cm above surrounding areas.
- Limitations in the incremental accuracy of the HEC-RAS water levels.
- A level of 50 cm is the level where it is assumed that the authorities will force evacuation from homes.

The PAR to be considered is the people at risk as a consequence of the dam break. Figure 6.9 illustrates how flooding depth relates to the definition of PAR due to a dam failure only when compared to PAR as estimated based on the USBR Procedure (total PAR).

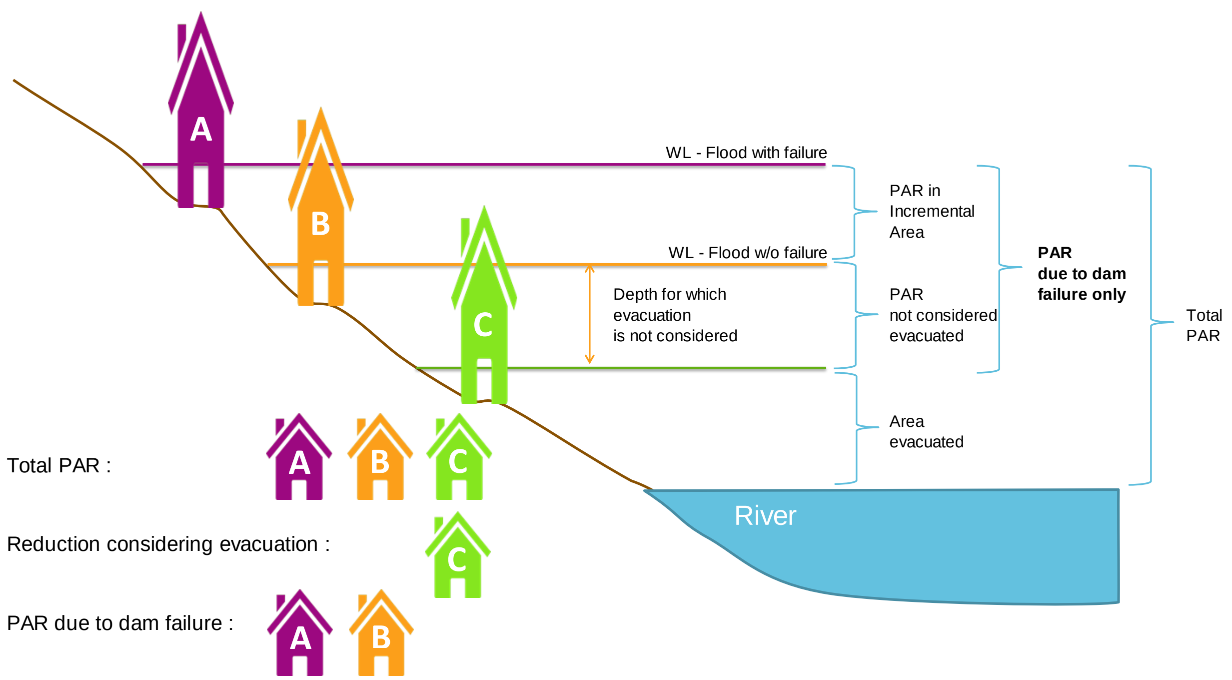

Figure 6.9 Evacuation Depth, as it Relates to Incremental PAR

6.10.2.2 Evacuation Indicators: Overtopping

All dams are overtopped during the PMF and 10,000-year flood. This also includes Highway 400 in the area downstream of Dam G during the PMF.

In order to estimate a conservative number of people at risk for the purpose of dam classification, overtopping of dams is considered as a clear sign that there is a risk. This is relevant for flood severity understanding prior to a dam failure under large flood conditions.

Dam overtopping is a direct indication of risk. It is recommended that, when water levels near overtopping, PCA should activate the EPP. Following the directives of the EPP, evacuation protocols should be initiated through the MMR or CA.

Crest elevation of these dams are presented in Table 1.2.

6.10.3 PAR Due to Dam Failure Only

The Graham empirical-based equation makes use of the total PAR, meaning the amount of people at risk from both the naturally occurring flooding and the additional breach induced flooding. To evaluate the potential Loss of Life associated with dam failure, it is necessary to differentiate between total PAR and PAR as a consequence of a dam failure. The PAR due to a dam failure corresponds to the people in the incremental flooded area plus people flooded by 50 cm or less by the natural flood just before the dam failure.

Note that a difference between the dam break water level and the natural flood level of less than 10 cm would be considered to have no significant effect and is omitted from the PAR due to a dam failure set.

For example, Figure 6.10 shows the inundation map, with and without failure, for the PMF, while Figure 6.11 shows the number of buildings affected by a dam failure at Main Dam during the PMF flood. Table 6.10 summarises the number of buildings and people exposed to flood for all scenarios with dam failure, considering the PAR as a consequence of the dam failure only.

Appendix I presents all inundation mapping, with and without failure, for all dam break scenarios.

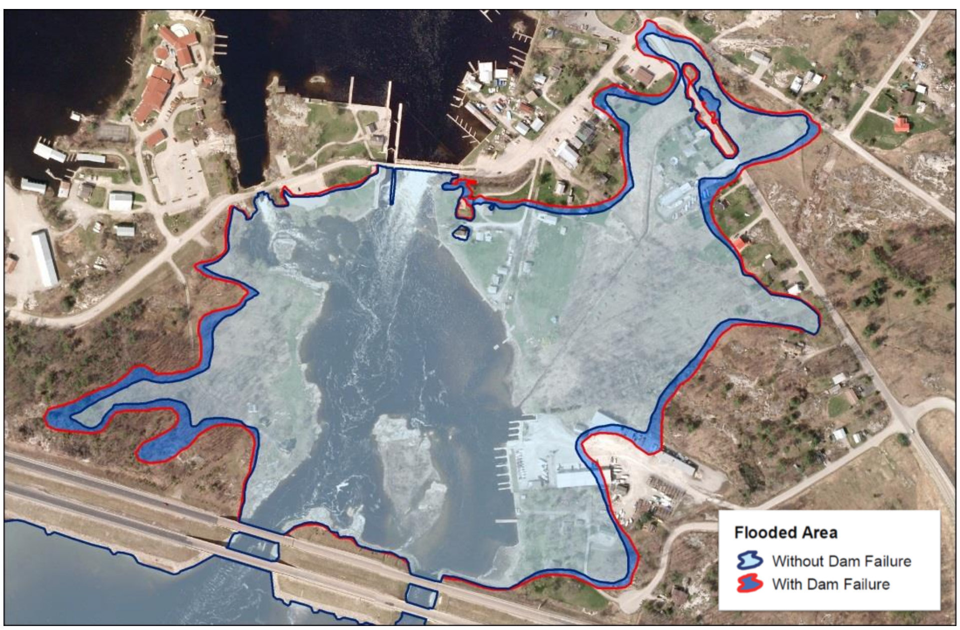

Figure 6.10 Main Dam – Inundation mapping – PMF Scenario (With and Without Dam Failure)

Figure 6.11 Main Dam – Location of the Buildings Impacted by the PMF

It can be seen from Table 6.10 that the night scenario PAR is the most severe for the Main Dam, with the PMF case being critical. For the PMF night-time failure of Main Dam, 20 people are affected.

Table 6.10 Main Dam – Summary of Buildings and People Exposed to Flood for All Scenarios with Dam Failure, Based on a Reduced PAR

ID	Permanent Population		Main Dam				
			Flood				
	Night ρ	Night ρ	Sunny Day	100	1,000	10,000	PMF
Boatshed	0	1	0	0	0	0	0
Commercial	0	2	0	0	0	1	0
Vacant	2.63	1.3	0	0	0	1	0
Mixed	2	2	0	0	0	0	1
Houses	2.63	1.3	0	0	4	6	7
Transient people*	0	12	0 or 12	0 or 12	0	0	0
TOTAL (buildings affected)			0	0	4	8	8
Reduced PAR - Night			0	0	11	18	20
Reduced PAR - Day			12	12	5	11	11

* Transient people are added to people at risk in buildings only for the day scenarios for Sunny Day and 100-year flood as no transient use is expected during large flood events. 0/12 corresponds to 0 transient people during the night and 12 during the day.

6.10.4 Fatality Rate

Applying the selected fatality rate of 0 – 0.02 (see Table 6.7) to the PAR due to dam failure under IDF conditions of 20 people and PAR due to dam failure under Sunny Day conditions of 12, the Loss of Life estimates reach 0.0 – 0.4 people and 0.0 – 0.24 people respectively.

6.11 Life Safety Assessment Results

Following the use of engineer's judgement to shed light on the project specific uncertainties, the loss of life has been revised. The following factors have been taken into account:

- Evacuation depth.
- Rate of water level rise, considered instantaneous.
- Separation of the incremental PAR from the total PAR.

Based on the USBR procedure, the Graham method and the judgement of its engineers, AECOM estimates for a failure at the Main Dam:

- Loss of Life of 0.
- Transient PAR of 12.

The Loss of Life and transient PAR are used as inputs in the Hazard Classification (Section 6.13 - Dam Classification).

6.12 Social and Economic Losses

The PCA hazard potential classification system for dam safety management purposes (Reference 49) lists *Property Losses* and *Environmental, Cultural and Heritage Losses.*

Property losses, as defined by the CDA Dam Safety Guidelines (Reference 8), include all losses relative to economic activities. In the current analysis, the physical value of assets is estimated under Property Losses while losses relative to economic activities are found under Economic Losses.

6.12.1 Environmental, Cultural and Heritage Losses

No significant Heritage or Culture site or property has been identified downstream of the dam. Thus there is no likelihood of any significant Heritage or Culture loss should a dam breach occur.

Environmental losses are generally summed up to wildlife habitat. The main threats to habitat under dam failure conditions are increased velocity, high sediment concentration (aquatic wildlife) and rising waters (surface wildlife). It is assumed that there is no sediment buildup in the reservoir. Sedimentation is not likely to occur upstream of the dam due to the time taken to travel through the reservoir. In addition, during the site inspection, the underwater camera did not reveal sedimentation at the dam. Therefore, a potential dam break would not lead to more than minimal habitat loss with high capability of natural restoration. Furthermore, flood wave routing has shown that increased velocities and water levels do not threaten significant habitat loss.

6.12.2 Property Losses

According to the method exposed in the technical bulletin of CDA (Reference 8), Table 6.11 shows the value of losses per property type in the flooded area for the Main Dam. The average MLS® price considered for the evaluation of the monetary value of damage is $395,800 per house (Reference 21). To simplify calculations, $400,000 is used in this study. Monetary value of other building types is based on extrapolation of this value.

Table 6.11 Hazards per Flooded House Number of Houses and Damage Costs – Main Dam

Type	Value	Main Dam - Damages				
		Sunny Day	100	1,000	10,000	PMF
Boatshed	$100,000	-	-	-	-	-
Commercial	$500,000	-	-	-	$500,000	-
Vacant	$450,000	-	-	-	$450,000	-
Mixed	$450,000	-	-	-	-	$450,000
Houses	$400,000	-	-	$1,600,000	$2,400,000	$2,800,000
TOTAL Damage		$-	$-	$1,600,000	$3,350,000	$3,250,000

6.12.3 Economic Losses

Significant flooding would cause economic losses to the businesses in the Port Severn area. Properties such as wharves or quays would suffer significant damages. Furthermore, tourism, which is the main economic activity of the region, would suffer additional losses. Quantifying the economic impacts would require a complete socio-economic analysis.

6.13 Dam Classification

The PCA Directive states that “The hazard potential classification assigned to a dam for the purpose of dam safety management, shall be based on the higher classification as determined from the incremental effects of a potential dam failure […]”. Table 6.12 presents a summary of the PCA Directive’s Dam Hazard Classification Table (Figure 2 of Reference 49). The equivalent CDA classification is also indicated.

Table 6.12 Dam Hazard Classification Table (Reference 49)

PCA Hazard Classification	CDA Hazard Classification	Life Safety Hazard		Third-Party Losses [1]	
		Loss of Life	Transient PAR	Property	Environmental / Cultural
Very Low	**Low**	0	0	≤ $122,000 (2009$)	Very low likelihood of negatively affecting status of wildlife population. Little or no damage to heritage sites.
Low		0	≤ 10	≤ $1.2 million (2009$)	Low likelihood of negatively affecting status of wildlife population. Minor reversible damage to heritage sites.
Significant	**Significant**	0	11 to 100 Temporary, seasonal use	≤ $12.2 million (2009$)	Appreciable loss of habitat with reasonable likelihood of applying recovery activities. Reversible damage to heritage sites.
High A	**High**	≤ 10	Permanent population at risk	≥ $12.2 million (2009$)	Extensive loss of habitat with little or no feasibility of applying recovery activities. Irreversible damage to heritage sites.
High B	**Very High**	11 to 100			
High C	**Extreme**	≥ 100			

[1] Complete definitions of the third-party losses criteria can be found in Figure 2 of the PCA Directive (Reference 48)

The Hazard Classification for Main Dam is selected by looking up each life safety and third-party loss determined from the dam breach analysis for Main Dam in Table 6.12. Table 6.13 presents the resulting Hazard Classification for each dam breach scenario. The highest classification is retained.

As a result, the Hazard Classification for Main Dam is Significant, as per PCA Directive (Reference 49). Under CDA Guidelines (Reference 8) the Hazard Classification for Main Dam is also Significant.

Table 6.13 Main Dam Classification Results

	Summary Results for the Incremental Zone				
	Sunny Day	1:100-yr	1:1,000-yr	1:10,000-yr	PMF
Life Safety					
Loss of Life	0	0	0	0	0
Transient PAR	12	12	0	0	0
Third-party losses					
Property losses	$0	$0	$1,600,000	$3,350,000	$3,250,000
Environmental / cultural losses	0	0	0	0	0
Hazard Classification	Very Low	Very Low	**Significant**	**Significant**	**Significant**

6.14 Selection of Design Criteria

6.14.1 Selection of the Inflow Design Flood (IDF)

The IDF is selected based on the Hazard Classification determined based on the consequences of a potential dam breach. The IDF Selection Table, shown in Table 6.14 is excerpted from the PCA Directive (Reference 49) and in accordance with the CDA Dam Safety Guidelines (Reference 8).

Since Main Dam is classified as a Significant Hazard dam, its IDF has a recurrence interval between 100-year and 1,000-year. Considering the unquantified economic impacts (losses from tourism and other businesses), the 1,000-year flood is selected. This amounts to an IDF equal to 502 m³/s.

Table 6.14 IDF Selection Table (Reference 49)

<table>
<tr><th rowspan="2">Hazard Potential Classification</th><th colspan="3">Range of Inflow Design Floods for Life Safety Hazard</th><th rowspan="2">Range of Inflow Design Floods for All Other Hazards</th></tr>
<tr><th>Expected Loss of life</th><th>Transient Population at Risk</th><th>Range of Inflow Design Floods For Life Safety Hazards</th></tr>
<tr><td>Very Low</td><td rowspan="3">0</td><td rowspan="3">For major flood events, transient use would not be expected</td><td rowspan="3"></td><td>25-yr flood to 100-yr flood</td></tr>
<tr><td>Low</td><td>100-yr flood</td></tr>
<tr><td>Significant</td><td>100-yr flood to 1,000-yr flood</td></tr>
<tr><td>High A</td><td>10 or less</td><td></td><td>1/3 between 1:1,000-yr flood and PMF</td><td>1,000-yr flood</td></tr>
<tr><td>High B</td><td>11 to 100</td><td></td><td>2/3 between 1:1,000-yr and PMF or the 10,000-yr flood whichever is greater</td><td></td></tr>
<tr><td>High C</td><td>More than 100</td><td></td><td>Incremental analysis or PMF</td><td></td></tr>
</table>

6.14.2 Selection of the Design Earthquake

The DBE is selected based on the Hazard Classification determined based on the consequences of a potential dam breach. The Earthquake Design Selection Table, shown in Table 6.15 is excerpted from the PCA Directive (Reference 49) and in accordance with the CDA Dam Safety Guidelines (Reference 8).

Since Main Dam is classified as a Significant Hazard dam, its DBE has a range of 500-year to 1,000-year. The 500-year earthquake is chosen as the classification is at the low end of the range (transient use of 12 people).

Table 6.15 Earthquake Design Selection Table (Reference 49)

Hazard Potential Classification	Range of DBE (Note 2) for Life Safety Hazards			Range of DBE (Note 6) for all other Hazards
	Expected LOL (Note 3)	Transient PAR	Range of DBE For Life Safety Hazards	
Very Low	0	0		No specific requirements
Low	0	10 or less		100-yr to 500-yr earthquake
Significant	0	11 - 100		500-yr to 1,000-yr earthquake
High A	10 or less	For situations in which over 100 transient persons may be at risk, loss of life would be expected	2,500-yr earthquake (Note 4)	1,000-yr earthquake (Note 5)
High B	11 to 100		5,000-yr earthquake	
High C	More than 100		10,000-yr earthquake or maximum credible earthquake	

Notes:

1. *Permanent population at risk shall be assumed to be equivalent to Life Safety Hazard, unless acceptable risk-based assessments are performed under an approved dam safety management plan to define the expected loss of life.*
2. *In general, transient persons reasonably expected to be subjected to incremental Life Safety Hazards as a result of a dam breach from a random event such as an earthquake are treated as permanent population at risk.*
3. *For the selection of design parameters where a Life Safety Hazard exists, the estimation of expected Loss of Life shall be defined by US Bureau of Reclamation's "A Procedure for Estimating Loss of Life Caused by Dam Failure" (DSO-99-06), or an acceptable equivalent. The selection of the design basis earthquake (DBE) within the range provided must be the magnitude that most closely relates to the range of Hazard Potential within the Hazard Potential Classification (HPC) as shown in the sliding scale method for earthquake at Figure 8.*
4. *The 2,500-yr earthquake is the building code standard and, as such, should be the minimum standard for HIGH hazard dams for life safety.*
5. *For existing dams having a SIGNIFICANT or higher HPC, a dam safety management plan may be used to reduce the risks posed by the dam to the tolerable range if the dam does not meet the DBE standards.*
6. *Where no threat to public safety exists, a maximum value of the 1,000-yr earthquake is used.*

6.15 Inundation Mapping

The results of the downstream routing for the IDF and Sunny Day dam failure scenarios are illustrated on inundation maps (Appendix J). The inundation maps show the following information at critical downstream locations:

- Distance from the dam.
- Time of flood arrival.
- Time of flood peak (from the initiation of the dam failure).
- Maximum flood levels.
- Maximum velocities.

6.16 Future Dam Safety Review Program

According to the PCA Directive, DSR studies must be periodically performed. Each review must be performed up to the standards in force at the time of review. Frequency of review is a function of hazard classification. All dams shall be inspected according to the minimum frequencies shown in Table 6.16.

Since the Port Severn Main Dam has a Significant classification, the PCA Directive suggests that it be the object of Dam Safety Reviews every 10 years. Also, engineering inspections should be carried out every 3 years with routine inspections every 4 months.

Engineering inspections can only be performed by qualified engineers experienced in dam and dam safety. Certified engineering technologists (C.E.T.), or equivalent, with sufficient experience in dams and dam safety, can also carry out the inspections, granted that they be supervised by a qualified engineer. Engineering inspections are defined in the PCA Directive (Reference 49) as:

> *[…] thorough visual inspections of dams and associated instruments, with detailed documentation of observations and assessment of conditions, features or deficiencies observed, highlighting changes from previous inspections, and providing recommendations for maintenance, repairs or other follow-up.*

Routine inspections must be performed by trained maintenance or operation staff, familiar with the site and facilities. They consist of the inspection of the structures and grounds with the objective of identifying and signaling changes in appearance or performance and signs of problems such as sinkholes, seepages, boils, clogged drains, slope slippage, and more (Reference 49).

Table 6.16 Dam Safety Review Required Frequency (Reference 49)

Hazard Classification (2)	First Dam Safety Review (1) (year)	Next Dam Safety Review (years)	Engineering Inspection (years)	Routine Inspection (months)
Very low	Not required	Not required	5	12
Low	2015	15	3	6
Significant	2013	10	3	4
High (no loss of life)	2012	7	1	2 (3)
High (loss of life)	2011	5	1	1 (3)

(1) First dam safety review is mandatory in years shown for dams deemed vulnerable either because of known poor condition in accordance with PCA rating system (Risk to Asset and/or Level of Service ratings) or known design deficiency affecting dam safety.

(2) The dam safety classification is to be validated during each engineering inspection.

(3) Measures such as remote monitoring, webcams and other techniques can be used to reduce the requirement for routine site inspection frequency for remote dams.

7. Dam Safety Analysis

7.1 Dam Hydraulic Capacity

Hydro-technical studies of the watershed and flood routing simulations are used to calculate the flows that can be discharged at the dam. Studying a dam's hydraulic capacity means:

- Evaluating its designed discharge capacity, under regular and extreme conditions.
- Evaluating its actual discharge capacity, under regular and extreme conditions.

This is necessary to assess:

- If and how the dam can safely discharge the expected floods.
- If and how the dam can be operated to fulfill its purpose.
- Which issues/deficiencies cause the most important problems to the safe discharge at the dam.

7.1.1 Operating Procedures

The Port Severn Dams are operated to maintain the water level within the minimum and maximum normal operating levels (180.42 to 180.50 m during summer and 180.20 to 180.50 m during winter). This provides the best conditions for use of the waterway for navigation, recreation and other purposes. The operators add or remove logs within the sluices to increase or decrease discharge so as to maintain water levels.

In the occurrence of a major flood, emergency operation calls for the urgent removal of logs. Water level rise could endanger the safety of the dam and result in undesirable impacts. Staff must be able to operate the dams to discharge the IDF, which as explained in Section 6.11 – Life Safety Assessment Results, is the 1,000-year flood equal to 502 m^3/s.

7.1.2 Discharge Analysis

Charts indicating the flow passing through the dam for a given water level and dam operation status (number of logs in place) or the number of logs to remove at a given water level are often found at the facilities to help the operators manage the facilities. None were found at Port Severn. Table 7.1 and Table 7.2 present the discharge which can be expected to flow through the Port Severn Dams based on the number of logs removed. These flow simulations are performed for water levels equal to 181.35 m and 180.50 m, respectively corresponding to the maximum water level under IDF conditions and the maximum normal operating level (MNOL). The following hypotheses and comments are related to the results:

- Little Go Home Bay Dam constantly discharges instream flow requirements through the logs. This discharge is minimal and has little effect on the overall capacity. It was evaluated based on engineering judgement during the site visit.
- When water levels rise above 180.62 m, the Little Go Home Bay Dam is overtopped and these overflows are added to the total discharge.
- The downstream water level can affect the discharge. Flow from the bottom logs can be impeded by the submergence phenomenon when downstream water level rises (values in orange). Maximum potential impact from submergence represents the possible flow decrease in percentage.
- The rock outcrop directly downstream of sluices 7, 8 and 9 restrict the flow out of these sluices (values in grey).
- Bottom logs under the dashed line are difficult to remove. They cannot be relied upon during flood events.

A detailed evaluation of the discharge capacity was conducted.

Free overflow is considered until the water level reaches the bottom of the deck. Free overflow is considered for the overtopping of the structures when the upstream water level reaches the deck or the top of the embankments or walls.

The potential for downstream submergence of hydraulic passages was also considered, especially for flood scenarios.

The weir equation which considers the effect of submergence is presented below.

$$Q = C_d S B_e \sqrt{2g} H^{3/2}$$

Where:

- *Q* = Discharge over Spillway
- C_d = Theoretical discharge Coefficient (range from 0.36 to 0.46)
- *S* = Submergence Coefficient (range from 0 to 1, where S=1 represents a free outflow)

$$S = \left[1 - \left(\frac{H_u}{H}\right)^2\right]^{0.5}$$

- B_e = Effective spillway width which considered the effect of the piers and their shape:

$$B_e = B - 2K_p H$$

- K_p = Theoretical pier coefficient (range from 0.0 to 0.1)
- *H* = *Upstream Head*
- H_u = Downstream Head

Pressurized flow is considered for all hydraulic passages when the upstream water level reaches the bottom of the deck. As for free overflow, the potential for downstream submergence was verified. The following equations were used.

For pressurized flow without submergence effect:

$$Q = C_d ab\sqrt{2gh_1}$$

Where:

- *Q* = Discharge under Gate
- *a* = Gate or Spillway Opening
- *b* = Spillway Width
- h_1 = Upstream Head
- C_d = Discharge Coefficient

$$C_d = C_{d0}\exp\left[\frac{-\bar{A}}{2}\left(1 - \frac{\delta^2}{6}\right)\right]$$

- C_{d0} = Base Discharge Coefficient

$$C_{d0} = \mathrm{z}\left[\frac{4 + 5e^{-0.76\delta}}{9}\right]$$

- ζ = 0.98 for under deck
- δ = Angle between the face of the dam deck and the horizontal
- $\bar{A}$ = Relative Opening

$$\bar{A} = \frac{a}{h_1}$$

For pressurized flow with submergence effect:

$$Q = C_c ab\sqrt{2g(h_1 - h_v)}$$

Where:

- Q = Discharge under Gate
- a = Gate or Spillway Opening
- b = Spillway Width
- h_1 = Upstream Head
- C_c = Contraction Coefficient
- F_2 = Downstream Froude Number
- h_s = Head for the downstream boundary submersion

$$h_s = \frac{1}{2}h^*[(1 + 8F_2^2)^{0.5} - 1]$$

$$h^* = C_c a$$

- h_v = Head of the stagnant water behind the gate or spillway and stoplogs

$$h_v = 2h^*\left(1 - \frac{h^*}{h_u}\right) + h^*\left[4\left(1 - \frac{h^*}{h_u}\right)^2 - 4\frac{h_1}{h^*}\left(1 - \frac{h^*}{h_u}\right) + \left(\frac{h_u}{h^*}\right)^2\right]^{0.5}$$

- h_u = Downstream Head

Table 7.1 Discharge at Main Dam at Maximum Water Level under IDF Conditions (181.35 m) as a Function of Stoplog Removal

Stoplogs Removed	Discharge (m³/s) Main Dam's Sluices 1	2	3	4	5	6	7	8	9	Dam E	Dam G	Little Go Home Bay Dam	Maximum Potential Impact from Submergence
0	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.19	9.08	3.53	22.70	-
1	14.08	14.08	14.08	14.08	14.08	14.08	14.08	14.08	14.08	13.79	5.42	*	-
2	19.58	19.58	19.58	19.58	199.58	19.58	19.58	19.58	16.26	19.02	7.52	*	-
3	25.63	25.63	25.63	25.63	25.63	25.63	25.63	25.63	16.26	24.70	9.81		-
4	32.16	32.16	32.16	32.16	32.16	32.16	32.16	32.16	16.26	30.75	12.26		-
5	39.13	39.13	39.13	39.13	39.13	39.13	39.13	39.13	16.26	37.14	14.85		-
6	46.51	46.51	46.51	46.51	46.51	46.51	46.51	46.51	16.26	43.81	17.57		-
7	54.26	54.26	54.26	54.26	54.26	54.26	51.55	46.76	16.26	50.71	20.39		-
8	62.36	62.36	62.36	62.36	62.36	62.36	51.55	46.76	16.26	57.83			0.3 %
9	70.78	70.78	70.78	70.78	70.78	70.78	51.55	46.76	16.26	64.69			1.2 %
10	79.50	79.50	79.50	79.50	79.50	79.50	51.55	46.76	16.26				2.6 %
11		88.51	88.51										4.0 %
12		97.75	97.75										5.7 %

Green: not affected by submergence - Orange: affected by submergence - Grey: log removal has no effect due to downstream rock outcrop

— - — bottom logs below dashed line cannot be relied upon for sluice operation

* Little Go Home Bay Dam is not operated for flood discharge.

Table 7.2 Discharge at Main Dam when Water Level is at MNOL (180.50 m) as a Function of Stoplog Removal

Stoplogs Removed	Discharge (m³/s) Main Dam's Sluices 1	2	3	4	5	6	7	8	9	Dam E	Dam G	Little Go Home Bay Dam	Maximum Potential Impact from Submergence
0	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.22	0.02	0.10	-
1	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.40	2.44	0.90	*	-
2	5.90	5.90	5.90	5.90	5.90	5.90	5.90	5.90	3.70	5.84	2.24	*	-
3	10.20	10.20	10.20	10.20	10.20	10.20	10.20	10.20	3.70	10.04	3.91		-
4	15.20	15.20	15.20	15.20	15.20	15.20	15.20	15.20	3.70	14.86	5.85		-
5	20.80	20.80	20.80	20.80	20.80	20.80	20.80	20.80	3.70	20.19	7.99		-
6	27.00	27.00	27.00	27.00	27.00	27.00	27.00	27.00	3.70	25.96	10.32		-
7	33.60	33.60	33.60	33.60	33.60	33.60	31.30	27.20	3.70	32.09	12.80		-
8	40.70	40.70	40.70	40.70	40.70	40.70	31.30	27.20	3.70	38.54			-
9	48.10	48.10	48.10	48.10	48.10	48.10	31.30	27.20	3.70	45.25			-
10	56.00	56.00	56.00	56.00	56.00	56.00	31.30	27.20	3.70				0.9 %
11		64.10	64.10										2.4 %
12		72.60	72.60										4.2 %

Green: not affected by submergence - Orange: affected by submergence - Grey: log removal has no effect due to downstream rock outcrop

— - — bottom logs below dashed line cannot be relied upon for sluice operation

* Little Go Home Bay Dam is not operated for flood discharge.

7.1.3 Adequacy of Discharge Capacity

A dam must be able to pass safely the IDF. The current section evaluates solely the adequacy of the design discharge capacity, or in other words: the flow the dam was designed to discharge. However, while a dam may have been designed to discharge a certain flow, the actual adequacy of discharge capacity must also take into account existing operation limitations. These are assessed in Section 9.3.2 – Operation Review for Dam Safety.

From the discharge analysis, it is determined that the maximum discharge the Port Severn Dams are designed to pass, with all sluices open is:

- 489 m^3/s at MNOL of 180.50 m.
- 502 m^3/s under IDF conditions, with water level reaching 180.57 m.

This shows that the dam was originally designed with sufficient capacity to pass the IDF flood without raising the water level to unacceptable levels (the maximum level acceptable to maintain dam safety under IDF conditions is 181.35 m).

Figure 7.1 shows the cumulative theoretical discharge at Port Severn for the maximum level under IDF conditions and at MNOL, assuming each sluice can be fully opened. The order in which sluices are opened is based on the capacity of the sluices and ease of operation. In Figure 7.1, sluices that can be opened using the log lifter are relied upon first, followed by those requiring the use of the manual winches. However, as recommended in Section 11.3 - Operational Procedures, it may be preferable to open the most difficult sluices first, when a flood is forecasted, as a safety measure. Note that at 181.35 m, water flows over the fully closed sluices.

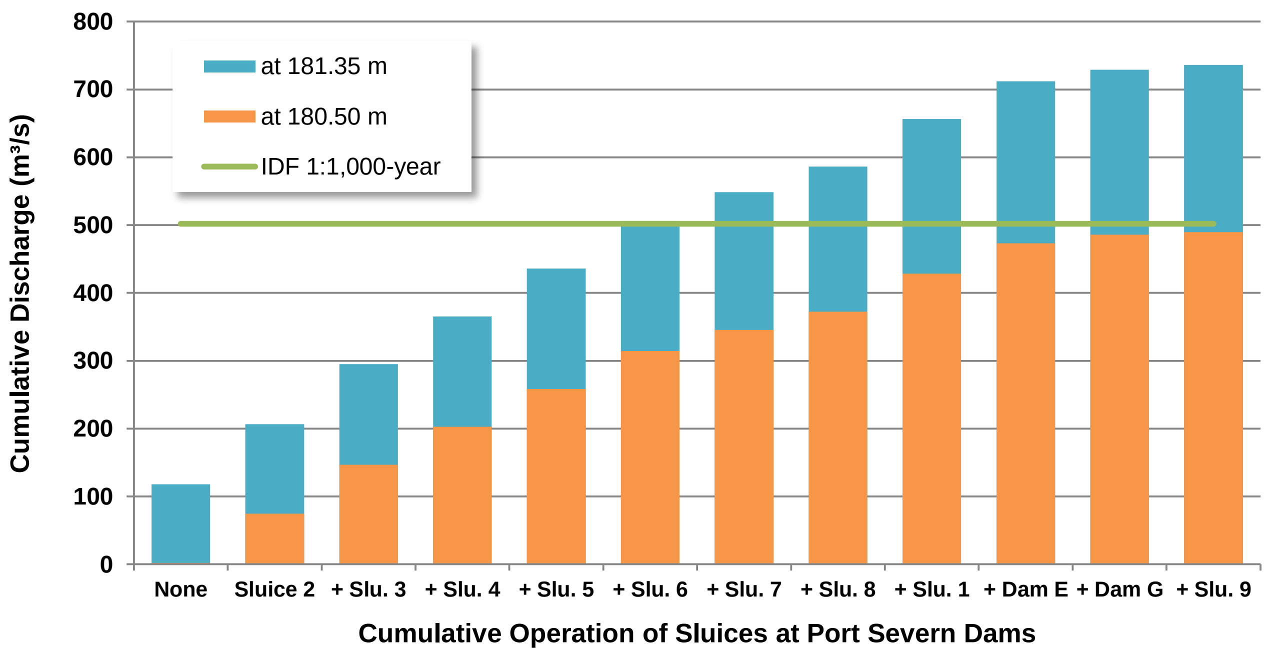

Figure 7.1 Cumulative Theoretical Discharge at the Port Severn Dams Following Recommended Operation Sequence

Considering that bottom stoplogs that can't be relied upon are still in place, Figure 7.2 shows the cumulative realistic discharge at Port Severn for the maximum level under IDF conditions and at MNOL, assuming a maximum of 8 stoplogs are removed from each sluice.

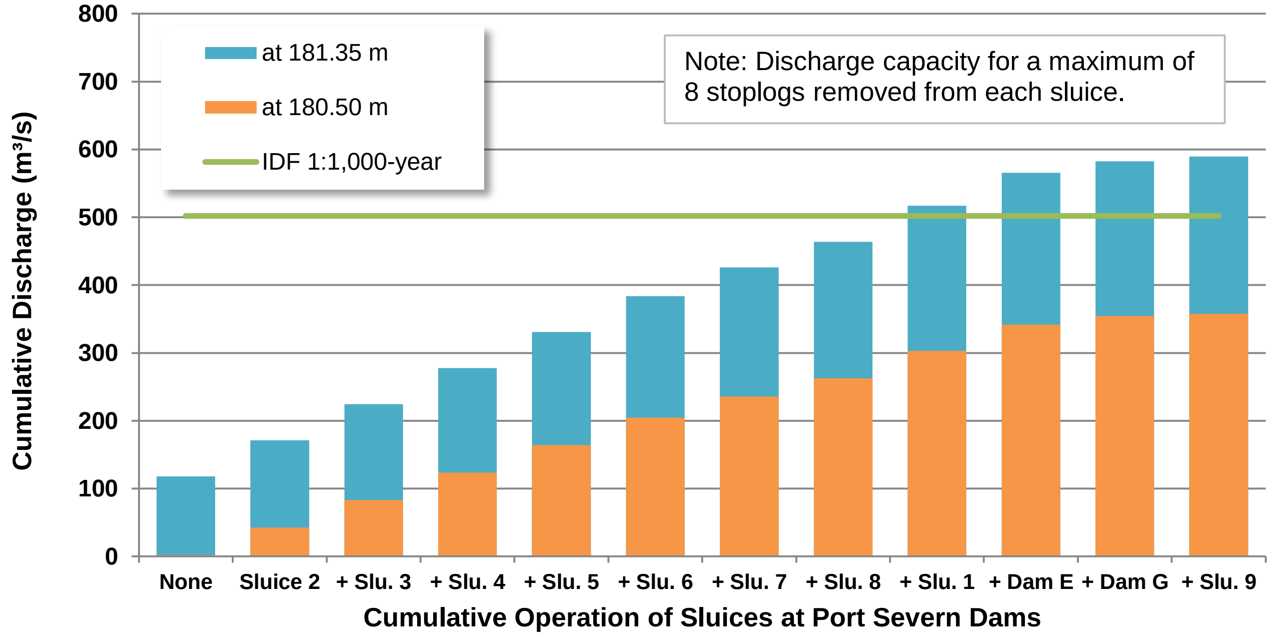

Figure 7.2 Cumulative Realistic Discharge at the Port Severn Dams Following Recommended Operation Sequence

The current discharge analysis shows that the Port Severn Dams are designed to discharge the IDF and can theoretically do so. However, it does not take into account operability issues that stem from staff capabilities, state and maintenance of the facilities and equipment, work under adverse conditions or others. This is demonstrated in Section 9.3.2 - Operation Review for Dam Safety.

7.2 Freeboard and Wave Action

This section discusses the freeboard available at Port Severn Dams and then determines the freeboard requirements, as outlined in the CDA.

Freeboard is the minimum vertical distance between the still pool elevation and the crest of the water impounding structure. This freeboard distance, or greater, should be maintained at all times, to ensure that there is always a margin of safety against overtopping of the structures. The freeboard allowance provides protection against waves, the setup (or tide) created by the wind and the effect of waves breaking on an inclined surface, called wave run-up. The CDA Technical Bulletin - Hydrotechnical Considerations for Dam Safety (Reference 16) defines two types of freeboard:

- Normal freeboard: difference in elevation between the lowest elevation of the top of the dam and maximum reservoir operating water level.
- Minimum freeboard: difference in elevation between the lowest elevation of the top of the dam and maximum still pool reservoir level that would result should the IDF occur.

7.2.1 Freeboard Requirements

7.2.1.1 Normal Freeboard

The normal freeboard is calculated such that there is no overtopping by 95 % of the waves caused by the most critical wind with a frequency of 1:1,000 when reservoir level is at MNOL (Reference 16). Normal freeboard calculation is applied to all structures, whether concrete or embankment.

7.2.1.2 Minimum Freeboard

For the calculation of the minimum freeboard, wave non-exceedance probability, wind annual exceedance probability (AEP) and reservoir level considered depend on other factors.

For embankment dams, the 95 % non-exceedance remains. For concrete structures, which can resist overtopping without serious damage, the non-exceedance probability can be reduced or overtopping may be allowed provided that the integrity of the dam, its abutments and ancillary structures is not compromised (Reference 16). In either case, if the structure’s crest is used as an access to outflow control structures, the non-exceedance should be increased to 99 %, such that only 1 % of the waves would overtop. This more stringent criterion is necessary for the safety of the operators.

Wind AEP for minimum freeboard is selected based on level of consequence. Table 7.3 presents the AEP associated with consequence levels.

Table 7.3 Annual Exceedance Probability for Minimum Freeboard (Reference 16)

PCA Consequence Level	CDA Consequence Level	AEP
Very Low and Low	Low	1:100
Significant	Significant	1:10
High A, High B, High C	High, Very High and Extreme	1:2

In all cases, the reservoir level considered for evaluation of the minimum freeboard is the extreme level during the passage of the IDF.

7.2.2 Determination of Wind Setup, Waves and Wave Run-up

This section describes the steps and outlines the assumptions used in the calculation of required freeboard at Port Severn. The procedure used is described in the Technical Bulletin - Hydrotechnical Considerations for Dam Safety (Reference 16).

The steps taken to establish the elevation reached by wave run-up against a structure are the following:

- Select the appropriate wind frequency for the given wind direction against the dam.
- Compute the wind set up, which is the maximum reservoir surface tilting due to the effect of a sustained wind in a specific direction.
- Compute the effective fetch for each structure.
- Compute the resulting waves.
- Compute the run-up of the breaking waves based on the slope and material of the structure.

Normal Freeboard

Normal freeboard is calculated using the same criteria for all structures. This includes no overtopping by 95 % of the waves and a wind AEP of 1:1,000.

Minimum Freeboard

For Lock 45, Dam D and the Upstream Shoreline Wall, the largest 5 % (95 % non-exceedance) of waves will be used in the calculation of minimum freeboard requirement. For Main Dam, some overtopping can be tolerated as there are no earth dams. When the crest of a concrete structure is used as an access to control structures, wave splashing during the IDF should be limited to 1 % (99 % non-exceedance) of the design waves for Main Dam, as this is used to access the stoplog sluices.

For the Port Severn Main Dam, which has a Significant classification (as per both PCA and CDA), a return period of 10 years will be used in the calculation of minimum freeboard for all structures.

Both types of freeboard are considered. The criteria are summarized in Table 7.4.

Table 7.4 Freeboard Allowance (Reference 16)

Freeboard	Wind Frequency	Wave Non-Exceedance	Structure
Normal Freeboard	1:1,000	95 %	All structures
Minimum Freeboard	1:10	95 %	Lock 45, Dam D and Upstream Shoreline Wall
	1:10	99 %	Main Dam

7.2.2.1 Wind Frequencies and Wind Direction.

In order to estimate the wave height and freeboard allowance, wind direction and speed at the site must be estimated. In the event that there is not a climate station at the site location, a representative site should be used.

Several climate stations were reviewed to obtain wind data for the Port Severn Site. Three Muskoka sites were chosen as representative due to the proximity to site (37.7 km) and the number of years available. These datasets were consolidated to obtain a unique dataset of 30 years. The source of the wind data is the National Climate Data and Information Archive (Reference 40).

Details regarding the three stations are shown in Table 7.5.

Table 7.5 Muskoka Climate Stations

Climate ID	Climate Station Name	Location						Distance from Site (km)	Period of Record	# Years
		Latitude			Longitude					
		°	'	"	°	'	"			
6115525	MUSKOKA A	44	58	0	79	18	0	37.5	1983 - 2005	23
6115524	MUSKOKA AWOS	44	58	29	79	18	12	37.7	2005 - 2009	5
6115529	MUSKOKA	44	58	29	79	18	12	37.7	2009 - 2013	5

The wind frequencies chosen for the wave setup and run-up calculation are based upon suggested return periods in the CDA (Reference 16). The range of wind speeds for different return periods and the associated directions are shown in the wind rose in Figure 7.3.

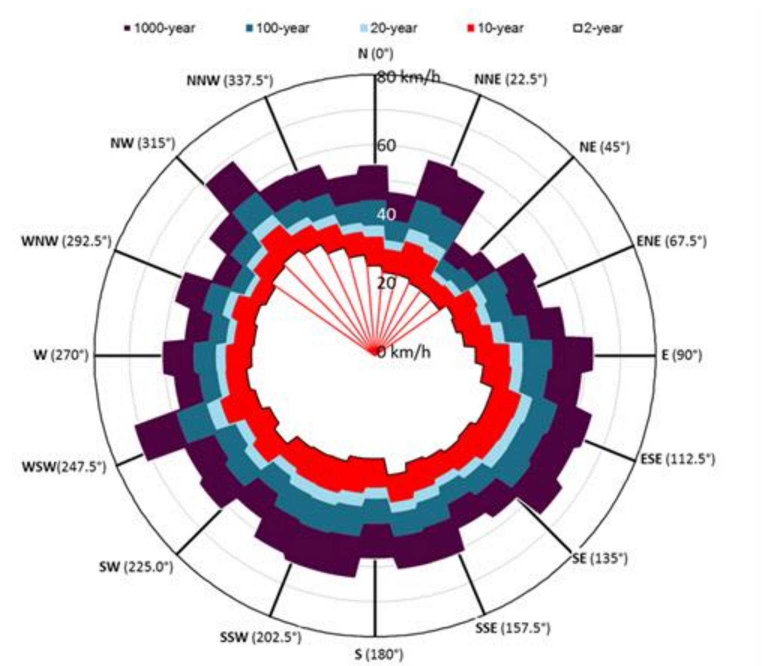

Figure 7.3 Muskoka Wind Rose

7.2.2.2 Wind Speed over Water

Wind stations located on land record wind speeds modified by a larger surface roughness than over water. Therefore, an escalation factor must be used to transpose land recorded wind speeds over a body of water.

The Shore Protection Manual (Reference 60) recommends that if the fetch is less than 16 km then the ratio of wind speed over water to wind speed over land (RL) can be taken to be 1.2, with the assumption that the boundary layer is not in full adjustment to the water surface. Therefore, in the case of Port Severn, the adjustment factor used is 1.2.

7.2.2.3 Wind Setup

This section calculates the wind setup for the dams at Port Severn. The wind setup will be added to the wave height to determine the freeboard allowance.

When the wind blows over a water surface, it exerts a horizontal stress on the water, driving it in the direction of the wind. This wind effect results in water accumulating at the downwind end of an enclosed body of water and a lowering of the water level at the upwind end. This total effect is called wind tide, and the “piling up” at the downwind end is referred to as wind setup.

Wind setup is a function of the wind speed, the water depth and the fetch length. The fetch length is the distance across open water where the wind can generate waves.

Wind setup effects may be transferred around substantial changes in direction in the reservoir and thus a longer fetch distance is warranted in this expression than the fetch length required for determining the wave height. The fetch length used for the Port Severn freeboard allowance calculation includes Little Lake and the entire Gloucester Pool. The measured fetch is 8,544 m long, as shown in Figure 7.4.

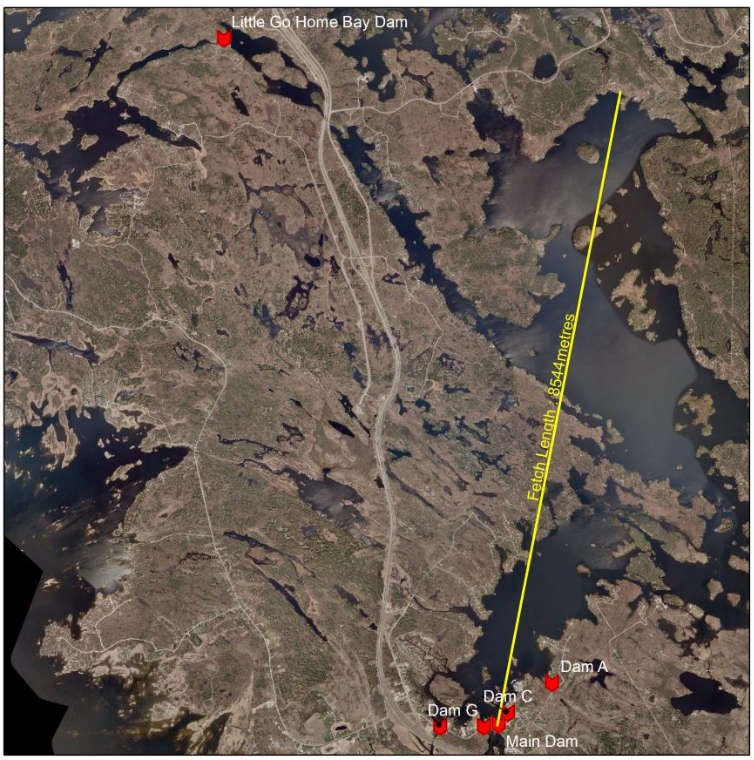

Figure 7.4 Fetch Length for Wind Setup

The wind setup results are shown in Table 7.6 and Table 7.7. The maximum wind setup for the 10-year case is 0.07 m, for a wind originating in the 330° (NNW) direction. The maximum wind setup for the 1,000-year case is 0.15 m, also for a wind originating in the 330° (NNW) direction.

7.2.2.4 Effective Fetch Length

This section discusses effective fetch and the method used to calculate the effective fetch length for a lake. In this case, as opposed to the wind setup case, islands and shorelines break the fetch length and therefore the wave generation.

In narrow reservoirs where shorelines are irregular, such as that of Little Lake and Gloucester Pool, width restrictions limit the growth of waves. A commonly accepted procedure for determining the effective fetch length consists of constructing nine radials from the point of interest at 3-degree intervals and extending these radials until they first intersect the shoreline. The length of each radial is measured and arithmetically averaged.

Figure 7.5 shows the fetch lengths under normal water levels and during the passage of the IDF. During elevated water levels, some islands are assumed to be submerged and therefore, the fetch is increased. This is also demonstrated in Table 7.6, with a maximum fetch of 1,094 m (MNOL) and Table 7.7 with maximum fetch of 2,216 m (IDF).

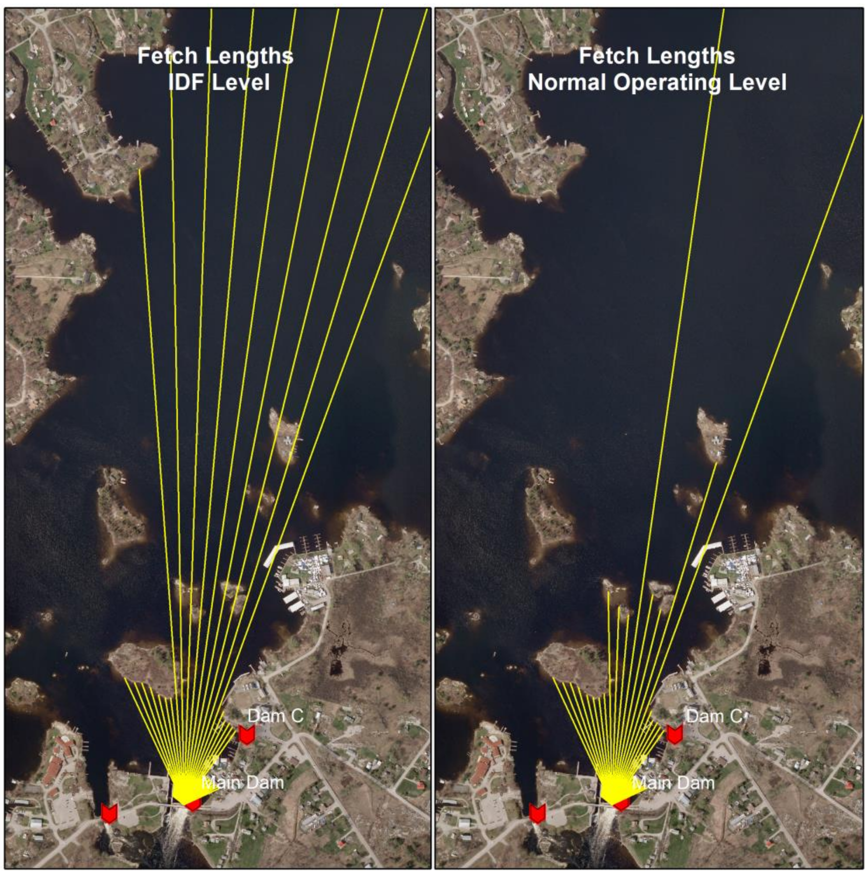

Figure 7.5 Fetch Length for Wave Calculations

7.2.2.5 Resultant Waves Height

This section discusses the results of the wave height calculations. The following sections will discuss implications for the freeboard allowance at the Port Severn structures.

The results for the wave calculation are shown in Table 7.6 and Table 7.7. The tables show several resultant wave heights. The significant wave height is the average of the highest one-third of all waves. Other calculated wave heights based upon the significant wave height are also shown below.

- Average of the highest 10 % of all waves → H_{10} = 1.27 H_s.
- Average of the highest 5 % of all waves → H_5 = 1.37 H_s.
- Average of the highest 1 % of all waves → H_1 = 1.67 H_s.

From Table 7.6 and Table 7.7, it can be seen that for the normal freeboard calculation (with 1,000-year wave), there is a significant wave height of 40 cm. For the minimum freeboard case (and the 10-year wave), there is a significant wave height of 34 cm, from a wind in the 10° direction.

7.2.2.6 Wave Run-up

When a wave reaches the toe of a sloping embankment, the wave will break on the embankment and run-up the slope to an elevation governed by the slope of the structure, the water depth at the structure toe, the roughness and permeability of the embankment and the incident wave characteristics. Wave run-up is the vertical difference between the maximum elevation attained by wave run-up on a slope and the water elevation at the toe of the slope, excluding wave action.

In the case of Port Severn, where the structures feature vertical upstream walls, the run-up is negligible, in the order of 2 cm.

7.2.3 Freeboard Results

Two types of freeboard are evaluated: normal and minimum. The normal freeboard case is assessed for no overtopping by 95% of waves at all structures. The water level considered is the MNOL, equal to 180.50 m. The minimum freeboard is assessed for two cases: for no overtopping by 95% of waves for Lock 45, Dam D and the Upstream Shoreline Wall and with overtopping by 99% of waves for the Main Dam. For both minimum freeboard cases studied, the maximum water level under IDF conditions, equal to 181.35 m, is considered. Associated wind frequencies are 1,000-year for the normal freeboard case and 1:10 for the others.

Table 7.6 shows calculated values of wave height, run-up and setup at different wind directions, calculated for the 1,000-year wind and a normal freeboard with overtopping non-exceedance of 95%. The normal freeboard wave reaches 0.66 m. This wave height consists of a wave of 0.55 m, a wave run-up of 0.02 m and a wind setup of 0.09 m. This corresponds to the 1,000-year wind speed of 52.1 km/h, from a direction of true north (0°). Figure 7.6 shows the normal freeboard wave and effective fetch length from different wind directions.

Table 7.7 shows calculated values of the wave height, run-up and setup at different wind directions, for the 95 % and 99 % waves, calculated for the 10-year wind and a minimum freeboard.

Considering the first minimum freeboard case, a height reaching 0.52 m would not be exceeded by 95 % of waves. This wave height consists of a wave of 0.46 m, a wave run-up of 0.02 m and a wind setup of 0.04 m. This corresponds to the 10-year wind of 33.9 km/h, from a direction of 10°. For the second minimum freeboard case, a height of 0.62 m would not be overtopped by 99 % of the waves. This wave height consists of a wave of 0.56 m, a wave run-up of 0.02 m and a wind setup of 0.04 m. Figure 7.7 and Figure 7.8 show the minimum freeboard. Also shown is the effective fetch length from different wind directions.

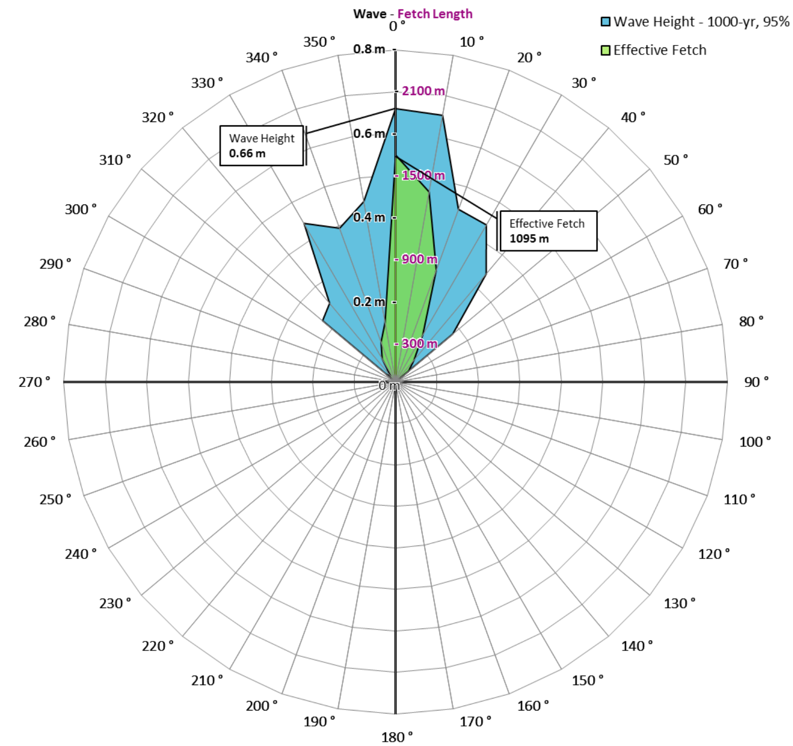

Figure 7.6 95 % Non-Exceedance Wave Height (with Run-Up and Setup) and Fetch Length – 1,000-Year Wind

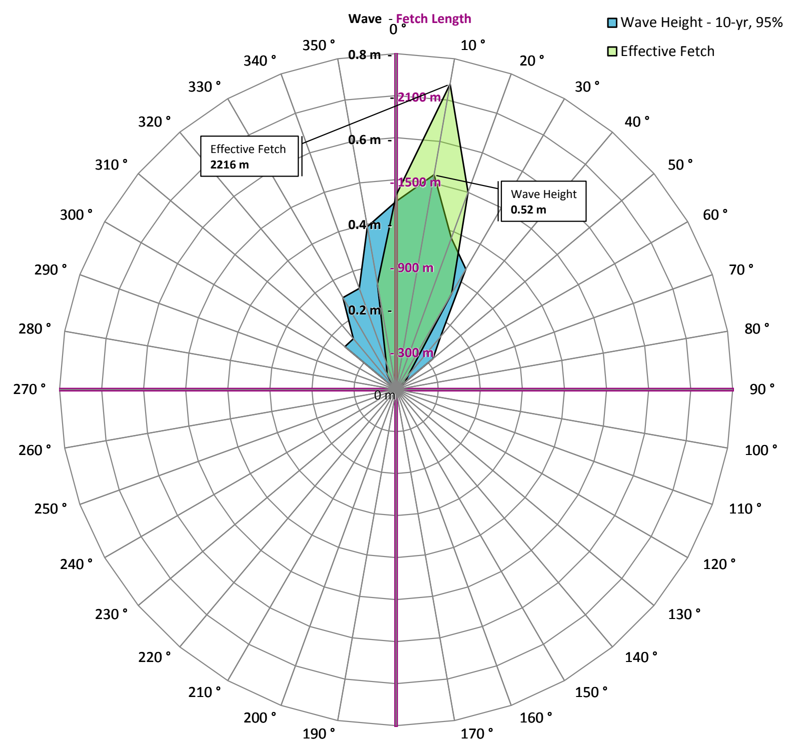

Figure 7.7 95 % Non-Exceedance Wave Height (with Run-Up and Setup) and Fetch Length – 10-Year Wind

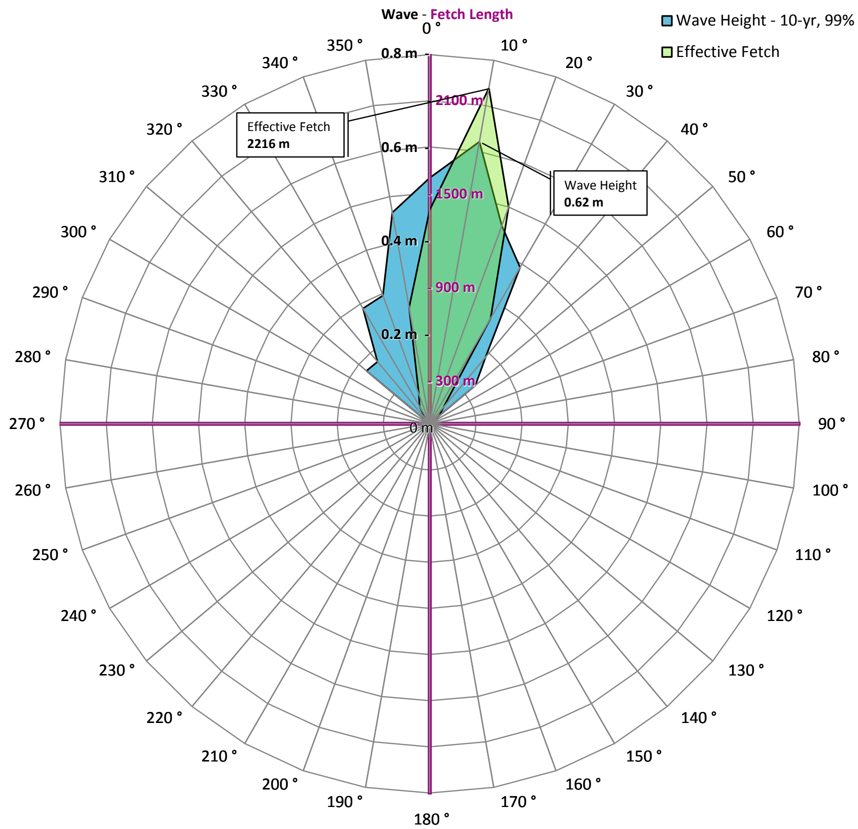

Figure 7.8 99 % Non-Exceedance Wave Height (with Run-Up and Setup) and Fetch Length – 10-Year Wind

Table 7.6 Port Severn - Normal Freeboard Condition (1,000-Year Wind) Wave Calculation for 95 % Non-Exceedance

Wind Direction (Degrees)	310	320	330	340	350	0	10	20	30	40	50
Effective fetch length (m)	**57**	**38**	**129**	**204**	**291**	**1,094**	**931**	**574**	**263**	**139**	**83**
1:1,000 wind speed (km/h)	51.4	58.1	68.9	57.3	56.8	52.1	54.4	47.0	58.0	56.4	39.8
Significant wave height (m)	0.09	0.09	0.19	0.20	0.23	0.40	0.39	0.26	0.22	0.16	0.08
95 % wave height (m)	0.12	0.12	0.27	0.27	0.32	0.55	0.54	0.35	0.31	0.22	0.11
Run-up (m)	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Wave and run-up (m) – 95 %	0.14	0.14	0.29	0.29	0.34	0.57	0.56	0.37	0.33	0.24	0.13
Wind setup (m)	0.09	0.11	0.15	0.11	0.11	0.09	0.10	0.07	0.11	0.10	0.05
Normal freeboard (m) – 95 %	**0.23**	**0.25**	**0.44**	**0.39**	**0.44**	**0.66**	**0.65**	**0.44**	**0.44**	**0.34**	**0.18**

Table 7.7 Port Severn - Minimum Freeboard Condition (10-Year Wind) Wave Calculation for 95 % and 99 % Non-Exceedance

Wind Direction (Degrees)	310	320	330	340	350	0	10	20	30	40	50
Effective fetch length	**57**	**38**	**129**	**204**	**768**	**1,390**	**2,216**	**1,500**	**792**	**139**	**83**
1:10-year wind speed km/h	39.3	42.3	46.2	41.5	39.1	35.5	33.9	30.5	33.7	32.5	27.6
Significant wave height (m)	0.06	0.06	0.12	0.13	0.24	0.28	0.34	0.24	0.20	0.08	0.05
95 % wave height (m)	0.09	0.08	0.16	0.18	0.32	0.39	0.46	0.33	0.27	0.11	0.07
99 % wave height (m)	0.11	0.10	0.20	0.22	0.39	0.47	0.56	0.41	0.33	0.13	0.08
Run-up (m)	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02	0.02
Wave and run-up (m) – 95 %	0.11	0.10	0.18	0.20	0.34	0.41	0.48	0.35	0.29	0.13	0.09
Wave and run-up (m) – 99 %	0.13	0.12	0.22	0.24	0.41	0.49	0.58	0.43	0.35	0.15	0.10
Wind setup (m)	0.05	0.06	0.07	0.06	0.05	0.04	0.04	0.03	0.04	0.03	0.02
Minimum freeboard (m) – 95 %	**0.16**	**0.16**	**0.25**	**0.26**	**0.39**	**0.45**	**0.52**	**0.38**	**0.33**	**0.16**	**0.11**
Minimum freeboard (m) – 99 %	**0.18**	**0.18**	**0.29**	**0.30**	**0.46**	**0.53**	**0.62**	**0.46**	**0.39**	**0.19**	**0.13**

7.2.4 Existing Freeboard at Port Severn

The following section reviews the available normal and minimum freeboard at Port Severn. The available normal and minimum freeboards for all structures are shown in Table 7.8.

In all cases, the Main Dam is not overtopped by wave, run-up and setup. It is important to note that the 99 % wave does not negate the ability to operate the flow control equipment on the Main Dam. Evaluated for normal freeboard, with water level of 180.50 m, the 1,000-year wave does not overtop the structures.

When considering minimum freeboard (with IDF water level), several structures are overtopped: Lock 45, Dam D and the Upstream Shoreline Wall. In the case of Lock 45 and the Upstream Shoreline Wall, there is slight over-topping during the still water level. However, the amount of 4 cm is considered to be negligible.

During the IDF, some structures could be overtopped by 5 % of the waves. For Lock 45 and Upstream Shoreline Wall, the waves will overtop the crest by 56 cm. For Dam D, the wave, run-up and setup will overtop the structure by 48 cm.

Table 7.8 Existing Freeboard

		Normal Freeboard		Minimum Freeboard	
Water Level		180.50 m	(MNOL)	181.35 m	(IDF)
Wave, Run-Up and Setup Height (m)		0.66 m	95%	0.52 m	95 %
				0.62 m	99 %
Site	**Crest Elevation**	**Available Freeboard**	**Freeboard with wave**	**Available Freeboard**	**Freeboard with wave**
Main Dam	183.02 m	2.52 m	1.86 m	1.67 m	1.05 m
Lock 45	181.31 m	0.81 m	0.15 m	-0.04 m	-0.56 m
Upstream Shoreline Wall	181.31 m	0.81 m	0.15 m	-0.04 m	-0.56 m
Dam D	181.39 m	0.89 m	0.23 m	0.04 m	-0.48 m

*Overtopping of structure (red)

7.2.5 Respect of Freeboard Requirements

For the Main Dam, there is adequate freeboard under both the normal and minimum cases. For Lock 45, Dam D and the Upstream Shoreline Wall, normal freeboard criteria are met, but not minimum freeboard as there is overtopping by waves during the IDF.

However, these are concrete structures and the grade of the fill behind Dam D and the Upstream Shoreline Wall is flat. This is shown in Figure 7.9 for Lock 45 and Upstream Shoreline Wall and Figure 7.10 for Dam D.

As explained in the CDA Hydrotechnical Bulletin (Reference 16), the assessment of the minimum freeboard for concrete structures is rather a function of the maintenance of its integrity.

> *Concrete dams can usually resist substantial overtopping without serious damage. Accordingly, the minimum freeboard requirement may be reduced or overtopping may be allowed provided that the integrity of the dam, its abutments and any ancillary structures is not compromised.*

While there will be some overtopping by waves and accumulation of water in these areas, no damage to the integrity of these structures is likely. This is due to the flat grade on the downstream side, meaning that there will be no velocities or ability for the water to erode the fill. Therefore, overtopping of Lock 45, Dam D and the Upstream Shoreline Wall is not likely to be an issue.

Overtopping of the walkways atop the Lock 45 gates would pose a risk to operators accessing the dam operation area. However, such overtopping by waves would only occur after the water level has reached 181.35 m, the maximum water level under IDF conditions. Operation of the Port Severn Dams aims at maintaining the water level under IDF conditions at 180.57 m. The water level would exceed 180.57 m if certain sluices cannot be fully opened. In practice, should the maximum water level under IDF conditions be reached, all operations would have already taken place and no operator would be at risk while accessing the dam operation area.

Figure 7.9 Fill behind Upstream Shoreline Wall and Lock 45

Figure 7.10 Fill behind Dam D

7.3 Stability Analysis

This section presents the methodology, assumptions and the design criteria that will be used for the stability calculations of the Port Severn Main Dam (Main Dam, Dam D, Lock 45 and Upstream Shoreline Wall). These structures are defined as dams by the Parks Canada Directive and as such are subject to dam stability calculations.

The criteria include:

- Considered loads.
- Loading combinations and diagrams.
- Safety factors.
- Allowable stress.
- Reference to standards and publication.

7.3.1 Considered Loads

7.3.1.1 Dead Loads

The dead load includes weight of concrete (References 24 and 29), weight of water and weight of the backfill, whose densities are as follows:

- γ concrete = 20.60 kN/m³
- γ water = 9.81 kN/m³
- γ backfill (saturated) = 21.0 kN/m³

The weight of the equipment is neglected. The weight of the road traffic on the bridge of the Main Dam is also neglected since it is a live load however the weight of the bridge itself is considered as part of the total weight of the dam.

7.3.1.2 Hydrostatic Pressure

Loads due to hydrostatic pressure are calculated considering the water level on each side of the dam. The upstream and downstream levels are determined by hydraulic studies (Section 6.3.4 - Flood Wave Routing) for normal, unusual and extreme operation conditions. The water level under IDF condition when all sluices are operated is 180.57 m. However, a more realistic operating scenario has been assumed to make allowance for operating difficulties. This considers operating only sluices 2 to 6 with an associated water level of 181.35 m.

The levels are shown in Table 7.9.

Table 7.9 Water Levels at Main Dam Considered for Stability Analysis

Main Dam	Upstream	Downstream
Maximum Operating Level (Summer)	180.50 m	176.48 m
Maximum Operating Level (Winter) [1]	180.40 m	176.48 m
Inflow Design Flood (1,000-year) [2]	181.35 m	177.66 m

[1] Winter levels should be maintained between 180.20 m and 180.30 m, but may exceptionally exceed up to 180.50 m in rare cases. 180.40 m is taken as the maximum operating level for stability analysis.

[2] Maximum upstream water level when sluices only 2 to 6 are open under IDF condition

7.3.1.3 Backfill Pressure and Silt Pressure

The backfill over a retaining wall is considered in our calculations. As there are no on-site measurements, it is assumed that the buoyant backfill pressure is equal to 11 kN/m^3 (unsaturated).

The angle of friction is assumed to be 30° for the backfill.

The silt pressure is not considered in calculations because the underwater video taken during the site inspection shows no signs of silt at the base of the dams.

7.3.1.4 Uplift

For the spillways, the uplift pressure at the foundation (concrete-rock interface) is equal to the headwater, measured from the heel of the base and varies linearly to the tailwater at the toe of the spillway. It is considered to act over 100 % of the foundation of the spillway.

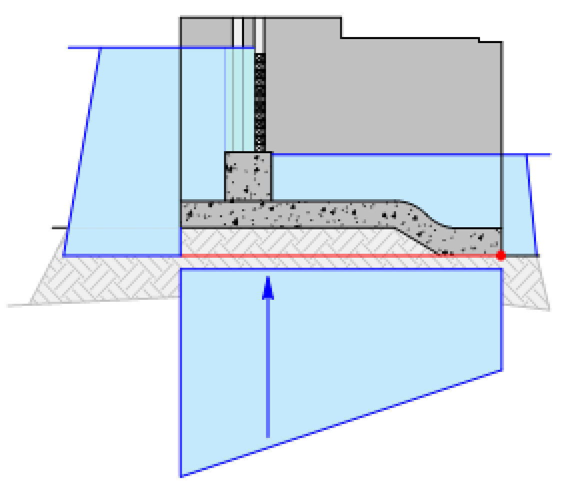

Figure 7.11 Uplift Pressure at the Concrete-Rock Interface

The uplift pressure at the pier base and within the spillway pier (concrete-concrete interface) is equal to the headwater measured at the studied level from the heel of the pier to the stoplogs line over the entire pier width. From there, it varies linearly from the headwater to the tailwater over a distance equal to the width of the pier. This force acts over 2/3 of the pier width. Finally, it is equal to the tailwater down to the toe of the pier, over the entire pier width.

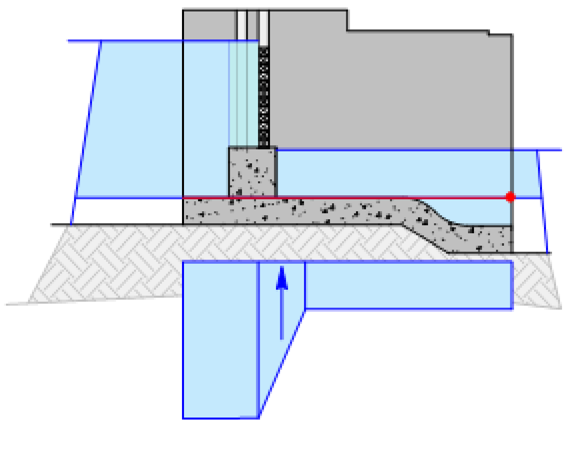

Figure 7.12 Uplift Pressure at the Concrete-Concrete Interface

For the retaining walls, the uplift pressure varies linearly from the headwater at the face of the reservoir to zero water pressure on the backfill. This is valid for both studied interfaces and the pressure acts over 100% of the base (Fig. 9 to 16 App. E2)

In any case, if cracks occur at the base of the structure due to a lack of compression, the uplift pressure will be considered equal to the full water head. This will act along the crack and will vary linearly from this point to the tailwater at the toe of the base. The propagation of that crack will be studied up to the point where it is stable and then the safety factors will be re-evaluated.

In every case, the uplift pressure is not affected by an earthquake due to the brief duration of the phenomenon (References 15, 36, 39 and 59).

7.3.1.5 Static Ice Pressure

Table 7.10 shows the normal, unusual and extreme loads cases for the static ice calculation. They have been calculated using the CEATI Ice Loads Calculation Program. The methodology for these calculations is shown in the Ice Loading Criteria Memorandum, produced in September 2012. A copy of the memorandum is presented in Appendix G. Note that the extreme ice load is used only for verification purposes and is not considered in the recommendations.

All loads will be acting 300 mm below the water level. All loads considered have a return period of 100-years.

The ice loading pressure acting on the existing timber stoplogs is estimated at 30 kN/m. All new timber stoplogs must meet the actual standard of 75 kN/m.

Table 7.10 Summary of Ice Loads

Load Case	Ice Load Value
Normal Ice Load	83.5 kN/m
Unusual Ice Load	125.3 kN/m
Extreme Ice Load	224.9 kN/m

7.3.1.6 Dynamic Ice Pressure

For stability analysis, dynamic ice pressure, resulting from ice impacting the dam structure will not be considered. This is because the flow velocity is low in the reservoir (Gloucester Pool) immediately upstream of the Main Dam.

7.3.1.7 Earthquake

At Port Severn, for the Main Dam, the category of DBE hazard is Significant and the return period is 500 to 1,000 years (Section 6.14.2) The 500-year earthquake chosen as the classification is at the low end of the range (transient use of 12 people).

The mean value of the peak ground acceleration (PGA) is 0.020 g for this probability, as calculated by the Geological Survey of Canada specifically for the Port Severn Dams (Reference 34). The mean value, rather than the median estimate of the PGA, is recommended for the engineering evaluation of a dam (Reference 10). Since this value is given for a firm soil (class C), the PGA has to be divided by the Reference Ground Condition factor (RGC) to obtain a “hard rock” value (Reference 12). For the Port Severn site, the RGC is 1.39 (References 1 and 2) and the resulting PGA for a rock foundation is therefore 0.014 g.

The stability analysis of the structures for seismic loads has been achieved by using a pseudo-static approach. This method is a simplified seismic analysis that can be used for small dams (maximum height of 30 m) and for seismic regions with a PGA ≤ 0.20 g (References 38 and 59). By using this method, it is also assumed that the analysed structure is rigid and that no liquefaction can occur in the dam foundation (References 36 and 38). Considering the fact that the Port Severn Dams are built on rock, no liquefaction of the foundation is possible and the structures will move in unison with the foundation in the event of an earthquake. The analysed concrete dams are also considered to be rigid.

In the pseudo-static method, the seismic accelerations in both horizontal and vertical directions are represented by equivalent static forces. The system above the failure surface is thus considered as a rigid block, and the inertia force associated with the mass of the structure is computed by the principle of the assumed earthquake acceleration (seismic coefficient) times the mass of the rigid block (References 26 and 59).

The horizontal force required to accelerate the concrete mass of the dam can therefore be written as the following equation (References 36 and 59):

$$F_h = \alpha_h \cdot W$$

Where:

- F_h = Horizontal earthquake force
- α_h = Horizontal seismic coefficient
- W = Weight of dam

For the calculation of the pseudo-static forces acting on the Main Dam, a horizontal seismic coefficient equal to 50% of the PGA is considered. A reduction factor of 50 % of the PGA is used to account for the fact that earthquakes in Eastern Canada have a high frequency content. The horizontal seismic coefficient is calculated using the following equation (References 36, 38 and 39):

$$\alpha_h = 0.5 \cdot PGA$$

Hence, the horizontal seismic coefficient (α_h) is equal to 0.0070 g.

The vertical seismic coefficient (α_v) is equal to 2/3 of the horizontal coefficient (References 15, 36 and 38). Since an earthquake produces oscillatory forces, the horizontal PGA and vertical PGA cannot occur at the same time. In the stability calculations, this fact is accounted for by multiplying the vertical seismic coefficient by 30 % when combining seismic horizontal and vertical forces (References 22, 36 and 39). The vertical seismic coefficient (α_v), if used simultaneously with the horizontal seismic coefficient, can therefore be written as:

$$\alpha_v = 0.3 \cdot \frac{2}{3} \alpha_h$$

Consequently, the vertical seismic coefficient (α_v) has a value of 0.0014 g.

The hydrodynamic forces acting on water-retaining structures are calculated according to the following equations of Westergaard (Reference 61):

$$Q_h = 0.726 \cdot (\alpha h \cdot \omega \cdot h \cdot C) \ \cdot y$$

$$MQ_h = 0.299 \cdot (\alpha h \cdot \omega \cdot h \cdot C) \ \cdot y^2$$

Where:

- Q_h = Horizontal hydrodynamic force
- MQ_h = Total overturning moment
- α_h = Horizontal seismic coefficient
- ω = Unit weight of water
- h = Depth of the reservoir
- C = Pressure coefficient for a dam with a vertical face = 0.735
- y = Distance below water surface

No ice pressure is considered when the earthquake loading is calculated. The uplift pressure is not affected by the earthquake due to the brief duration of the event (References 15, 36, 39 and 59).

If a crack occurs at the base of the structure during the earthquake, a post-seismic analysis of each normal loading case will be performed. The same loads will be considered except for the uplift which will consider the full head water along the crack.

7.3.1.8 Drag Force

The drag force is a force applied on the pier structure due to the water flowing around it. This force will not be included in the Dam Safety Review because it is negligible for the structures currently studied. To demonstrate this, the worst case scenario is considered. Below are the relevant maximum values and resultant drag under the

IDF to calculate the drag on the pier. The formula used is from Streeter (Reference 56). Table 7.11 shows the assumptions and results of this analysis.

Table 7.11 Drag Forces on Main Dam Pier

Parameter	Value	Note
Velocity	2.1 m/s	This is the average velocity estimated at the pier face.
Pier height	4.0 – 4.5m	This is taken from the IDF water level to the control structure (the bottom sill below the stoplogs).
Pier width	1.89 m	Width of pier at the widest point.
Drag coefficient	0.9	This is the value for an object between a 2:1 ellipse and a circular object.
Drag force	210 kN	This is the total drag force over 10 piers and 9 spillways.
Force per metre	2.9 kN/m	This is the force per metre along the entire dam length.

It is important to note that in reality, the drag force values will be less than the one calculated above because of the absence of negative pressure behind the pier (due to water falling over the spillway) and of the actual pier wetted height being lower under any other water level condition.

Comparing the normal ice load to the drag force (83.5 kN/m and 2.9 kN/m respectively) shows that the drag is negligible to other forces. Therefore, it will not be considered as a load scenario for stability calculations in the Port Severn DSR.

7.3.1.9 Braking Force

The braking force of the road traffic will not be considered as a significant load in the present analysis since the Canadian Highway Bridge Design Code (Reference 7) recommends its use for the limit states only, which is not the case for stability analysis.

7.3.2 Loading Combinations

Different loading combinations are considered and divided into normal, unusual and extreme loading cases. All the stability analyses are conducted at the foundation of each structure and at different levels within the concrete mass.

7.3.2.1 Spillways

At the Main Dam, two sections of the spillway are studied. Both sections are composed of a pier and of half of the sluice on either side of the pier. Piers 3 and 8 are studied since they respectively represent the highest and the smallest piers of the spillway. Table 7.12 details the loading combinations and Figures 1 to 8 of Appendix E2 illustrate those loading combinations for the spillway.

Table 7.12 Loading Combinations on the Spillway

Normal Loading Cases	
N1 – Maximum operating level in summer	$D + H_1 + U$
N2 – Maximum operating level in winter with normal static ice load	$D + H_2 + I_N + U$
Unusual Loading Cases	
I1 – Maximum operating level in winter with unusual static ice load	$D + H_2 + I_u + U$
I2 – IDF level	$D + H_3 + U$
I3 – Post-seismic analysis of N1	$D + H_1 + U_{pq}$
I4 – Post-seismic analysis of N2	$D + H_2 + I_N + U_{pq}$
Extreme Loading Cases	
E1 – Earthquake	$D + H_1 + Q + U$
E2 – Maximum operating level in winter with extreme static ice load	$D + H_2 + I_E + U$

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

Where:

- D = Dead load.
- H_1 = Hydrostatic pressure with maximum operating level in summer.
- H_2 = Hydrostatic pressure with maximum operating level in winter.
- H_3 = Hydrostatic pressure with Inflow Design Flood level (IDF).
- I_N = Normal static ice load.
- I_u = Unusual static ice load.
- I_E = Extreme static ice load.
- Q = Earthquake load.
- U = Uplift pressure.
- U_{pq} = Uplift pressure in post-seismic condition.

7.3.2.2 Retaining Walls

Dam D, the Upstream Shoreline Wall and the walls of Lock 45 are all considered as retaining walls. All walls are retaining backfill except the left wall of Lock 45 which is retaining the water of the reservoir. Each wall will be studied by choosing a 1 m long section and applying the following loading combinations. Table 7.13 details the loading combinations while Figures 9 to 16 of Appendix E2 illustrate those loading combinations for a typical retaining wall and Figures 17 to 24 illustrate those loading combinations for the left wall of Lock 45.

Table 7.13 Loading Combinations on the Retaining Walls

Normal Loading Cases	
N1 – Maximum operating level in summer	$D + H_1 + B + U$
N2 – Maximum operating level in winter with normal static ice load	$D + H_2 + I_N + B + U$
Unusual Loading Cases	
I1 – Maximum operating level in winter with unusual static ice load	$D + H_2 + I_u + B + U$
I2 – IDF level	$D + H_1 + B + U_{pq}$
I3 – Post-seismic analysis of N1	$D + H_3 + B + U$
I4 – Post-seismic analysis of N2	$D + H_2 + I_N + B + U_{pq}$
Extreme Loading Cases	
E1 – Earthquake	$D + H_1 + B + Q + U$
E2 – Maximum operating level in winter with extreme static ice load	$D + H_2 + I_E + B + U$

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

Where:

- D = Dead load.
- H_1 = Hydrostatic pressure with maximum operating level in summer.
- H_2 = Hydrostatic pressure with maximum operating level in winter.
- H_3 = Hydrostatic pressure with Inflow Design Flood level (IDF).
- I_N = Normal static ice load.
- I_u = Unusual static ice load.
- I_E = Extreme static ice load.
- B = Backfill pressure.
- Q = Earthquake load.
- U = Uplift pressure.
- U_{pq} = Uplift pressure in post-seismic condition.

The water level between the two gates of the lock is considered to be at the downstream level during the summer (H1) and for the IDF (H3). However, it is considered for safety reasons that the water is equal to the upstream level during the winter (H2). If the water level within the lock is lowered (for repairs, interventions or otherwise directed by the Sector Manager), the loading case would be the same as for the summer case (N1) because the downstream water level is apply below the considered rotation point.

7.3.2.3 Stoplogs

At the Main Dam, the capacity of the stoplogs stacked in all sluice is studied. The upper stoplog is under the ice load and the lower stoplog is under the effect of the highest water head only. Only stoplogs in sluice 2 or 3 are studied. Table 7.14 details the loading combinations.

Table 7.14 Loading Combinations Acting on the Stoplogs

Normal Loading Case	
N1 – Maximum operating level in summer	H_3
Extreme Loading Case	
E1 – Maximum operating level in winter with normal static ice load	$H_2 + I_N$

Where:

- H_3 = Hydrostatic pressure with Inflow Design Flood level (IDF).
- I_N = Normal static ice load for stoplogs (30 kN/m see Section 7.3.1.5).

7.3.2.4 Lock Gates

The review of the capacity of lock gates is performed with respect to the CSA-O86 2009 standard. This standard is based on limit state design. The timber material selected is Douglas Fir No.1, which is commonly used for equipment such as the lock gates found at Port Severn. The design is set in a wet environment.

Lock gates are subject to the hydrostatic force acting for long periods of time, during which the force acts continuously. During winter time, additional ice loads act for short periods of time. The action of the hydrostatic load upstream of the downstream gate and the ice loads on the upstream gate are studied since these are the scenarios presenting the largest risk. The worst-case scenario for the ice load is experienced when the upstream lock gate is subject to the ice loading on the upstream reservoir side, while the water level within the lock is lowered (for repairs, interventions or otherwise directed by the Sector Manager). During summer time, the hydrostatic load upstream of the downstream gate reaches 7.32 m.

A lock gate is made up of stoplogs stacked on top of each other and held together by steel rods. For the purpose of this study, only the action of the hydrostatic load on the lower log and the action of the ice load on the upper stoplog is studied. Stoplogs are studied as isolated elements, not part of a group. In reality, part of the load pushing on a log is transferred to the others through the steel rods. Also, an element of the load pushing on a stoplog is countered by the friction force acting on the log. As wet timber absorbs water, it expands and seals the gap between itself and the other logs or bottom sill.

Table 7.15 details the loading combinations.

Table 7.15 Loading Combinations Acting on the Lock Gates

Normal Loading Case	
N1 – Maximum operating level in summer	H_3
Extreme Loading Case	
E1 – Maximum operating level in winter with normal static ice load	$H_2 + I_N$

Where:

- H_3 = Hydrostatic pressure with Inflow Design Flood level (IDF).
- I_N = Normal static ice load for stoplogs (30 kN/m see Section 7.3.1.5).

7.3.3 Stability Factors

In order to determine if a structure is stable, stability factors are used to compare results with acceptance criteria given in the reference publications.

Those stability factors are:

- The sliding factor.
- The uplift factor.
- The allowable stress over the analyzed section.
- The position of the resulting force.

7.3.3.1 Sliding Factor

The sliding factor is calculated as follows (References 39 and 15):

$$S.F. = \frac{(\Sigma Fv)\cdot tan\phi + (C\cdot Ac)}{\Sigma Fh}$$

Where:

- ΣFv = Sum of vertical forces
- $\tan\phi$ = Fiction coefficient
- C = Cohesion at the interface
- Ac = Area of the section in compression
- ΣFh = Sum of horizontals forces

For each structure, the stability analysis is carried out at the concrete-rock interface and at different levels through the mass of the structure (concrete-concrete interface).

Table 7.16 presents the values for cohesion (C) and of the angle of friction (ϕ) for each structure. These are taken from References 24 and 39. Table 7.17 then follows with the allowable sliding factors, as per the CDA Guidelines (Reference 8). The acceptable peak sliding factors consider the friction and the cohesion of the interface (no test) while the residual sliding factors consider only the friction of the interface.

All the analysis are conducted assuming the cohesion values presented in Table 7.16 for the peak sliding but the minimum cohesion value required to reach the factor for the peak sliding is also calculated for comparison.

Table 7.16 Selected Sliding Factors

Structures	Interface	Peak Sliding	Residual Sliding
For all structures (gneiss)	Concrete-rock	C = 350 kPa ϕ = 40°	C = 0 kPa ϕ = 34°
For all structures	Concrete-concrete	C = 300 kPa ϕ = 55°	C = 0 kPa ϕ = 45°

Table 7.17 Acceptable Sliding Factors

Loading Cases	Peak Sliding	Residual Sliding
Normal	> 3.0	> 1.5
Unusual	> 2.0	> 1.3
Extreme	> 1.3	> 1.1

7.3.3.2 Uplift Factor

The uplift factor is calculated as follows (Reference 39):

$$U.F. = \frac{(\Sigma Fv - U)}{U}$$

Where:

- ΣFv = Sum of vertical forces
- U = Uplift pressure

The minimum acceptance criteria for the uplift factor are respectively 1.2, 1.1 and 1.1 for the Normal, Unusual and Extreme loading cases (Reference 36)

7.3.3.3 Allowable Stress

At the base of the structure (concrete-rock interface) no tensile stress is acceptable. Within the structure (concrete-concrete interface), tensile stress may be acceptable as long as it is limited to 5 % of the concrete compressive strength (denoted as f'_c) and that all other acceptance factors are met (References 15 and 39). If the tensile stresses should exceed these values, a crack is assumed at the heel of the base and the propagation of that crack will be studied up to the point where it is stable.

The compressive stresses at the toe of the structure should be limited to 0.3 f'_c under normal loading cases, to 0.5 f'_c under unusual loading cases and to 0.9 f'_c under extreme loading cases for both interfaces (References 8 and 15).

According to the investigation reports available (References 24 and 29), the concrete compressive strength (f'_c) for each structure is considered as follows:

- Main Dam = 20 MPa.
- Lock 45 = 14 MPa.
- Dam D = 12 MPa (taken as the lowest of the two sample results of 13.3 and 12.1).
- Upstream shoreline wall = 14 MPa.

7.3.3.4 Position of the Resultant Force

The resultant force acts:

- in the middle third of the base for the normal loading cases (100 % of the base in compression),
- in the median half of the base for the unusual loading cases (75 % of the base in compression) and
- within the base for the extreme loading cases (Reference 15).

7.3.4 Stability Review Results

7.3.4.1 Main Dam – Pier 3

Pier 3 is representative of all the highest piers (piers 2, 4 and 5) because they are similar in dimension and are exposed to the same loads. The analysed section is composed of the pier and of half of the sluice on either side of the pier.

For the concrete-rock analysis, the base of the section considered includes the base of the pier and the base of the sluice on each side. For the concrete-concrete analysis, the base of the section only considered the base of the pier itself. The hydraulic and ice pressures are considered on the pier and on the stoplogs on either side of the pier because those horizontals forces will be transferred to the pier and therefore cannot be neglected.

Table 7.18 presents a summary of results for the loading cases considered for pier 3.

Table 7.18 Summary of Results on Pier 3

Load Case	Interface Showing Worst Results	Safety Factor		Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
		Uplift	Sliding				
N1	Concrete-rock	1.50	17.03	-3.91	Median Half	92 %	Acceptable
N2	Concrete-rock	1.41	6.35	-23	Within the base	42 %	Not stable
I1	Concrete-rock	1.51	5.28	-33	Within the base	38 %	Not stable
I2	Concrete-rock	1.55	19.18	2.58	Middle third	100%	Stable
I3	Concrete-rock	1.50	16.90	0.46	Median Half	92 %	Stable
I4	Concrete-rock	1.41	14.25	-22	Within the base	42 %	Not stable
E1	Concrete-rock	1.50	16.33	-5.13	Median Half	92 %	Stable
E2	Concrete-rock	1.22	0.39	-56	Within the base	1 %	Not stable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

Pier 3 is not stable at the concrete-rock interface (Table 1.1 of Appendix E1). Safety factors for the uplift and the peak sliding are respected for all load cases, except for E2. However, the position of the resultant force does not meet the standards as required under all the normal and unusual loading cases.

Since Main Dam is an existing dam, it might be acceptable to allow a small percentage of the base to be under tension under normal loading cases, if all other criteria are met (Reference 15). Case N1 shows that only 8 % of the base is in tension, which could be acceptable and considered stable. However, under all ice loading cases (N2, I1 and E2), more than half of the base is in tension causing the pier to be unstable. Therefore, piers 2, 3, 4, and 5 must be stabilized for ice loading.

A permanent option to stabilize the piers is to install a post-tensioned anchor in each pier, as shown on Figure 7.13. Table 1.1A of App. E1 shows that the only case encountering tension forces with the anchor installed would be the extreme ice load (E2). However, such stress is acceptable for an extreme load case.

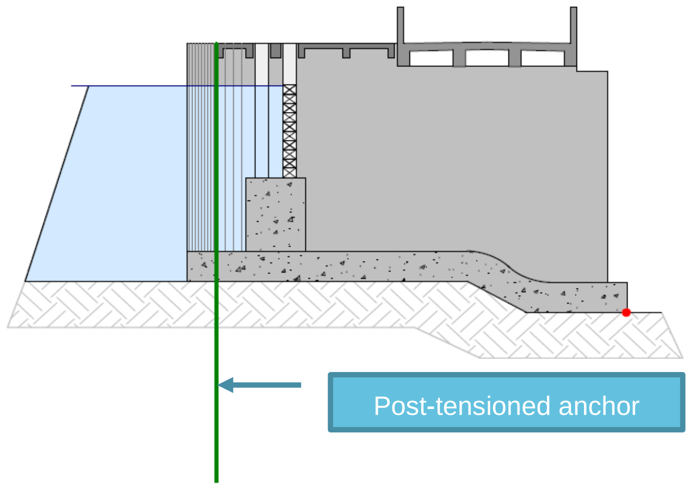

Figure 7.13 Recommended Remedial Work at Main Dam - Post-Tensioned Anchor in the Pier

A temporary option to stabilize the piers is to add concrete to the base of the spillway and to reduce the ice pressure by mechanical means. This option might be explored during the detailed engineering but it remains a non-permanent solution that requires maintenance and surveillance, while the installation of anchors is a permanent, low maintenance solution.

At both the concrete-concrete interfaces (Table 1.2 and 1.3 of App. E1), tension is acceptable. This is because the bonded concrete lift-joint can develop a tensile strength estimated to approximately 0.05 f'_c. Under these conditions and under all load cases, the structure is stable and the results show the forces needed to stabilize each load. However, installation of the post-tensioned anchors would increase the factor of safety for the concrete-concrete interface considerably.

7.3.4.2 Main Dam – Pier 8

Pier 8 is representative of all the smallest piers (piers 6, 7 and 9) because they are similar in dimension and are exposed to the same loads. The analysed section is composed of the pier and of half of the sluice on either side of the pier.

For the concrete-rock analysis, the base of the section considered includes the base of the pier and the base of the sluice on each side. For the concrete-concrete analysis, the base of the section only considered the base of the pier itself. The hydraulic and ice pressures are considered on the pier and on the stoplogs on either side of the pier because those horizontals forces will be transferred to the pier and therefore cannot be neglected.

Table 7.19 presents a summary of results for the loading cases considered for pier 8.

Table 7.19 Summary of Results on Pier 8

Load Case	Interface Showing Worst Results	Safety Factor Uplift	Safety Factor Sliding	Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
N1	Concrete-rock	1.90	34.80	12	Middle Third	100 %	Stable
N2	Concrete-rock	1.91	21.96	-0.39	Median Half	98 %	Acceptable
I1	Concrete-rock	1.56	11.34	-6.85	Within the base	59 %	Not stable
I2	Concrete-rock	1.30	30.38	14	Middle Third	100 %	Stable
I3	Concrete-rock	---------- N/A ----------					
I4	Concrete-rock	---------- N/A ----------					
E1	Concrete-rock	1.90	33.55	11	Middle Third	100 %	Stable
E2	Concrete-rock	1.22	0.35	-22.25	Within the base	1 %	Not stable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

For pier 8, the results are similar to those of pier 3.

Since Main Dam is an existing dam, it might be acceptable to allow a small percentage of the base to be under tension under normal loading cases, if all other criteria are met (Reference 15). Case N2 shows that only 2 % of the base is in tension, which could be acceptable and considered stable. However, under the other two ice loading cases (I1 and E2), more than half of the base is in tension causing the pier to be unstable. Therefore, pier 8, as well as piers 6, 7 and 9, must be stabilized for ice loading.

Akin to what is recommended for pier 3, post-tensioning anchors could be installed to stabilize these piers too (Table 2.1A of App. E1).

At the concrete-concrete interface on pier 8 (Table 2.2 of App. E1), the pier is stable under all load cases. This is because of the tensile strength considered at the bonded lift-joint, which is estimated at approximately 0.05 f'_c. Under these conditions and under all load cases, the structure is stable. However, installation of the post-tensioned anchors would increase the factor of safety for the concrete-concrete interface considerably.

7.3.4.3 Lock 45 Left Wall

The left wall of the lock chamber is analysed over a width of 1 m and the rotation point of the section is taken on the inside of the lock chamber. The first analysis (Table 3.3 of Appendix E1) is done at the level of the rock in the reservoir at the concrete-rock interface. The second analysis is done at the bottom of the inside chamber, at the concrete-concrete interface (Table 3.4 of Appendix E1).

Table 7.20 presents a summary of results for the loading cases considered for the left wall of Lock 45.

Table 7.20 Summary of Results on Lock 45 Left Wall

Load Case	Interface Showing Worst Results	Safety Factor		Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
		Uplift	Sliding				
N1	Concrete-Rock	7.99	31.38	33	Middle Third	100 %	Stable
N2	Concrete-Rock	8.25	none	56	Middle Third	100 %	Stable
I1	Concrete-Rock	8.25	none	56	Middle Third	100 %	Stable
I2	Concrete-Rock	6.28	19.80	1.71	Middle Third	100 %	Stable
I3	Concrete-Rock	-- N/A --					
I4	Concrete-Rock	-- N/A --					
E1	Concrete-Rock	7.98	29.68	31	Middle Third	100 %	Stable
E2	Concrete-Rock	8.25	none	56	Middle Third	100 %	Stable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

At the concrete-rock interface (Table 3.3 of App. E1) the left wall is stable under all load cases. For all three ice load cases (N2, I1 and E2) the water and the ice pressures in the reservoir and in the lock are considered equal therefore canceling each other. All three load cases are then identical and the safety factor for the peak sliding cannot be computed.

At the concrete-concrete interface (Table 3.4 of App. E1) both walls are separated by a backfill but attachted to one another by tie-rods which are there to cancel the backfill pressure on the walls. The analysed section considere only the wall at the left of the chamber because it is the narrower and always under water pressure.

The analysed section is stable under all load cases. This is due to the tensile strength of the bonded lift-joint, which has been estimated at 0.05 f'_c. But for the unusual and extreme ice load cases the tensile strength is not enough and a crack is assumed in the lift-joint, but since there is movement of the wall, the ice pressure is reduced and the crack does not propagated more into the lift-joint.

A second analysis is done on the left wall of Lock 45 towards the stairs.

According to information provided by PWGSC, a repair was carried out on the top of the left wall in 1985, downstream of the lock chamber. The left wall was demolished down to a depth of 2 m from the crest over the width of the wall. It was reconstructed afterward but no information is available regarding connection between the two different concrete pours.

Figure 7.14 illustrates the left wall of Lock 45 and the location of the cracks and joints.

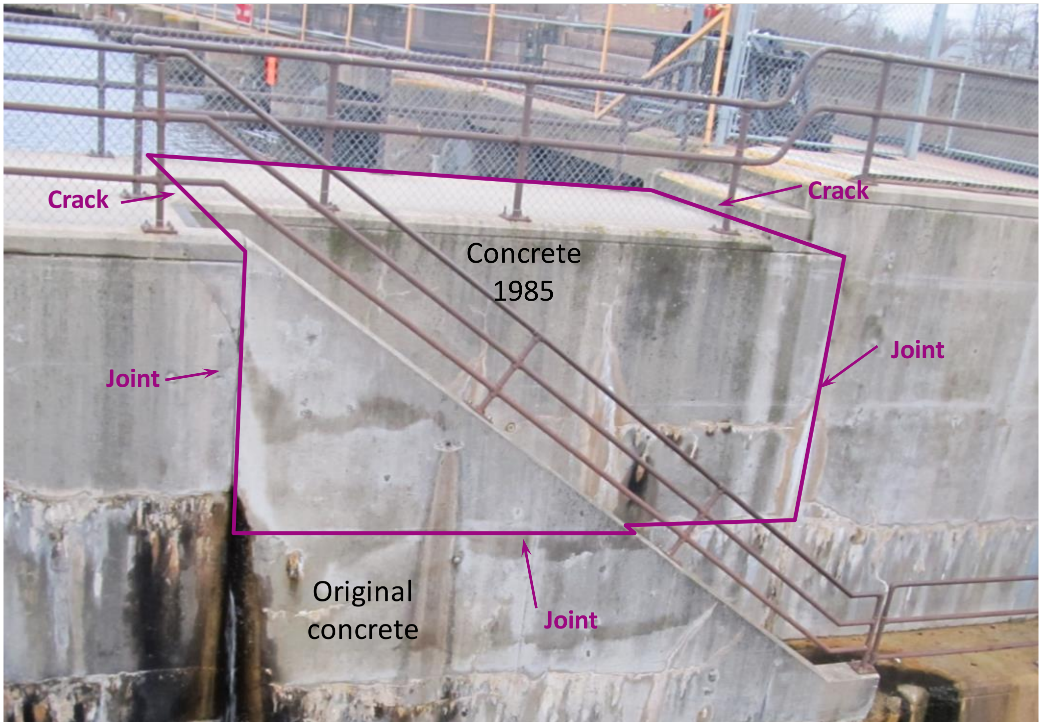

Figure 7.14 Lock 45 Left Wall – Identified Issue

The worst case scenario concerning this repair would be when the original and rehabilitated (1985) concrete sections are not properly joined, by way of anchors or a shear key. In this case, only the friction between the two concrete sections (in the horizontal joint) would maintain the block in place under water and ice pressure from the reservoir. This friction is sufficient to resist the load when there is water pressure on the wall. However, there is a sliding of the block toward the lock chamber under all ice loads.

Therefore, work is needed to limit the current damage at the construction joints. Dowels need to be placed to ensure that the block of concrete won't move under the ice pressure.

7.3.4.4 Lock 45 Right Wall

The right wall of the lock chamber is analysed over a width of 1 m. The rotation point of the analysed section is taken on the inside of the lock chamber. The first analysis (Table 3.1 of Appendix E1) is done at the level of the rock behind the wall, at the concrete-concrete interface, because the failure surface at this level is smaller than the one between the concrete wall and the rock wall. A second analysis is done at the level of the concrete step, at the concrete-concrete interface. Table 7.21 presents a summary of results for load cases considered for the right wall of Lock 45.

Table 7.21 Summary of Results on Lock 45 Right Wall

Load Case	Interface Showing Worst Results	Safety Factor Uplift	Safety Factor Sliding	Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
N1	Concrete-concrete	4.93	1074	35.15	Middle Third	100 %	Stable
N2	Concrete-concrete	8.07	13.08	159	Middle Third	100 %	Stable
I1	Concrete-concrete	10.49	9.39	214	Middle Third	100 %	Stable
I2	Concrete-concrete	3.64	41.01	69	Middle Third	100 %	Stable
I3	Concrete-concrete	N/A					
I4	Concrete-concrete	N/A					
E1	Concrete-concrete	5.13	175.05	41	Middle Third	100 %	Stable
E2	Concrete-concrete	16.42	6.23	348	Middle Third	100 %	Stable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

The right wall of Lock 45 is stable under all load cases at both concrete-concrete interfaces (Table 3.1 and 3.2 of App. E1). This is due to the tensile strength of the bonded lift-joint, which has been estimated at 0.05 f'_c. None of the loading cases require more than half of the maximum tensile strength to stabilize the wall.

7.3.4.5 Lock Gates

Table 7.22 presents a summary of results for load cases considered for the Lock Gates.

Table 7.22 Summary of Results on the Lock Gates

Load Case	Water Level	Sill Level	Ice Force	Hydrostatic Pressure at Sill Level	Factor of Safety		Acceptable
N1	181.35 m	177.09 m	------	39 kPa	Bending: hydrostatic pressure	3.29	Yes > 1.5
E1	180.40 m	177.09 m	30 kN/m	32.9 kPa	Bending: ice pressure	1.82	Yes > 1.5

For this case, the bending stress in the middle of the log and the shearing and compression stresses at the ends of the log are verified. The results of the evaluation of the theoretical capacity show that the lock gates, under their current configuration, meet the CSA-O86 standard.

7.3.4.6 Upstream Shoreline Wall

The highest section of the wall is analysed over a width of 1 m and the rotation point is taken on the face of the wall in contact with the reservoir. Table 7.23 presents a summary of results for load cases considered for the Upstream Shoreline Wall.

Table 7.23 Summary of Results on the Upstream Shoreline Wall

Load Case	Interface Showing Worst Results	Safety Factor Uplift	Safety Factor Sliding	Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
N1	Concrete-rock	4.85	136.25	47	Middle Third	100 %	Stable
N2	Concrete-rock	5.01	6.40	1.55	Middle Third	100 %	Stable
I1	Concrete-rock	5.01	6.96	181	Within the base	26 %	Acceptable
I2	Concrete-rock	3.82	32.06	0.65	Middle Third	100 %	Stable
I3	Concrete-rock	N/A					
I4	Concrete-rock	N/A					
E1	Concrete-rock	4.85	34.87	90	Middle Third	100 %	Stable
E2	Concrete-rock	5.01	67.83	884	Within the base	47 %	Acceptable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

At the concrete-rock interface (Table 4.1 of App. E1) the wall is stable under the summer (N1) and the earthquake (E1) load cases.

Under all other cases, the hydrostatic pressure and the ice pressure are strong enough to cause a movement of the wall toward the backfill. The backfill then behaves like passive force in response to that movement.

Under the normal ice load case (N2) and the IDF load case (I2), the passive earth pressure coefficients (Kp) required to stabilize the wall are 2.61 and 1.12 respectively. These are below the maximum value of Kp of 3.0. Therefore both cases are stable and have the appropriate passive force.

Under the unusual and extreme ice load cases (I1 and E2) the maximum abutment pressure is not strong enough to stabilize the wall and a crack is assumed downstream of the rotation point (Fig 9 to 16 of App. E2). However, once the wall has moved toward the backfill, the contact between the ice cover and the wall is lost and the ice pressure will no longer act to destabilize the wall. Those cases are considered acceptable.

At the concrete-concrete interface (Table 4.2 of App. E1) the upstream shoreline wall is stable because the maximum tensile strength of the bounded lift-joint is estimated at 0.05 f'_c.

Under the extreme ice load case (E2) the wall moves toward the backfill even though the maximum tensile strength of the joint and the maximum abutment pressure are not high enough to resist the pressure of the ice. However, once the wall has moved toward the backfill, the contact between the ice cover and the wall is lost and the ice pressure will no longer act to destabilize the wall.

The Upstream Shoreline Wall is stable under all loading scenarios.

7.3.4.7 Dam D

The highest section of the dam is analysed over a width of 1 m and the rotation point is taken on the face of the dam in contact with the reservoir. Table 7.24 presents a summary of results for load cases considered for Dam D.

Table 7.24 Summary of Results on Dam D

Load Case	Interface Showing Worst Results	Safety Factor Uplift	Safety Factor Sliding	Allowable Stress	Position of the Resulting Force	Length of the Base in Compression	Conclusion of the Analysis
N1	Concrete-rock	4.99	1906	21	Middle Third	100 %	Stable
N2	Concrete-rock	5.18	11.09	104	Middle Third	100 %	Stable
I1	Concrete-rock	5.18	10.11	219	Within the base	34 %	Acceptable
I2	Concrete-rock	3.80	42.09	2.28	Middle Third	100 %	Stable
I3	Concrete-rock	N/A					
I4	Concrete-rock	N/A					
E1	Concrete-rock	4.98	54.86	58	Middle Third	100 %	Stable
E2	Concrete-rock	5.18	14.07	877	Within the base	47 %	Acceptable

Note : Case E2 is used only for verification purposes and is not considered in the recommendations

At the concrete-rock interface (Table 5.1 of App. E1) the dam is stable for the summer (N1) and earthquake (E1) load cases.

Under all other load cases, the hydrostatic pressure and the ice pressure are strong enough to cause a movement of the dam toward the backfill. The backfill then behaves like a passive force in response to that movement.

Under the normal ice load case (N2) and the IDF load case (I2) the passive earth pressure coefficients (Kp) required to stabilize the wall are 2.75 and 1.05 respectively which are below the maximum value of Kp of 3.0. Therefore, both cases are stable and have the appropriate passive force.

Under the unusual and extreme ice load cases (I1 and E2) the maximum abutment pressure is not strong enough to stabilize the wall and a crack is assumed downstream of the rotation point (Fig 9 to 16, App. E2). However, once the wall has moved toward the backfill, the contact between the ice cover and the wall is lost and the ice pressure is no more a force that can eventually destabilize the wall. Those cases are considered acceptable.

At the concrete-concrete interfaces (Table 5.2 of App. E1) the dam is stable because of the maximum tensile strength of the bonded lift-joint that is estimate at 0.05 $f'_{c.}$ The results show the forces needed to stabilize each loading case.

7.3.4.8 Stoplogs

Table 7.25 presents a summary of results for load cases considered for the stoplogs.

Table 7.25 Summary of Results on the Stoplogs

Load Case	Water Level	Sill Level	Ice Force	Hydrostatic Pressure at Sill Level	Factor of Safety		Acceptable
N1	181.35 m	176.77 m	------	45 kPa	Bending: hydro pressure	1.06	No < 1.5
E1	180.40 m	176.77 m	30 kN/m	35.6 kPa	Bending: ice pressure	0.83	No < 1.5

The review of stoplog capacity is performed with respect to the CSA-O86 2009 standard. This standard is based on limit state design. The timber material selected is Douglas Fir No.1, which is commonly used for equipment such as the stoplogs at Port Severn. The stability assessment considers new stoplogs in a wet environment.

Stoplogs are subject to the hydrostatic force for long periods of time, during which the force acts continuously. During winter time, additional ice loads act for short periods of time. Table 7.14 details the loading combinations acting on the stoplogs. Only the action of the hydrostatic load on the bottom log of sluices 2 or 3 and the action of the ice load on the upper stoplog are studied. Stoplogs are studied as isolated elements, not part of a group. In reality, a part of the load pushing on a stoplog is countered by the friction force acting on the same log. As wet timber absorbs water, it expands and seals the gap between itself and the other logs or the bottom sill.

For each log, the bending stress in the middle of the log and the shearing and compression stresses at the ends are verified. The results of the evaluation of the theoretical capacity show that the stoplogs, under their current configuration, do not meet the CSA-O86 standard. Under the hydrostatic load, the bending stress exceeds the allowed limit by 30 %. Under the ice load, the bending stress limit is exceeded by 45 % and the shearing, by 18 %.

Exceedance of the limits set by the standard doesn't imply failure. For example, the logs currently in place do not yield under the existing hydrostatic loads. Computations performed as per CSA-O86 make use of safety factors that reduce allowable loads.

As explained earlier, logs are supported by the friction force acting between them. The failure of a log could cause a chain-reaction where all logs in the sluice would fail. This would be considered a dam break and could cause damage downstream.

7.3.5 Post Seismic Deformation

During an earthquake, Main Dam's pier 3 is the only structure assumed to have the tensile force acting at the concrete-rock interface (Table 1.1 of App. E1). As the installation of post-tensioning anchors has been recommended to keep the dam stable under every load case, there will be no post-seismic deformation.

8. Water Impoundment Review

The Water Impoundment Review identifies locations on the reservoir rim where water can flow out of the reservoir under MNOL and flood condition. These locations correspond to topographic points that are at a lower elevation than the maximum water level during the passage of the Inflow Design Flood.

A visit to Port Severn was undertaken on November 5th 2012. The purpose of the visit was to observe areas identified as openings in the reservoir rim. In addition, the visit sought to determine where water was flowing into and out of the reservoir including the inspection of several culverts, photos of which are shown in Appendix B4.

The IDF selected and their maximum corresponding water levels for the dams at Port Severn are shown in Table 8.1.

Table 8.1 Inflow Design Flood for Dams at Port Severn

Structure	Inflow Design Flood	Upstream Water Levels
Main Dam	1:1,000	181.35 m
Dam C	1:1,000	181.35 m
Dam E	1:100	180.50 m
Dam G	1:100	180.50 m

The Main Dam, Dam E and Dam G can safely pass the 100–year flood without an increase in operating water level (180.50 m). Therefore, there is no issue with the closure of the reservoir rim under the 100–year flood (the IDF associated with Dam E and Dam G).

Given the above, the IDF used in this analysis is the 1,000-year flood. Under these flood conditions, the water level can reach 181.35 m in Gloucester Pool. This elevation has been validated during the dam break analysis and dam classification (Section 6.3).

8.1 Digital Elevation Model

This section describes the digital elevation model (DEM) used in the Port Severn Impoundment Review. This section also describes the limitations with the DEM and the methods used to resolve the limitations.

Analysis involving the impoundment of water at Gloucester Pool/Little Lake requires that an accurate representation of the surrounding topography be used. The elevation of points on the reservoir rim needs to be accurately described to determine where water will actually flow under flood conditions.

For Port Severn, a DEM has been used to identify these areas. The DEM is a cost effective and reliable way to understand points on the reservoir which may be subject to overflow and is easily analysed with common Geographic Information System (GIS) software, such as ESRI ArcMap.

The DEM used for the analysis is derived from the Ontario Base Topographical maps (Southern Ontario, 1:10,000; Northern Ontario, 1:20,000) and referenced to the NAD83 Datum. The cell resolution is 10 meters, which means that the DEM is composed of tiles of 10 m by 10 m, each of them associated with a unique z-value (elevation). The vertical accuracy of this data is in the order of ± 2.5 m.

The Port Severn site was surveyed in May 2012. This gave accurate topographical information for the surveyed areas, which were then used to update areas in the DEM. This rebuilt DEM was used for the analysis of the reservoir rim and is therefore more accurate near the areas surveyed (near the dam sites).

For areas outside of the surveyed zones the DEM is limited in accuracy. An analysis of the difference between the surveyed and DEM points shows an average difference of approximately ±1.2 m.

In areas where survey was not available, several water level elevations on the DEM were considered. This was to determine if areas indicated as gaps in the reservoir rim were an issue or merely the result of the limited accuracy of the DEM.

To overcome this limitation, several elevations above and below the 181.35 m elevation were analysed and mapped to determine if there was a potential opening in the reservoir. The result is a representation of the topography, where the water path is clearly visible and the effect of differing water elevations can be understood. The elevations used and a visual example of the analysis are shown in Figure 8.5 and Figure 8.6.

It is important to note that the topographic survey is generally higher in elevation than the sampled DEM elevation. This difference is considered conservative because the areas which are shown on the DEM as gaps in the reservoir rim may be at a higher elevation in reality.

8.2 Reservoir Rim and Impoundment Review

This section outlines the areas which appear to be an opening in the reservoir rim under the IDF conditions. The different elevations used and a visual example of the analysis are shown in Figure 8.5 for Areas 1 to 3 and Figure 8.6 for Area 5. Table 8.2 summarizes the areas identified as potential openings in the reservoir rim.

Table 8.2 Description of Potential Openings in the Reservoir Rim

Area	Description
Areas 1, 2 and 3	• These areas correspond to the areas where water can flow under Highway 400. These areas consist of lakes and swampy land interconnected with culverts. Gentle flow was observed during a site inspection (on November 5, 2012), however the direction of the flow was not obvious. • At the maximum water level under IDF conditions of 181.35 m, water will flow out of the reservoir at these points. This is confirmed by analysis shown in Figure 8.5, which shows potential paths for water toward the north-west to Georgian Bay based upon the DEM. • Images of these areas are shown in Appendix B4.
Area 4	• This area is an opening in the reservoir rim under Port Severn Road. The opening is formed by a CSP750 culvert which will act as an outlet during the IDF. • Downstream of the culvert, the resultant flow enters Georgian Bay upstream of the Highway 400 Bridge.
Area 5	• Area 5 is an area where water forms a loop around several lakes and swampy areas at the maximum water level under IDF conditions of 181.35 m. While technically not an opening in the reservoir rim, this area allows water to flow into Area 4, where there is an outlet. • In this area, there is a possibility for the water to flow out of the reservoir under a water level of 182 m to 184 m. This analysis and range of levels are mentioned because the accuracy of the DEM is limited, as discussed in Section 8.1.

A map of Gloucester Pool and Little Lake with potential reservoir openings is shown in Figure 8.1. A detailed view of Area 4 is shown in Figure 8.2. Figure 8.3 shows a detailed view of Area 3. A general view of Areas 1 – 3 is shown in Figure 8.4. Figure 8.5 and Figure 8.6 show location of potential openings at areas 1 to 3 and expected flow directions as well as DEM surface elevations.

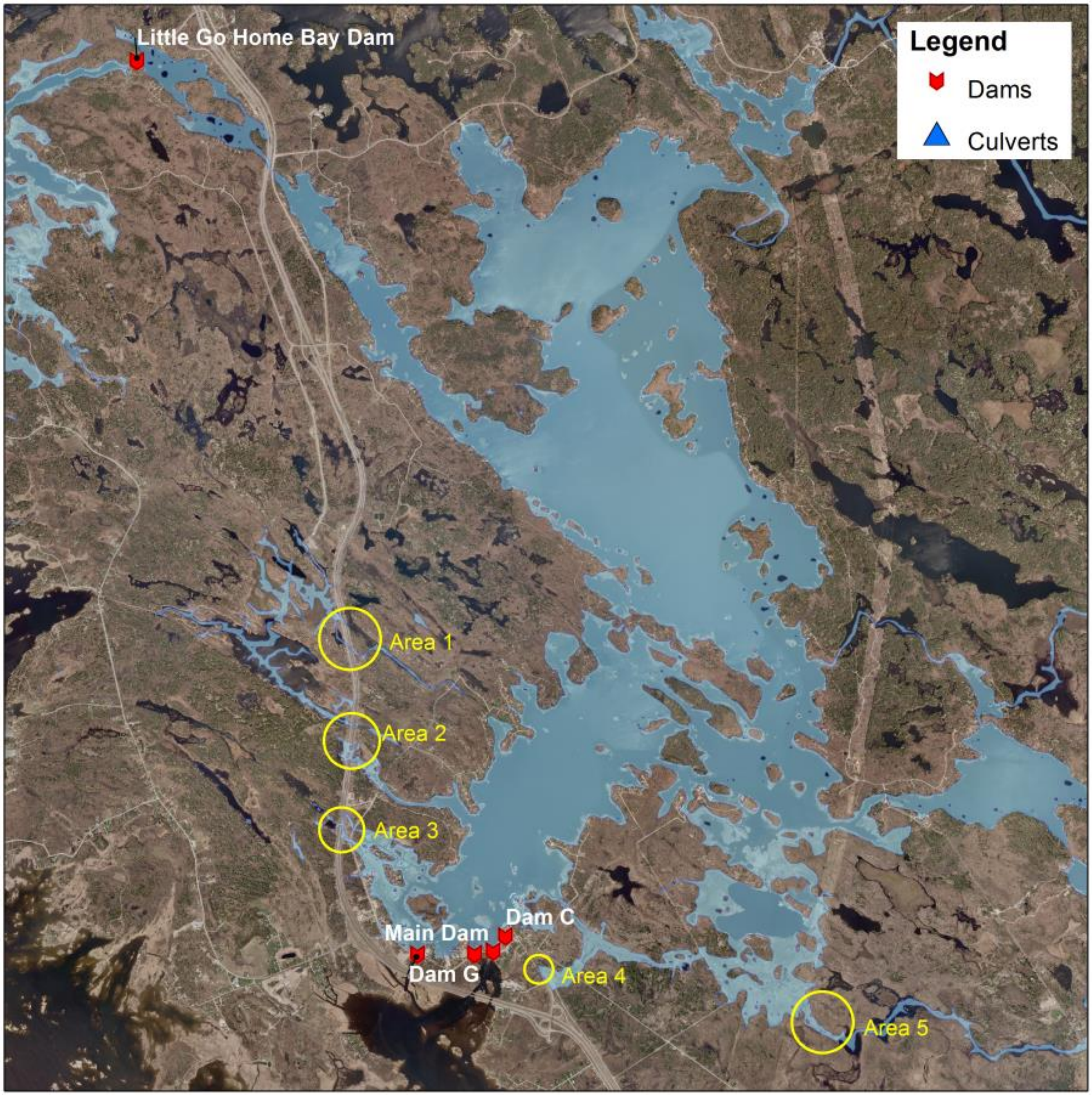

Figure 8.1 Openings in Reservoir Rim at 181.35 m

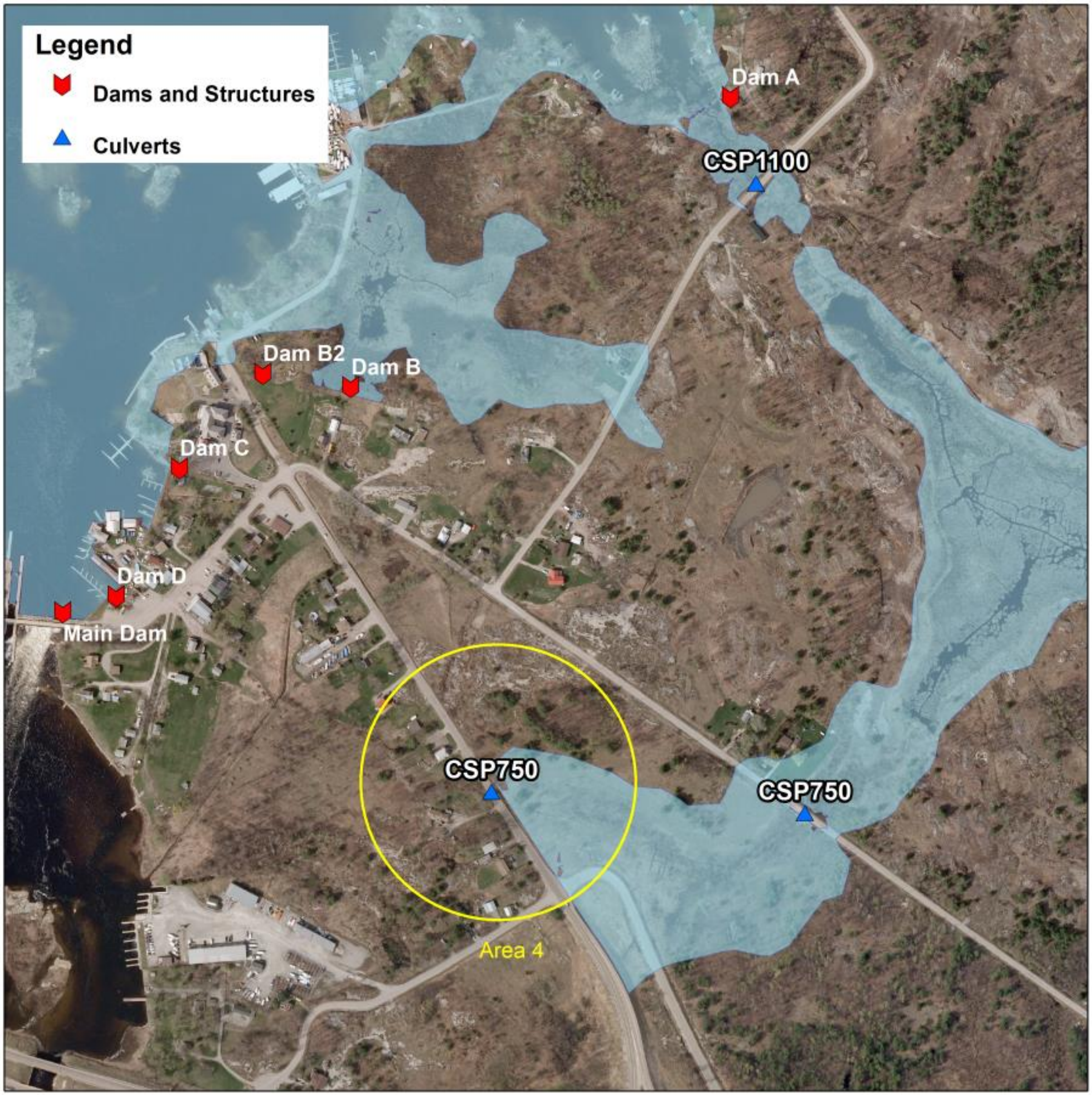

Figure 8.2 Openings in Reservoir Rim at 181.35 m (Area 4)

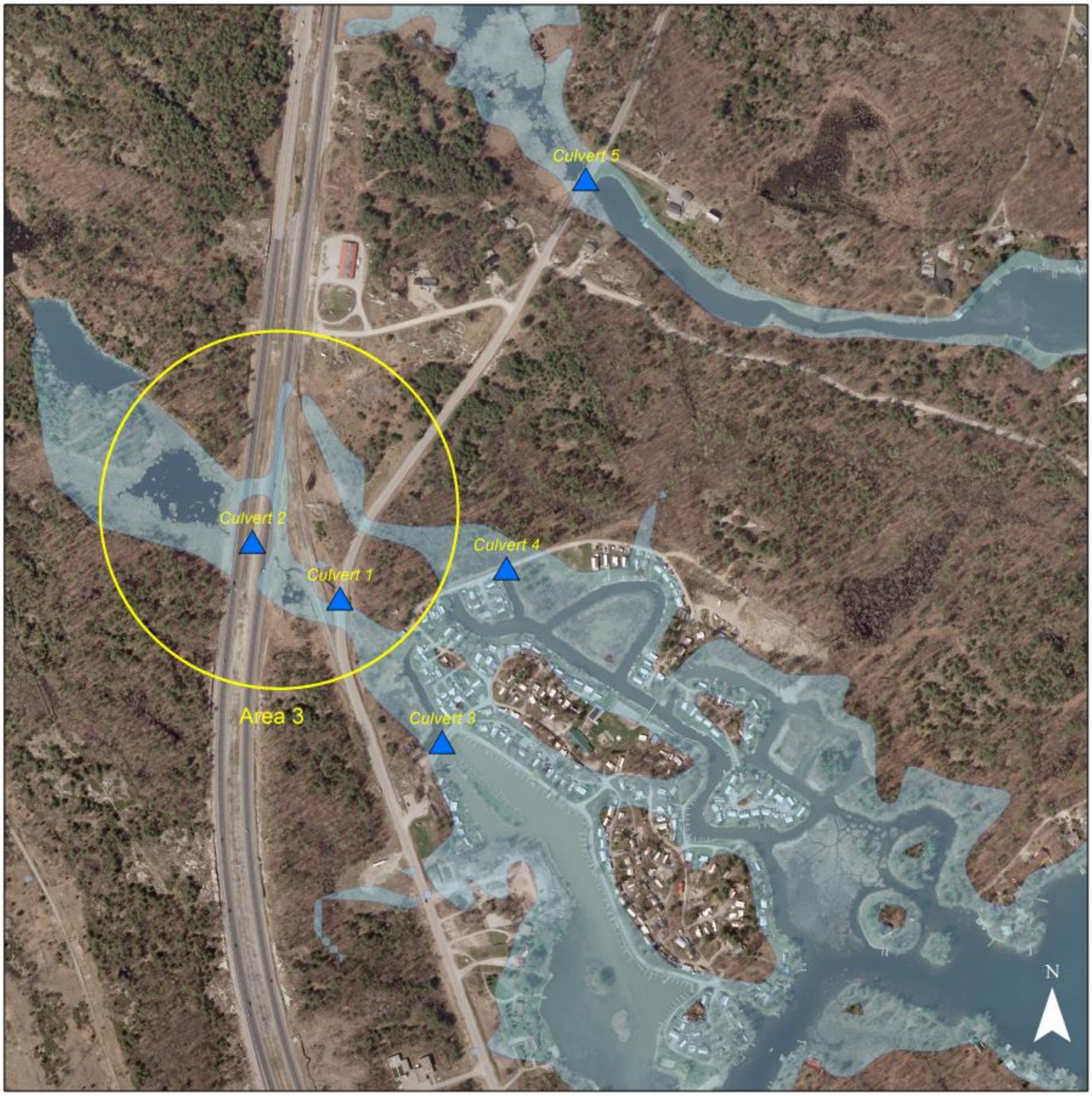

Figure 8.3 Openings in Reservoir Rim at 181.35 m (Area 3)

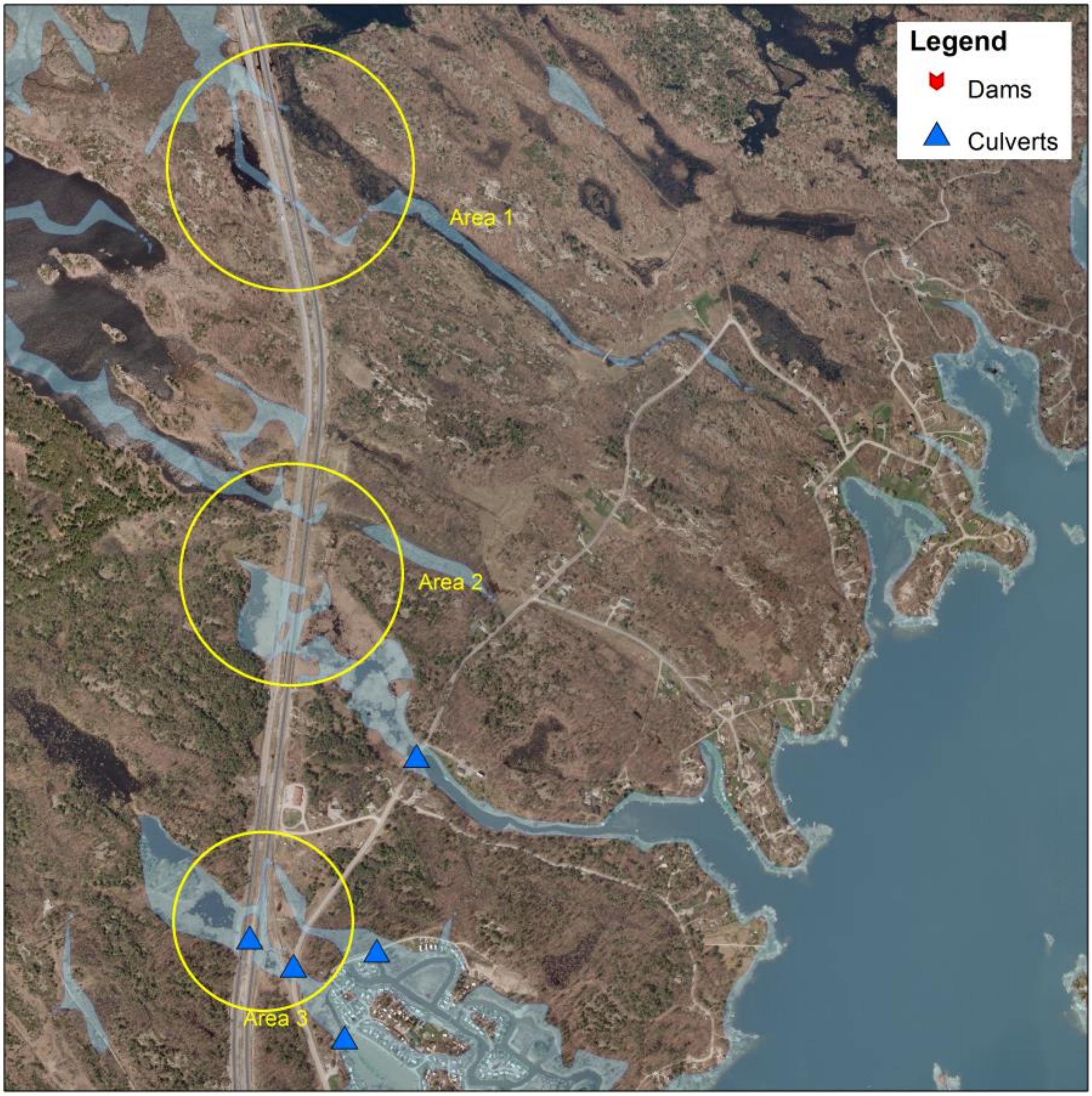

Figure 8.4 Openings in Reservoir Rim at 181.35 m (Areas 1 to 3)

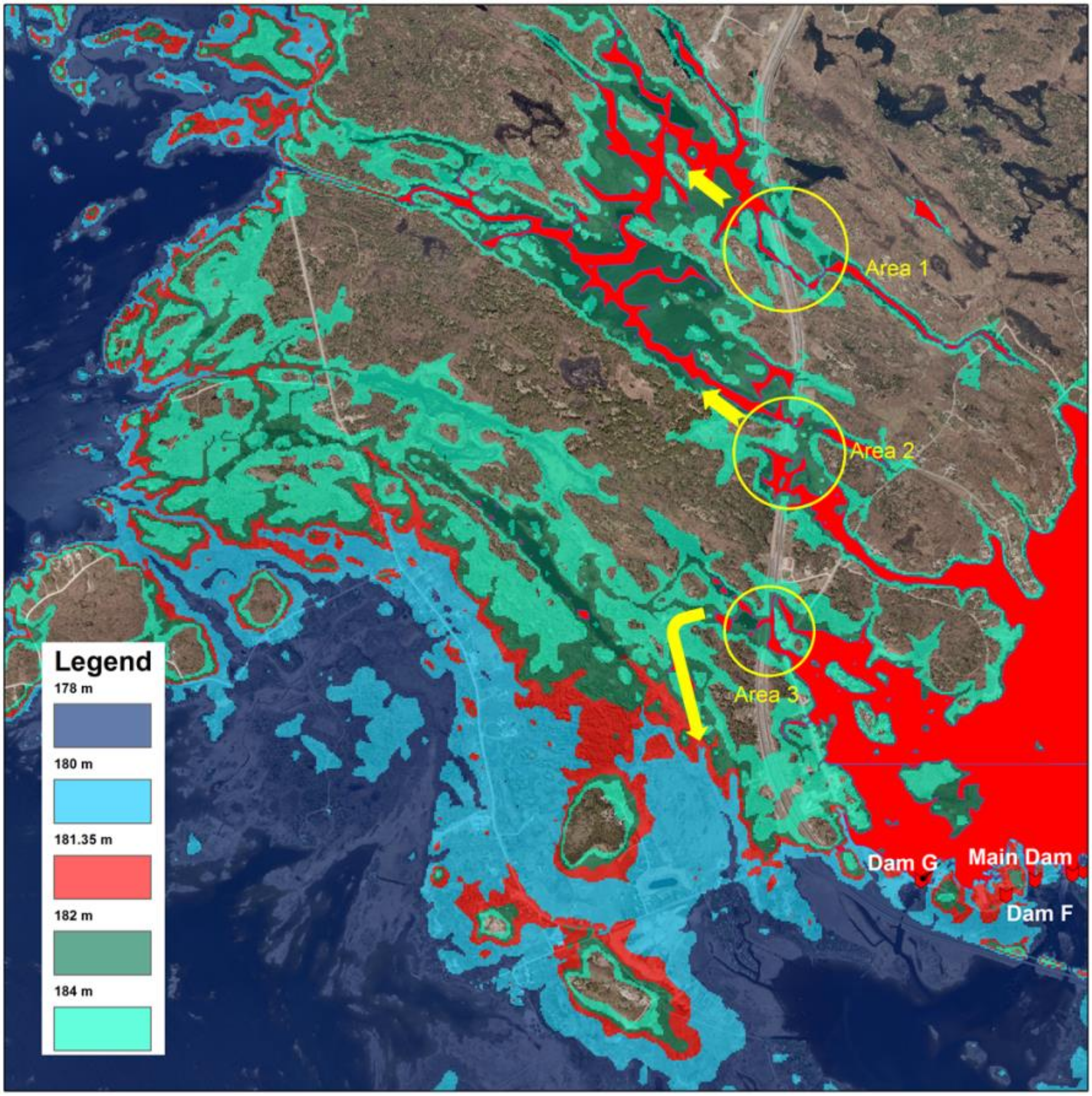

Figure 8.5 Location of Potential Openings at Areas 1 to 3 and Expected Flow Directions

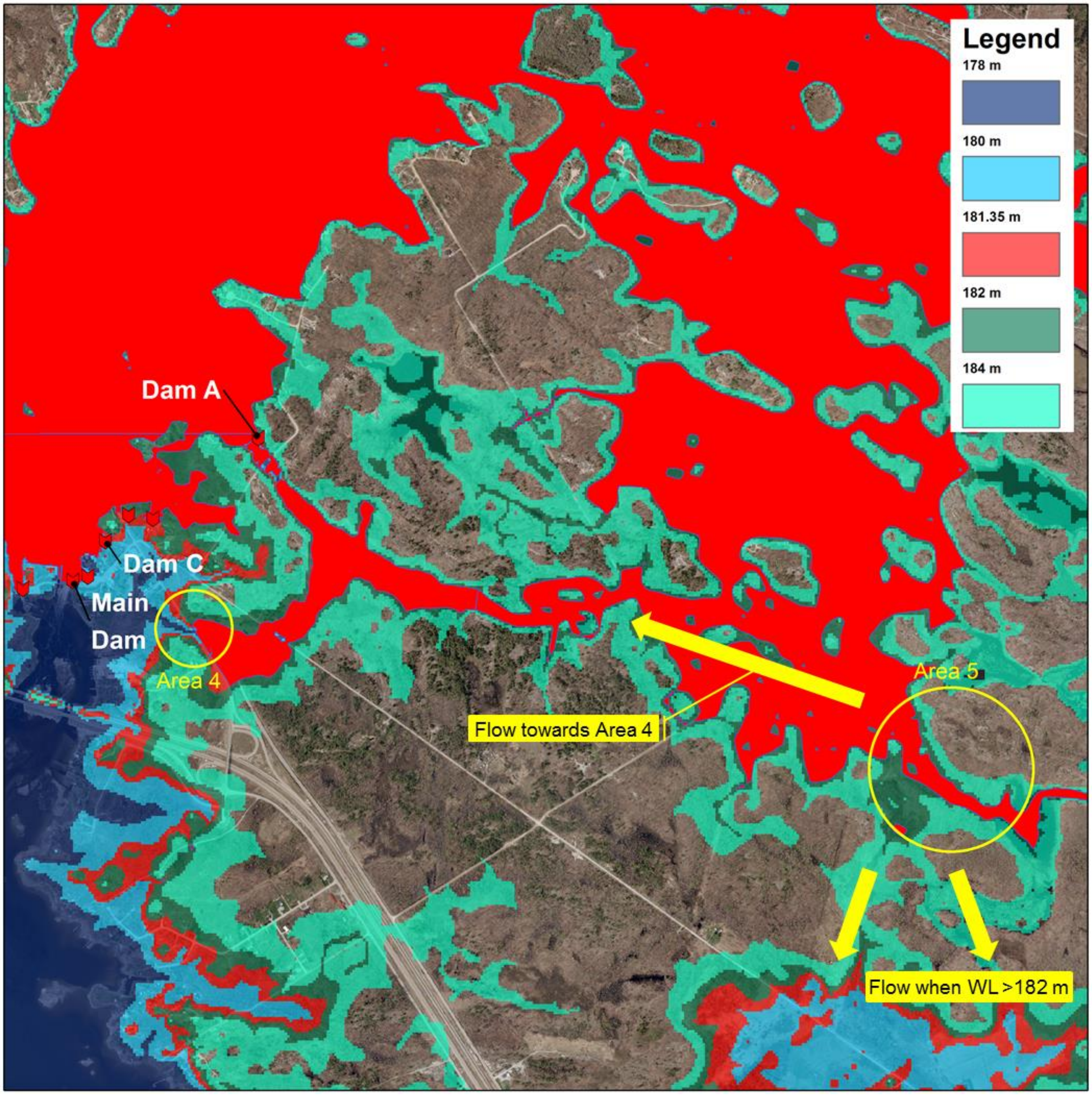

Figure 8.6 Location of Potential Openings at Area 5 and Expected Flow Directions

8.3 Wetland South of Dam A

This section discusses the wetland area south of Dam A and enclosed by Baguley and St Amant Roads.

A significant opening (breach) was observed on the east end of Dam A during AECOM's inspection visit. During the survey, a second opening, located in the centre of Dam A, was observed and is now filled by a beaver dam. These openings seem to have been made intentionally to allow the land south of Dam A to drain toward the reservoir.

A plan view of the area is shown on Figure 8.7. A sketch of the longitudinal profile along the yellow path on Figure 8.7 is shown on Figure 8.8.

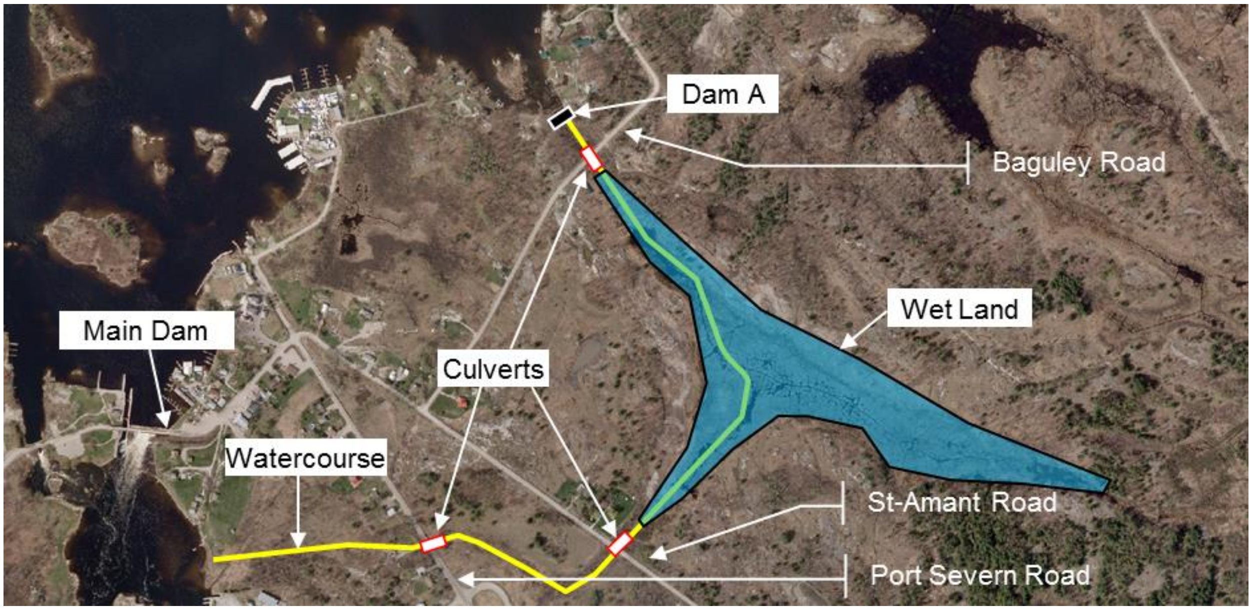

Figure 8.7 Watercourse Downstream of Dam A

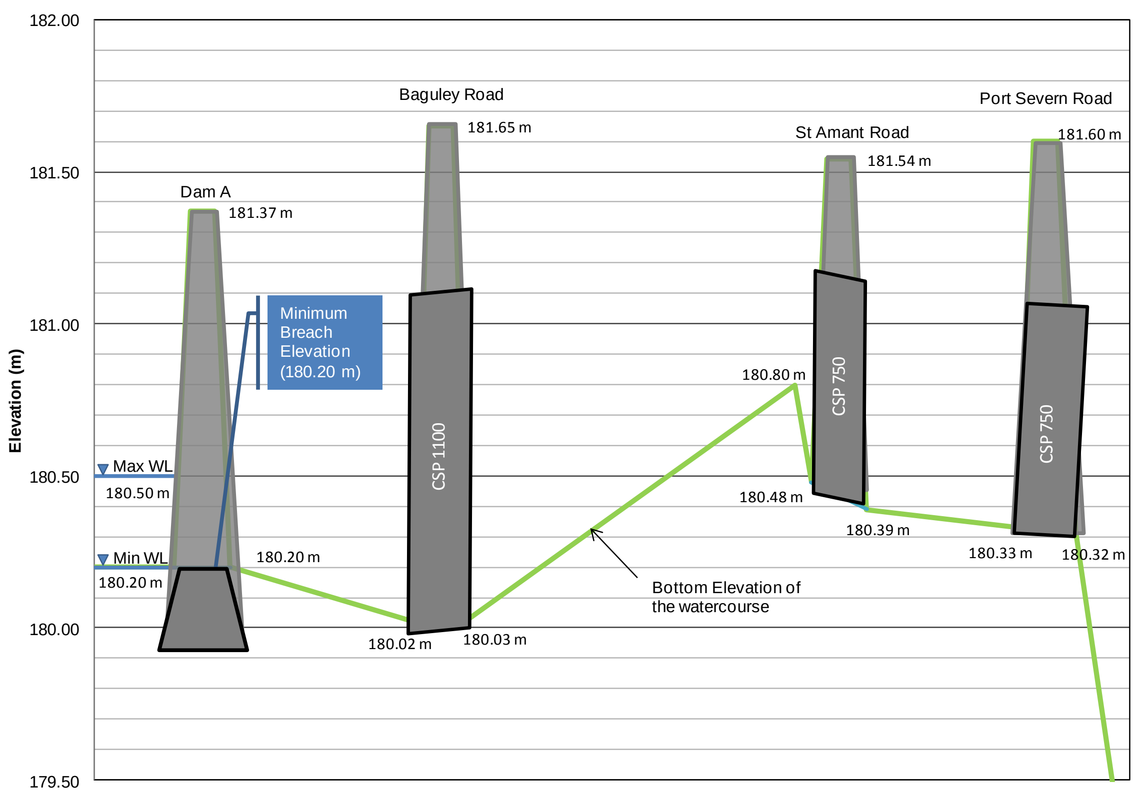

Figure 8.8 Longitudinal Profile of the Watercourse between Dam A and the Bay Downstream of Main Dam

The natural land, north of St-Amant Road, is at elevation 180.80 m and therefore acts as a natural water barrier. This barrier prevents the flow of water through the corrugated steel pipe culvert under St-Amant Road, creating a wetland between Dam A and St-Amant Road. This catchment appears to be captive (no outlet) due to the presence of Dam A, which was built without hydraulic passage, according to the as-built drawing.

Before breaches were made, Dam A had a crest elevation of 181.37 m and water from this catchment was retained south of the dam. It is likely that the only outlet for water to flow out of the system was through the lowest point in the catchment, near St-Amant Road, at elevation 180.80 m. This resulted in the flooding of a large portion of the wetland.

The breaches in Dam A have a minimum elevation of 180.20 m. Therefore, the water level outlet is lowered by 0.6 m, reducing the flooded area.

8.4 Water Impoundment Assessment

This section presents the results of the Water Impoundment Review. The results are categorized by water level.

8.4.1 Maximum Normal Operating Level

The normal operating range is 180.42 m to 180.50 m. At this level there are no major issues with the impoundment of the reservoir. There is some minor flow in areas 1, 2 and 3 through existing culverts, however this is not considered to be a hazard to public and property under normal operating level.

In the wetland area, south of Dam A, water is ponding due to an incorrectly located outlet. The breach in Dam A is not seen as a safety issue and does in fact allow water to drain from the wetland area.

8.4.2 IDF Water Level

At the maximum water level under IDF conditions of 181.35 m, flow can exit the reservoir at four locations – Areas 1, 2, 3 and 4. As discussed previously, culverts are located in these areas. However, they are not designed as outlets from the reservoir.

During the IDF, local roads and Highway 400 may act as water-retaining structures, which could be dangerous to the public. These roads were not designed to retain water and failure may occur, especially if there is a pressure differential (from differing water levels on either side of the road). Furthermore, the existing culverts are not designed as outlets for the reservoir’s waters.

In Areas 1 – 3, there is little danger to the public as flow will be contained in existing water courses. There are a small number of homes or businesses in Macey’s Bay (7.3 km north-east of Port Severn on Honey Harbour Road), but the natural stream will prevent flooding, provided the Highway 400 culvert capacity is sufficient.

In addition, the flow will have low velocity due to the flat terrain and will discharge into Georgian Bay without obstruction. For this reason, there are no works to be recommended for Areas 1 -3.

In Area 4, water can pond upstream of Port Severn Road which poses a threat to downstream houses and road traffic. Section 11.6.1 outlines the recommendations for Dam A.

9. Operation Review

This section reviews the methods and procedures used by the staff to operate the Port Severn Dams.

Observations gathered are presented in Section 4 – Observations, Inspection and Document Review. Testing of the flow control equipment was performed during the site inspection by PCA operators. Based on observations of the operations (Section 4) and the dam hydraulic capacity safety analyses (Section 7.1), the current section analyses the situation and describes whether the operation can meet the objectives. Operation review covers the following:

- Standing Orders.
- Staff capability and training.
- Operating procedures:
    - Dam operation.
    - Lock operation.
- Maintenance.

9.1 Standing Orders Review

The Trent-Severn Waterway Standing Orders were reviewed to make sure that operational procedures, OHS and public safety are respected. Concurrently, in light of the observations done and through the completion of the Dam Safety Review, any amendments to the SO deemed useful by AECOM are presented in the current section.

- Based on observations from the Site Inspection, there is an environmental risk associated with the spilling of fuel during refuelling of the diesel engine for the hydraulic log lifter. The TSW SO 7.7 – Operation of a Hydraulic Log Lifter should be amended to include directives regarding the provision of a containment system.
- Based on discussions with the operators, hooking the stoplogs directly with the hydraulic log lifter's boom is often problematic. The TSW SO 7.7 – Operation of a Hydraulic Log Lifter should be amended in Step 3 to provide further details on how to hook stoplogs manually.
- Operators have mentioned that using the log lifter is not always possible or sufficient to seal the gaps between logs. The TSW SO 7.7 – Operation of a Hydraulic Log Lifter should be amended to include detailed procedure on how to use a portable jack to seal the gaps between logs. This procedure should emphasize safety, as the use of the portable jack entails access to the gains.
- TSW SO 7.2 – Water Control Procedures at Waterway Dams gives the responsibility of adding or removing logs to maintain the water level for navigation purposes to the Lockmaster. However, it does not specify which actions (addition/removal) must be performed at which water levels. It is understood that these are site specific and mentioned in another document or that the Lockmaster's judgment comes into play.
- While the TSW SO address the management of water level for navigation purposes, they make no mention of extreme conditions. Procedures on dam management to pass large floods should be added to the TSW SO 7.2 – Water Control Procedures at Waterway Dams or in a new section. These should cover, but not be limited to: chain of command, responsibilities and accountability of the Lockmaster and others including if the chain of command is broken, order in which the sluices and logs are to be managed, chart of actions to undertake at different water levels.
- TSW SO 2.5 – Precautions against Drowning indicates safety measures to prevent fall and drowning for operators and staff. Given the proximity of the public (few meters) to the open sluices, the standing order should be amended to include the necessity of creating a restricted area during operations. It should include responsibility of the staff, size of the restricted area and means to manage the public presence.

- TSW SO 2.5 – Precautions against Drowning indicates safety measures to prevent fall and drowning for operators and staff. Provisions detailing which safety devices are proper and how and when they should be used should be added.
- TSW SO 2.5 – Precautions against Drowning indicates safety measures to prevent fall and drowning for operators and staff. However, it makes no mention of rescue procedures should someone fall in the water. Instructions on how to proceed in such a case or emphasis on rescue training should be added.
- TSW SO 2.5 – Precautions against Drowning indicates that staff must be trained in the proper use of all safety equipment and devices. It should indicate the frequency at which such trainings are dispensed. It should also indicate that all new hires be properly trained.
- TSW SO 7.4, 7.5 and 7.7 detail the operation of the manual winches and log lifter. However, it makes no mention of training. Most specifically, training on how to operate the equipment under critical conditions should be dispensed regularly as such situations do not happen often and operators could forget how to act in these conditions.
- TSW SO 2.11 – Operations during Severe Weather does not make mention of winter, cold or icy conditions. It should be amended to indicate how the dam must be maintained and operated under wintery conditions. It should also address operator safety under these conditions, particularly pertaining to frostbite, slipping hazard and manipulation of frozen mechanisms.

The SO do not offer any indication on how the operators should proceed when large floods occur or are impending. The SO should, at the minimum, address the following:

- What communication lines exist between the TSW operators for emergency situations?
- What information should be communicated between the TSW operators under emergency situations?
- At which point (expressed as a water level, sluice opening, discharge or other) should a state of emergency be communicated?
- Which sluices and/or dams should be opened first in anticipation of a flood event?
- When should sluices and/or dams be opened in anticipation of a flood event?
- How many sluices and/or dams should be opened in anticipation of a flood event?
- Under which conditions (expressed as a water level, sluice opening, discharge or other) can the operators no longer safely operate the Port Severn Dams?

Note that the SO are independent of the EPP and ERP. The SO remain a tool for the operating staff, detailing how they should proceed to their tasks. The EPP and ERP are more complete plans on how to prepare and respond to the emergency.

9.2 Staff Capabilities and Training

From the interviews and in light of the observations noted in Section 4 – Observations, Inspection and Document Review and the analyses performed in Section 7.1 – Dam Hydraulic Capacity, it was gathered that:

- The operation crew based at Washago (one hour from Port Severn) is responsible for the operation of six dam sites, totalling 13 dams.
    - The operation crew staff has been reduced from six full-time experienced operators to four full-time experienced operators and one seasonal hire.
    - It must be noted that experience is key, especially in the operation of the equipment during significant, rare flood events. Veteran operators should pass down their knowledge and experience to younger operators.

- Periodic training is not being dispensed regularly enough:
  - It has been 5 to 7 years since a formal fall arrest safety training program was attended. Given the experience of the operators, at least one team member has not attended fall arrest safety training.
  - Training on the use of the equipment under hazardous conditions has not been dispensed recently.
  - Training on rescue operations procedures has not been dispensed recently.
  - The staff is not trained to operate the Port Severn Dams during critical floods, such as the IDF. Examples of such situations include: removal of bottom logs under high head and water velocity, emergency actions such as the breaking of logs that cannot be removed, use of the equipment under stormy weather, and more.

9.3 Dam Operation Procedures Review

Review of the dam operational procedures is required to assess the capability and reliability of the operations under usual, unusual and extreme situations such as under the IDF. Flow control is performed by adding and removing stoplogs in the sluices with the log lifter or manual winches. The Main Dam is generally managed to maintain water levels for navigation purposes, however, the Port Severn Dams must also be safely managed to protect lives, the environment and property.

9.3.1 Operation Review for Water Levels

The main function of the Main Dam is to manage water levels for navigation purposes. The operation review for water levels analyses the procedures performed and the capability to operate equipment to manage water levels for navigation purposes. In light of the observations noted and the analyses performed, it is understood that:

- The staff is sufficiently experienced and knowledgeable to operate the facilities.
- There are sufficient resources to operate the dam for water level management.
- Operation is done following orders from the Sector Manager, who is instructed by the Water Management Engineer on the increase or decrease of discharges.
- Mostly, sluices 2 and 3 are managed. It is preferable to operate all sluices regularly rather than rely on the same ones. Use of sluices allows the operators to detect any emerging issues.
- The log lifter can be efficiently used to add/remove sluices to manage the water levels.
- The backup manual winches can be used efficiently and under acceptable delay to add/remove sluices to manage the water levels should the log lifter not be usable.
- Seasonal operation is well-managed. There are no issues pending.

9.3.2 Operation Review for Dam Safety

Under flood conditions, urgent intervention is required to open enough sluices to discharge water and keep the dam safe. Section 7.1.3 – Adequacy of Discharge Capacity details the Port Severn Dams' discharge capacity under different conditions. While the dams may have the capacity to discharge the required flows, it must be shown that the operators can safely operate the dams to achieve the discharge objectives in due time. Operation Review for Dam Safety reviews the procedures performed and the capability to operate equipment to discharge flows with the aim of maintaining safe conditions at the dam. The operation review encompasses all the Port Severn Dams (Main Dam, Dam E and Dam G) since they work together to control the water levels in the Gloucester Pool and Little Lake.

The log lifter at the Main Dam is the primary equipment used to add and remove logs to control water levels and discharge. The manual winches are the backup log lifting system. Both must be maintained in working order. In light of the observations noted in Section 4 – Observations, Inspection and Document Review and the analyses performed in Section 7.1 – Dam Hydraulic Capacity, the following conclusions are drawn:

- Staff capabilities
  - The staff is not sufficiently experienced and knowledgeable to operate the facilities under IDF conditons: most have not experienced significant floods and no training has been provided on operation under emergency situations.
  - There is currently the bare miminum number of operators to operate the dam for flood discharge. However, there are no experienced back-up crews.
  - Operation is done following orders from the Sector Manager, who is instructed by the Water Management Engineer on the increase or decrease of discharges.
- Main operating equipment:
  - The log lifter can easily be operated to control water levels under normal conditions.
  - The log lifter is in an acceptable working condition and must remain so at all times.
  - The lower logs (logs below the eighth) currently cannot be relied upon to discharge floods. While operators believe the logs can be manipulated, during the flow tests they were unable to remove them.
  - Under critical conditions, such as when facing an impending IDF, the log lifter can be operated to discharge the required flows.
- The backup system (manual winches):
  - The manual winches are in good working condition and must remain so at all times.
  - The manual winches can be operated to control water levels under normal conditions.
  - Use of the manual winches is physically demanding and hazardous, especially under unfavorable weather conditions.
  - Should the log lifter not be usable at all, the backup manual winches cannot be safely and efficiently operated to discharge the IDF under the prescribed delays. Manual winches are physically difficult to operate. It also takes much time to remove a log.
  - The log lifter cannot reach sluices 1 and 9 because of fences at either end. The manual winches must be lifted from the dam deck rails using a pole as a lever to be brought on the other side of the log lifter to reach sluices 1 and 9
- Operational risks:
  - Difficulty to operate submerged logs can delay sluice opening.
  - The poor conditions of many stoplogs, especially of the dees, can delay sluice opening.
  - Should a crucial piece of equipment break, there is no spare part available on site.

9.3.2.1 Operation for Total Discharge Capacity

Section 7.1 – Dam Hydraulic Capacity evaluates the capacity to evacuate the IDF under different operation scenarios. The discharge of the IDF of 502 m^3/s is possible without raising the water level to unacceptable levels (180.57 m, 0.07 m above MNOL) if all sluices are entirely opened. However, following the investigations (Section 4 – Observations, Inspection and Document Review) it is understood that it may be difficult to fully open all the sluices under their current state.

To evaluate the capacity of the facilities to discharge the floods under problematic circumstances and indicate how they can be operated to achieve this objective, different scenarios are assumed. The five scenarios address the hypothetical inoperability of the sluices. For each operability scenario, the operation aims at identifying the flow discharged under MNOL conditions (180.50 m) and checking the capacity to pass the IDF of 502 m^3/s. For the passage of the IDF, the maximum water level under IDF conditions of 181.35 m is selected. Inability to discharge the IDF when the water level is at the maximum acceptable threshold would suggest an inadequate discharge capacity.

- Scenario 1: no issues, all equipment functions well.
- Scenario 2: bottom logs are stuck, broken or otherwise impossible to remove in all sluices.
- Scenario 3: bottom logs are stuck, sluices 1 and 9 are inoperable, either because the manual winches can't be used, flooding downstream of sluice 9 is unacceptable, all logs in stuck in these sluices or for any other reason.
- Scenario 4: bottom logs are stuck, sluices 1, 7 to 9 are inoperable for the reasons stated above.
- Scenario 5: bottom logs are stuck, sluices 1, 7 to 9, Dam E and Dam G are inoperable for the reasons stated above and/or access to Dam E and Dam G is impossible.

The following lists the results that can be drawn from the analysis of Table 7.1 and Table 7.2. Table 9.1 summarizes the results. Stoplogs settings shown in Table 9.1 are the ones considered optimal under the issues faced. They minimize recourse to the manual winches and transportation of resources to Dam E and Dam G.

- Scenario 1: If the Port Severn Dams are maintained in good working condition, then the IDF can be safely discharged by:
  - Fully opening sluices 2 to 6.
- Scenario 2: If no more than eight logs can be removed from any sluice, whether it is because they are damaged, because there is too much water flowing over or any other reason, then the IDF can be safely discharged by:
  - Removing the top eight logs from sluices 1 to 7 and at least 4 from sluice 8 or;
  - Removing the top eight logs from sluices 2 to 8 and at least 6 from sluice 1 (preferred since it reduces the use of the manual winches).
- Scenario 3: If no more than eight logs can be removed from any sluice and sluices 1 and 9 are inoperable, either because the manual winches can't be used, flooding downstream of sluice 9 is unacceptable, all logs are stuck in these sluices or for any other reason, then the IDF can be safely discharged by:
  - Removing the top eight logs from sluices 2 to 8 and at least 7 from Dam E.
- Scenario 4: If no more than eight logs can be removed from any sluice and sluices 1, 7, 8 and 9 are inoperable, then a maximum of 449 m^3/s can be discharged by:
  - Removing the top eight logs from sluices 2 to 6 and Dam E and fully opening Dam G.
- Scenario 5: If no more than eight logs can be removed from sluices 2 to 6 and all other sluices and dams are inoperable, then a maximum of 384 m^3/s can be discharged by:
  - Removing the top eight logs from sluices 2 to 6.

Table 9.1 Minimum Sluice Operability Scenarios at Port Severn Dams

Scenario	Issues	Open Sluices	Level (m)	Flow (m^3/s)
1	• None	Sluices 2 to 6 ► fully opened	181.35	502
			180.50	314
2	• Bottom logs remain in place*	Sluices 2 to 8 ► 8 logs removed Sluice 1 ► 6 logs removed	181.35	502
			180.50	291
3	• Bottom logs remain in place* • Sluices 1 and 9 inoperable	Sluices 2 to 8 ► 8 logs removed Dam E ► 7 logs removed	181.35	502
			180.50	295
4	• Bottom logs remain in place * • Sluices 1, 8 and 9 inoperable	Sluices 2 to 7 ► 8 logs removed Dam E ► 8 logs removed Dam G ► 7 logs removed	181.35	449
			180.50	262
5	• Bottom logs remain in place * • Sluices 1, 7, 8 and 9 and dams E and G inoperable	Sluices 2 to 6 ► 8 logs removed	181.35	384
			180.50	205

* Bottom logs refer to all logs below the eighth log, when applicable.

9.3.2.2 Operation for the Maximum Daily Incremental Discharge

The maximum daily incremental discharge is the maximum increase in flow which can be expected to occur over for a day for a given flood event. It is studied to assess whether the facility can be realistically operated to add sufficient capacity to the dams' total discharge. While a dam may have sufficient discharge capacity available, operators must be able to open sluices at a rate faster than the increasing flood flow.

As previously mentioned in Section 5.5 - Maximum Daily Incremental Discharge, the maximum daily incremental increase in flow during the IDF is 224 m^3/s. As this incremental increase occurs in the latter part of the flood hydrograph, sluices passing 278 m^3/s are already opened, for a total of 502 m^3/s.

The time required to operate the facilities is evaluated for the different scenarios of sluice operability. The scenarios consider the following potential issues:

- Stoplog removal / sluice operability:
  - Base case scenario considers that all stoplogs in all sluices can be removed
  - Pessimistic scenario considers the current status of the facilities, such that the bottom stoplogs (stoplogs below eighth stoplog) are not removed from the sluice
- Preliminary operation of sluices:
  - Base case scenario considers that the flood was well forecasted, and the sluices most difficult to operate were opened first.
  - Pessimistic scenario considers that the flood is managed in a reactive manner and that the first sluices opened were the easiest to operate, leaving the most difficult ones to be opened during the passage of the maximum daily incremental discharge.
- Defective equipment:
  - Base case scenario considers that the log lifter is in good working condition.
  - Critical scenario assumes the log lifter is out of service requiring the use of the manual winches at all sluices.

- Stoplog breaks:
  - Four different scenarios of stoplog breaking are considered (percentage of stoplogs that break).

There exist other issues that may delay operations but are not considered, such as the use of tongs, difficulty hooking onto damaged dees and the increasing difficulty to hook the logs as the water level rises. These are included as contingencies. The time required to perform each task is as noted during the site investigation and presented in Section 4.4 – Current Operation Procedures.

While the maximum daily incremental discharge is calculated over 24 hours, it is unrealistic to assume that workers would work an entire day. Considering the staff capabilities and their other responsibilities (See Section 4.4.1.1 – Staff Capabilities), a maximum of eight hours of labor dedicated to the operation of the Port Severn Dams is deemed realistic. Under IDF conditions, three teams of two operators can work up to 16 hours per day, for a maximum incremental flow release of 224 m^3/s per day. As such, the removal of logs at Port Severn to allow the additional discharge of 224 m^3/s must be done in less than eight hours.

The first analysis aims at answering the question: Under the current conditions, can the Port Severn Dams be operated quickly enough to safely discharge the IDF? It looks at the time required to remove enough logs so that the water level remains below 181.35 m, which is the maximum allowable level under which the dam is stable. Table 9.2 presents the results, from which it can be seen that:

- The back-up manual winches do not allow removal of stoplogs fast enough, under any scenario.
- If more than 25% of stoplogs break during removal, it will not be possible to open the sluices fast enough, under any scenario.
- If the log lifter can be operated and 25 % or less of the stoplogs break during removal, it should be possible to open the sluices in time, regardless of the sluices already open.
- If the dam is fully rehabilitated and all logs can be removed, it is possible to open the sluices in time, regardless of the sluices already open.
- If the dam sluices remains in the current state and bottom logs cannot be removed:
  - If the flood is forecasted and operators open the most difficult sluices first, removal of additional stoplogs to meet time requirements can be achieved in time.
  - If the flood is not forecasted and operators open the most easily accessible sluices first, removal of sufficient stoplogs can still be done in time, but the margin of error is reduced.

The second analysis aims at answering the question: Given a full rehabilitation such that all logs can be removed, can the Port Severn Dams be operated quickly enough to safely discharge the IDF while maintaining water levels near MNOL? It looks at the time required to remove enough logs so that the water level remains below 180.57 m during the passage of the IDF. This requires that the dam is fully rehabilitated and all logs can be removed. Table 9.3 presents the results, from which it can be seen that:

- The back-up manual winches do not allow removal of stoplogs fast enough, under any scenario.
- In the Port Severn Dams’ current state, considering the difficulty to remove the lower stoplogs, it is impossible to discharge the IDF without raising the water level above 180.57 m.
- It is possible to discharge the maximum daily incremental discharge under acceptable delay only if efficient flood forecasting is performed and most sluices are open in anticipation.

Table 9.2 Time Required to Operate the Dams for the Required Minimum Sluices' Opening to Add the Maximum Daily Incremental Discharge (224 m³/s) to the Flood without Exceeding the Maximum Water Level of 181.35 m

Hypothesis 1: Issues with Stoplog Removal	Hypothesis 2: Management of Sluices before Max. Daily Incremental Discharge	Operation during Max. Daily Incremental Discharge	Time Required to Operate (h) Log Breaks 0% Manual Winches	0% Log Lifter	10% Log Lifter	25% Log Lifter	50% Log Lifter
No issues	<u>Base scenario</u> Sl. 1 → 10 logs removed Dam E → 9 logs removed Dam G → 7 logs removed	Sl. 2, 3 → remove 12 logs Sl. 4 → remove 10 logs	*18*	3	5	7	*10*
	<u>Pessimistic scenario</u> Sl. 2, 3 → 12 logs removed	Sl. 4-6 → remove 10 logs	*16*	3	5	7	*10*
Bottom 8 logs remain in place	<u>Base scenario</u> Sl. 1 → 8 logs removed Dam E → 8 logs removed Dam G → 7 logs removed Sl. 8 → 8 logs removed	Sl. 2-5 → 8 logs removed Sl. 6 → 6 logs removed	*18*	3	5	7	*13*
	<u>Pessimistic scenario</u> Sl. 2-4 → 8 logs removed	Sl. 5-8 → remove 8 logs Sl. 1 → remove 6 logs	*31*	6	7	8	*13*

* Values of time required in green highlight acceptable delays (< 8 hours) and those in *red (italic)* highlight unacceptable delays.

Table 9.3 Time Required to Fully Open the Dams in Order to Add the Maximum Daily Incremental Discharge (224 m³/s) to the Discharge Capacity while Maintaining the MNOL (180.50 m)

Hypothesis 1: Issues with Stoplog Removal	Hypothesis 2: Management of Sluices before Max. Daily Incremental Discharge	Operation during Max. Daily Incremental Discharge	Time Required to Operate (h)				
			Log Breaks				
			0%	0%	10%	25%	50%
			Manual Winches	Log Lifter			
No issues	Base scenario Sl. 1, 5-9 → 10 logs removed Sl. 5 → 8 logs removed Dam E → 9 logs removed Dam G → 7 logs removed	Sl. 2, 3 → remove 12 logs Sl. 4 → remove 10 logs	*19*	3	5	8	*12*
	Pessimistic scenario Sl. 2, 3 → 12 logs removed Sl. 4-5 → 10 logs removed Sl. 6 → 4 logs removed	All sluices fully opened	*32*	*23*	*26*	*31*	*39*

* Values of time required in green highlight acceptable delays (< 8 hours) and those in *red (italic)* highlight unacceptable delays.

9.4 Lock Operation Review

The review of the lock operational procedures is required to understand the conditions under which the lock, portion of the lock or the lock gates, is acting as a dam/water-retaining structure.

The lock gates act as a water-retaining structure when water on one side of a gate is higher than on the other. At all times, at least one of the gates acts as a water-retaining structure. The most significant case is when the downstream gate is subject to a high hydrostatic pressure from the basin side, while the water level on the downstream side is low.

Stability is demonstrated for well-maintained lock gates. Logs in gates must be maintained in good conditions.

9.5 Maintenance

The SO 8.5 – Custodial Care of Control Dams specifies the minimum standards of custodial care. For the most part, operators at Port Severn are said to follow the SO. However, the poor condition of some logs suggests that maintenance is performed mostly on the equipment that is frequently used. It is important that maintenance also be performed on equipment (logs, sluices) that is kept for emergency situations. Failure to do so can create a hazard during a large flood. In summary:

- Maintenance performed as per SO 8.5 is satisfactory.
- The bottom logs and logs in less operated sluices are not adequately maintained.
- The rate of replacement of the logs is not sufficient.
- It is believed that dams are properly maintained during winter, though site visits did not take place in winter and this was thus not verified.

10. Safety Review

10.1 Occupational Health and Safety (OHS)

In light of the observations made and studies performed, the Occupational Health and Safety status of the operators working on the Main Dam, Lock 45, Upstream Wall and Dam D sites is reviewed in this section. To assess the situation, the audit focused on tasks performed by PCA staff while operating flow control equipment at Main Dam, Dam E, and Dam G (see Section 4.6.1 – Occupational Health Safety (OHS)).

The following operator safety issues were reviewed and evaluated with respect to the Canada Labour Code (Reference 30) and the TSW Operational Standing Orders (SO), version revised in April 2010 (Reference 50).

- Access to the site and the dam deck for operations (including access during adverse weather conditions).
- Safety features of the dam (i.e. guardrails, gain cover).
- Operators' safety measures.
- Falling hazards.
- Dam and lock operations procedures (i.e. stoplog operation, lock gate, stoplog jacking, ice and debris removal, etc.).
- Stoplog winches and log lifter.
- Communication.
- Training of operation staff on dam equipment operating procedure and in emergency response.

Table 10.1 lists all the issues reviewed that warrant an intervention or at least, further revisions. Falling hazards and night time operation present OHS risks and do not comply with safety standards. These must be remedied as soon as possible and Standing Orders revised.

In summary, it was seen that there is currently no secure way for the operators to attach themselves during operations on the Main Dam. In addition, lighting is absent making night operation difficult. On Lock 45, manoeuvring around the locks is hazardous as the railings do not meet the standard. Also, a comprehensive training program should be given to operators. This is required to ensure that operation can occur in fair and adverse weather conditions.

Table 10.1 Occupational Health and Safety Issues Review

OHS Issue	Situation	Requirement	Current Actions Performed	Review
Slip / fall hazard at Lock 45 steps	Leakage causes a slip / fall hazard	No specific requirement	Periodic maintenance (cleaning of the steps) is performed	Unclear. It is unclear whether maintenance is performed sufficiently to guarantee safety of workers and public
Fall hazard during operations	Operators do not attach themselves securely nor do they always wear lifejackets when working over gains (for ice chipping, log manipulation, jacking and more). There are no engineered anchors or tie off locations on the dam or on the log lifter	SO 7.4: each employee must wear appropriate PPE and be tied off SO 2.3: life jackets must be worn whenever conditions exist where there is a risk of drowning, including whenever the gains are opened	Operators tie off to the log lifter or fence rail. Lifejackets are worn sporadically.	Hazardous. There is currently no safe and secure way for the operators to attach themselves during operations.
Fall hazard during winter conditions	Could not be observed	SO 7.1: snow and ice shall be removed from working areas and sand or salt shall be spread	Based on interviews with the operators, it is believed that the facilities are cleared and well maintained.	Satisfactory.
Railings to protect from falling hazard	Railings at Lock 45 do not meet the safety requirements of NBCC 2005 and OBC 2006 This pathway is the access to the dam operation area			Hazardous. Manoeuvring the locks without proper railings is hazardous.
Use of the manual winches	Manual log lifting is physically demanding and hazardous, especially in adverse weather conditions.	SO 7.4 – Removal of Stoplogs in a Dam using Winches	Used only for sluice 1 and as a backup system for the other sluices	Unclear. The manual winches pose risks of injuries. Yet, they are mostly used as a backup system. Alternative backup systems should be investigated.
Use of the log lifter	Use of mechanized equipment can lead to injury		Log lifter only operated by qualified personnel. Safety training has not been dispensed recently.	Satisfactory. Safety training should be dispensed periodically.

OHS Issue	Situation	Requirement	Current Actions Performed	Review
Night time operation	No night lighting present on the Main Dam Inadequate lighting on the hydraulic log lifter.	No specific requirement		Hazardous. Operation area must be well lit.
Training	Operators were last trained 5 to 7 years ago in OHS.	SO 2.5: staff must be trained in proper use of safety equipment but no regular period is mentionedSO 3.1: cites that staff must be trained in fire prevention		Inadequate. Training should be offered regularly on the following topics: - rescue training - fire prevention - use of PPE - falling hazards and working in heights - dam operation, especially under flood conditions SO should reflect this and indicate the frequency of training sessions.
Life rings	At least one life ring at the dam; at least two at the lock	SO 2.5: must have at least 1 ring per dam SO 2.5: must have at least 2 rings per lock		Satisfactory.
Pike poles	Available	SO 2.5: pike poles at lock and dam		Satisfactory.
Personal protective equipment (PPE) and life jackets (PFD)	Available, but not always worn	SO 2.5: all safety equipment and devices are in place on site during working hours SO 2.3: life jackets must be worn whenever conditions exist where there is a risk of drowning SO 7.4: the equipment that must be available for operation is listed		Unclear. Operators must wear PPE and PFD.
Permanent ladders	Available	SO 3.1: two recessed fixed ladders on each side		Satisfactory.
Fastening lifting equipment	Operation of the equipment was satisfactory	SO 7.1: winches and log lifters must be fastened before use		Satisfactory.
Securing logs	Operation of the equipment was satisfactory	SO 7.1: logs removed must be placed on rails or rolling strips which are secured to the deck of the dam for this purpose.		Satisfactory. Does not meet SO Guidelines for staff, however a safe working environment is maintained.

OHS Issue	Situation	Requirement	Current Actions Performed	Review
Gain covers	Operators walk on the gain covers			Satisfactory. Gain covers are in good condition and safe to walk on
Log handling	Handling of logs can cause injury, particularly to the back or feet		Safety boots must be worn Peaveys must be used to move logs around	Satisfactory Workers wear the safety boots and are proficient in the use of the peaveys

10.2 Public Safety

The Dam Safety Review includes the execution of a Public Safety Risk Assessment. The Public Safety Risk Assessment at Port Severn is performed in accordance with the Canadian Dam Association's (CDA) Guidelines for Public Safety around Dams, issued in 2011 (Reference 14). The Risk Assessment is the second process of the Risk Management Process, as defined by the CDA. Before starting the Risk Assessment, the dam owner should establish the context, including policies, accountabilities, risk criteria, standards and procedures for evaluating the risks. The Risk Assessment includes three main steps (Reference 14):

- Identification of risks
  - What can happen?
  - When and where?
  - How and why?
- Analysis of risks
  - Identify existing controls (physical and procedural).
  - Determine and assign values of consequences.
  - Determine associated likelihoods.
  - Determine levels of risks.
- Evaluation of risks
  - Compare the results of risk analysis with risk criteria to determine whether risk can be tolerated.

Should the Risk Assessment process conclude that the risk is too great and cannot be tolerated as is, then the dam owner should consider the execution of Risk Treatment. This third and final process in Risk Management, aims at identifying and assessing options, preparing and implementing treatment plan and evaluating residual risks.

Consequently, the current DSR includes the Risk Assessment process. Complete Risk Assessment can be found in the Public Safety Around Dams report (Reference 3). The current section summarizes the Risk Assessment.

The Port Severn Main Dams are part of the Trent-Severn Waterway National Historical Site and as such, site accessibility and public activities and interactions are high. The site is located within permanent residential communities as well as resorts and touristic attractions. Public recreational activities include water sports such as boating, canoeing, kayaking, swimming and fishing. In addition, several marinas exist at the site, with some boathouses moored during the navigation season. The dams also serve as access roadways.

The sites present significant hazards, which could result in risk of serious injuries and fatalities when combined with the high degree of public activities and interaction.

The most critical public activities and related hazards for Main Dam, Dam D and Lock 45 are mainly:

- **Boating**: The exposure of the public to dangerous hydraulic conditions such as headpond currents where boats can be dragged toward the structure. Access to the Lock 45 can be hazardous depending on flow conditions due to recirculation currents created by the flow going through Main Dam.
- **Canoe/kayak**: The lack of signage for portage routes and the deficient guard wire could result in canoes/kayaks ending up in hazardous areas and being exposed to the highest rated (5) risk.
- **Swimming/Diving**: Due to lack of signage and fencing, dangerous areas such as the headpond and spillway are accessible to swimmers without adequate warning or restriction.

- **Fishing from shore/structure, walking, biking**: The current dam and road configuration means that pedestrians are using dam operation decks to cross the Severn River and adjacent streams. Access to the operation deck of the dam exposes the public to hazards such as mobile heavy mechanical equipment, unsecured gain covers and inadequate fencing/railings.
- **Ice fishing/skating/snowmobile**: Within the study area's boundaries, there are no activities taking place from or on the ice. The flow going through the dams normally doesn't allow for thick ice formation.

11. Recommendations

Following the review of the dam safety of the Main Dam, Lock 45, Upstream Shoreline Wall and Dam D of the Port Severn hydraulic works, recommendations are put forth to improve the safety of the public, operators and the environment.

The following sections detail each of the recommendations and how they apply to a particular work or activity. The time of implementation (prioritization) and costs for each recommended action are estimated in Table 11.1 of Section 11.7.

11.1 Additional Studies and Information Gaps

It is recommended that PCA proceed with the following studies:

- Given that the Main Dam has a hazard classification of Significant, then DSR studies should be performed every 10 years (see Table 6.16).
- Given that a Sunny Day failure would have major impact/consequences, it is highly recommended that the following studies be performed: EPP, ERP and OMS. Each of these documents should cover all the Port Severn Dams. These include a Threat Assessment.

Also, in order to ensure the longevity and continued operation of the Port Severn Main Dam, PWGSC/PCA may consider performing a Recapitalization Study. The purpose of this study is to quantify the cost and benefit of refurbishing the Port Severn Main Dam to a condition where it will provide another 40+ years of useful life. The completion of a Recapitalization Study is outside of the required works to bring the structures to a standard recommended by the CDA. However, it is part of the responsible management of the Port Severn Main Dam and PWGSC/PCA could benefit by including this study in a package with the recommended works and remedial measures.

11.2 Rehabilitation / Repair of Assets

11.2.1 Main Dam

The following works are required to improve the stability and structural reliability of the Main Dam spillway and avoid further degradation:

- All cracks and opened construction joints need to be repaired.
  - Opened joints and cracks of small width can be injected with a cement grout (pressure grouting) or polyurethane.
  - Opened joints and cracks of larger width require that the concrete surface be removed up to the healthy concrete before new concrete is put in place to fill the gap.
- Deteriorated concrete surfaces must be repaired in the upstream and downstream drawdown zones, under the upstream deck where the rebar is visible and around the stoplog gains.
  - To repair this sort of degradation, concrete surfaces must be removed up to the healthy concrete and a new concrete should be put in place to protect the structure.
  - Dowels should be placed between the new and old concrete surfaces to ensure the connection.
- Post-tensioning anchors must be placed in every pier of the Main Dam to stabilize the dam at the concrete-rock interface. As long as the anchors are not installed, staff should proceed to breaking the ice to prevent formation of the ice load. The installation of water-agitators is not recommended as it would be quite expensive for a temporary solution.

The following works are required to restore the capacity and improve the reliability of operations at the Main Dam:

- Sluice 9 should be sealed off.
  - Logs should not be operated anymore. Low hydrostatic pressure and no operation reduce the risk of failure.
  - Concreting is not necessary, but is preferable. It could be done if costs are not prohibitive.
- Active sluices (all except 9) should be overhauled to restore them to a good functioning condition.
- Rails on the dam deck must be replaced.
- Options should be looked at to allow the log lifter to reach sluice 1. Removal/displacement of fence should be considered.

The following are recommended to ensure that all required equipment be available and functional in order to meet the operational objectives:

- The design of the stoplogs needs to be revised and updated to recent standards of the wooden beam design industry (i.e CSA-O86-09). PCA should investigate the cost and effectiveness of alternative solutions, such as steel plated logs, logs made of other materials, etc.
- Equipment to prevent spills while refueling and allow cleanup of spills (absorbent material) should be available on site.

Considering the log lifter, two alternatives are possible: replacement or rehabilitation. For dam safety considerations, it is not necessary to replace the log lifter with a new model. While the current unit has reached the end of its expected life expectancy, it is still in good condition and minor repairs are sufficient. However, the use of the log lifter poses an OHS risk, as the operators cannot rely upon it to tighten the stoplogs and must enter the gains to perform hydraulic jacking. A new unit would notably provide safer working conditions since it would have the capacity to compress the logs together (eliminating the current log jacking practice) and provide the operator with a cabin to shelter the operator from unfavourable climatic conditions.

It is thus recommended to:

- Replace the log lifter with a new model.

However, should PCA wish to keep costs as low as reasonably possible, it could choose to rehabilitate the log lifter. This includes replacing the log lifter sideway adjusting rails, rust proofing, replacing the capacity and data plate and operator compartment safety gate and clearly marking the operator control.

The manual winches, used as backup for sluices 2 to 8 and primary operating at sluices 1 and 9, are functional but their use is physically demanding and difficult. It is recommended to:

- Investigate the replacement of manual winches with modern equipment. Beebe type winches may be used; however operators prefer the current robust winches.

11.2.2 Lock 45 and Upstream Shoreline Wall

The following works are required to improve the stability and structural reliability of the left wall of Lock 45 between the stairs and the Main Dam (where leaking of water through the wall is observed), and to avoid further degradation:

- All significant cracks need to be injected with a cement grout (pressure grouting) or polyurethane.
- Dowels need to be placed to ensure the connection between the two concrete pours (pressure grouting) or polyurethane.

The following works are part of responsible maintenance that should be performed at Lock 45 to keep it safe and avoid further degradation.

- All cracks should also be injected with a cement grout to prevent further degradation at the same time as the previously stated, more important repairs.
- Until the left wall is repaired and leakage is stopped, public access to the stairs should be limited and signs warning of slippery surface should be put up.

The presence of sinkholes at the Upstream Shoreline Wall suggests that water seeps through the degraded wall, causing transportation of soil particles and erosion. The following work is required to avoid further degradation of the Upstream Shoreline Wall.

- Deteriorated concrete surfaces in the drawdown zone must be repaired.
  - To repair this type of degradation, the concrete surfaces must be removed up to the healthy concrete and a new concrete should be put in place to protect the structure.
  - Dowels should be placed between the new and old concrete surfaces to ensure the connection.
- Eroded soil material of the sinkholes should be replaced by a similar material placed with appropriate compaction.

Mechanization of the Lock 45 gates has not been deemed worthwhile for dam safety purposes. The current gate operation mechanism is fully functional and does not pose any outstanding risk. However, it is one of the works that would be considered in a Recapitalization Study.

11.2.3 Dam D

The presence of sinkholes suggests that water seeps through the degraded wall, causing transportation of soil particles and erosion. The following work is required to avoid further degradation at Dam D.

- Deteriorated concrete surfaces in the drawdown zone must be repaired.
  - To repair this type of degradation, the concrete surfaces must be removed up to the healthy concrete and a new concrete should be put in place to protect the structure.
  - Dowels should be placed between the new and old concrete surfaces to ensure the connection.
- Eroded soil material of the sinkholes should be replaced by a similar material placed with appropriate compaction.

11.3 Operational Procedures

11.3.1 Staff Capabilities

Two teams of two experienced and trained operators are dedicated to the six dam sites, totalling 13 dams, between Washago and Port Severn. During significant floods, a third team could be made up of the seasonal employee and the Sector Manager. Such a team has the ability to operate safely the IDF, considering ideal condition of a functional log lifter and a maximum of 10 % of stoplogs break. However, considering that the IDF would also have to be managed at all dams in the sector, the staffing requirements at Port Severn must be consistent with required operations at other dams in the sector. This minimum staffing requirement as well as stoplogs and lifting devices condition requirement will allow minimum opening of sluices to ensure that water level will be kept below 181.35 m, not to overtop the water-retaining structures of Port Severn.

Thus, as there are no backup crew members or contingencies, it is recommended that staff numbers be increase to at least six full-time operators. This would free up the Sector Manager so that he/she could concentrate on managerial duties. However, there would still not be any redundancy to cover for absences or other emergencies.

11.3.2 Training

Periodic mandatory training on the operation of the equipment under critical conditions, such as the IDF, should be provided (see Section 11.5.1 – Occupational Health and Safety for recommendations on safety training). Such training should address, at the very least:

- Use of the equipment under hazardous meteorological conditions.
- Opening sequence of sluices and logs.
- Removal of sluices under high head.
- Actions to undertake should logs get jammed or break.
- Repairs and rehabilitation of the equipment, particularly the log lifter.
- Proficiency with uncommonly used equipment, such as tongs or the manual winches.
- Increase flood understanding

11.3.3 Flow Control Operations

As seen in Section 7.1.3 - Adequacy of Discharge Capacity, if all sluices can be fully opened, the IDF of 502 m^3/s can be discharged while keeping water levels at 180.57 m (7 cm above the MNOL). However, following the investigations (Section 4 – Observations, Inspection and Document Review) it is understood that it may be difficult to fully open all the sluices under their current state. In that case, as shown in Table 9.1, the IDF can be discharged without overtopping the maximum acceptable water level under IDF conditions of 181.35 m.

Concerning the operation of the Port Severn Dams for flood control, the following recommendations are made:

- Forecasting of flood events should be well established so sluice operation can start early.
- Communication channels and chain of command should be established within the TSW so that operation can be planned in advance.
- In the event that the 1,000-year or greater flood is forecasted, sluices should be opened pre-emptively, before the water level starts to rise above the MNOL (180.50 m). The sluices most difficult to operate (those requiring the manual winch) should be opened first so that when the peak incremental daily discharge arrives, the easiest and most efficient sluices to open are left. Thus, sluices can be opened in the following order:
  - Using the manual winch: sluice 1, Dam E and Dam G.

- Using the log lifter: sluices 2 to 8.
- In the event that the 1,000-year or greater flood has not been forecasted, sluices 2 to 8 should be opened as quickly as possible. Should the operators encounter difficulties in the opening of the sluices, Section 7.1.3 – Adequacy of Discharge Capacity details which sluices should be opened to discharge sufficient flow.
- Sluice 8 should only be used when necessary, under significant flood conditions, at flow through it may erode the downstream exposed surface.

Also, while it has been demonstrated that IDF flood can be discharged under different operability scenarios, all active sluices (1 to 8 at Main Dam, Dam E and Dam G) must be fully functional at all times to make sure that risk associated with dam safety is kept as low as reasonably possible. This implies that:

- Logs, particularly log dees, must be in good condition so that they can be easily hooked with the equipment.
- The log lifting equipment (both regular and backup) must have the capacity to be operated when the logs are under high head of water.
- Staff must be trained to operate the equipment (both regular and backup) and remove/add logs under difficult conditions.

At the maximum water level under IDF conditions of 181.35 m, the Port Severn Main Dam remains safe. However, flooding may still impact property upstream. It is recommended that the Port Severn Main Dam be operated to maintain the water levels as low as possible during floods. As demonstrated in Section 7.1 - Dam Hydraulic Capacity, a fully functional Port Severn Main Dam could maintain water levels at 180.57 m under IDF conditions.

Since the Port Severn Main Dam can be adequately operated to manage water levels and discharge floods, gate mechanization would bring little value for dam safety purposes. While gate mechanization is not recommended for dam safety purposes, PCA/PWGSC may consider it for asset recapitalization purposes. This is outside the scope of the DSR but should be looked into if a Recapitalization Study is performed.

11.4 Maintenance and Surveillance

Concerning the maintenance of the Port Severn Dams, the following recommendations are made:

11.4.1 Main Dam

- Regular inspections of the Main Dam, especially of its foundations, should be performed by qualified personnel to provide any early warning signs of the development of unsafe trends in behavior:
  - Engineering inspections at a minimum frequency of 1 every 3 years.
  - Routine inspections at a minimum frequency of 1 every 4 months.
  - Continued maintenance and surveillance should be performed at whenever operating.
- All active sluices should be operated annually to make sure that they will be functional in case of emergency:
  - This includes a rotation of stoplogs to prevent deterioration of the less-used, bottom stoplogs.
  - Rotation should see the less-used logs be placed on top.
  - When operating sluice 8 for maintenance reasons, stoplogs should be inserted in the maintenance gain to protect the downstream exposed surface from erosion.
- The log lifter must be maintained in good condition at all times.
  - Equipment should be regularly inspected.
  - Operators should be trained so they may repair the equipment if necessary.
- Spare parts for the log lifter must be available on site at all times.
- Replacement rate of wooden stoplogs should be increased to 6 per year for the Port Severn Dams.

- Log book of maintenance, operation and incidents should be kept, as per SO 9.1 – Station Logbook. It would also be preferable if the logbook mentioned unusual operation required during large flood events. Log book should cover all Port Severn Dams.

11.4.2 Lock and Upstream Shoreline Wall

- Perform regular inspections of the Lock 45 and Upstream Shoreline Wall, notably to monitor the appearance of sinkholes since they could be indicative of significant degradation of the concrete wall allowing internal erosion and transportation of soil particles:
  - Engineering inspections at a minimum frequency of 1 every 3 years.
  - Routine inspections at a minimum frequency of 1 every 4 months.
  - Continued maintenance and surveillance should be performed.
- Gate mechanisms should be locked when not in use, even if the handles are safely stored in the shop.

11.4.3 Dam D

- Regularly perform the usual inspections of Dam D, notably to monitor the appearance of sinkholes since they could be indicative of significant degradation of the concrete wall allowing internal erosion and transportation of soil particles:
  - Engineering inspections at a minimum frequency of 1 every 3 years.
  - Routine inspections at a minimum frequency of 1 every 4.

11.5 Safety

11.5.1 Occupational Health and Safety

The following recommendations concern the OHS:

- Inadequate equipment and facilities
  - All safety equipment must be regularly inspected, including record keeping, to ensure proper conditions of the equipment.
  - Proper tie-off location or engineered anchors should be installed.
  - Railings should be installed at Lock 45. Tubular steel handrails, with 31 mm diameter for each of the three tubes, should be installed at the lock gates.
  - Appropriate lighting of the dam deck is required. While the installation of permanent lampposts is the ideal solution, it is an expensive measure. This measure should be considered if recommended by an eventual Recapitalization Study. In the meantime, the purchase of portable tower lights is a more flexible and cost-effective solution. Two portable light towers, with generators, are sufficient to light the work area.
  - Lights on the hydraulic log lifter should be replaced.
  - While used mostly as a backup system, the manual winches are hazardous to operate. It is recommended to investigate alternative backup systems.
- Unsafe behaviour
  - To improve safety on the operator walkways, public access should be temporarily restricted during lock and spillway operation.
  - Periodic, mandatory training on the use of safety equipment and devices should be dispensed.
  - Periodic, mandatory training on rescue and first aid should be dispensed.

Recommendations for evacuation thresholds will be outlined in the Emergency Preparedness Plan and Emergency Response Plan.

11.5.2 Public Safety

Following the identification, analysis and evaluation of risks, it is understood that there are public safety risks at the Port Severn Main Dams that must be reduced or mitigated. It is thus highly recommended that PCA pursue the Risk Management Process and engage in Risk Treatment. More details on the risks identified, analyzed and evaluated can be found in the report on Public Safety Around Dams (Reference 3).

All suggested corrective measures refer to CDA Guidelines and Associated Technical. It is important to note that the mitigation measures presented in this section should not be implemented without first going through a proper Risk Treatment analysis. This section will help PCA find means to reduce the risk rating of the most critical hazards and decide whether there is an added value in implementing the recommendation.

11.5.2.1 General Recommendations:

- Public Education:
  - Unlike power utilities and other "industrial" dam owners, PCA has the obligation of encouraging and welcoming public visits and activities while being mindful of their safety. Hence the use of extensive fencing as a risk reduction measure is often not appropriate. Instead, the use of creative means to educate both the public and its own staff may be more suitable.
  - One example to consider is the OPP/OPG "Stay Clear Stay Safe" campaign. PCA may want to consider starting/joining a similar campaign and bringing in the aspect of navigable waterways, docks and locks safety, etc.
  - Another avenue would be a brochure campaign with wide distribution in communities around the facilities and in schools, public libraries, supermarkets, internet and TSW/PCA website, etc. Currently a brochure and a video on the PCA web site only addresses safe boating in and out of locks.
- Standing Orders:
  - The TSW Standing Orders reviewed cover a variety of topics (health and safety, employment conditions, operation instructions, etc.). It is not clear how this document is made accessible to staff and what means there are to navigate through it and access the appropriate section when needed. As part of the PCA Risk Management Process, a distinct Standing Order should be added to emphasize the TSW/PCA staff's role regarding public safety.
- Training:
  - It is paramount that operators and site personnel be trained to be cognisant of the site hazards and potential risks. It is important that the operators be able to recognise and deal with hazardous situations and the public accordingly.
  - It is recommended that sessions be held at least once a year during a relatively quiet time (e.g. winter), to review and discuss the public safety policy, the Standing Orders, past incidents and lessons learned, etc. These sessions could be held in conjunction with OHS meetings, for example, and attendance should be mandatory.
- Review and Audits:
  - As part of the Public Safety Management System, supervisory staff and management staff should carry out reviews and audits to make sure directives and procedures are understood and followed. For example, the Standing Orders require reporting of public incidents using specific forms. No such reports could be located in the field or office during site visits.
- Incident Log:
  - Record keeping processes need to be reviewed and enforced. All records of incidents and near misses should be accessible and available during the periodic reviews of public safety Risk Assessments and of the Public Safety Plan for each site. It is recommended that a site specific Public Safety Incident Report Log be started. A template is available in Appendix D of the Public Safety Risk Assessment Report (Reference 3).

- Public Safety Plan:
  - A Public Safety Plan for the site should be developed, reviewed and updated periodically with reviews of the public safety Risk Assessment. The Public Safety Plan should address the use of the dam structures by the public.
- Emergency Response Plan (ERP):
  - As part of the Public Safety Plan, a site specific ERP should be implemented.
  - The ERP should think about operational and non-operational hours. For example, the new format for warning and danger signage should include the name of the dam and the emergency phone number. It would also be a prudent to include GPS coordinates for remote dams.

11.5.2.2 Site Specific Suggested Corrective Measures - Main Dam

All suggested corrective measures refer to CDA Guidelines and Associated Technical Bulletins. A proper Risk Treatment analysis should be undertaken. This section aims to help PCA find means to reduce the risk rating of the most critical hazards. Figure 11.1 presents an example of public safety control measures at a control dam.

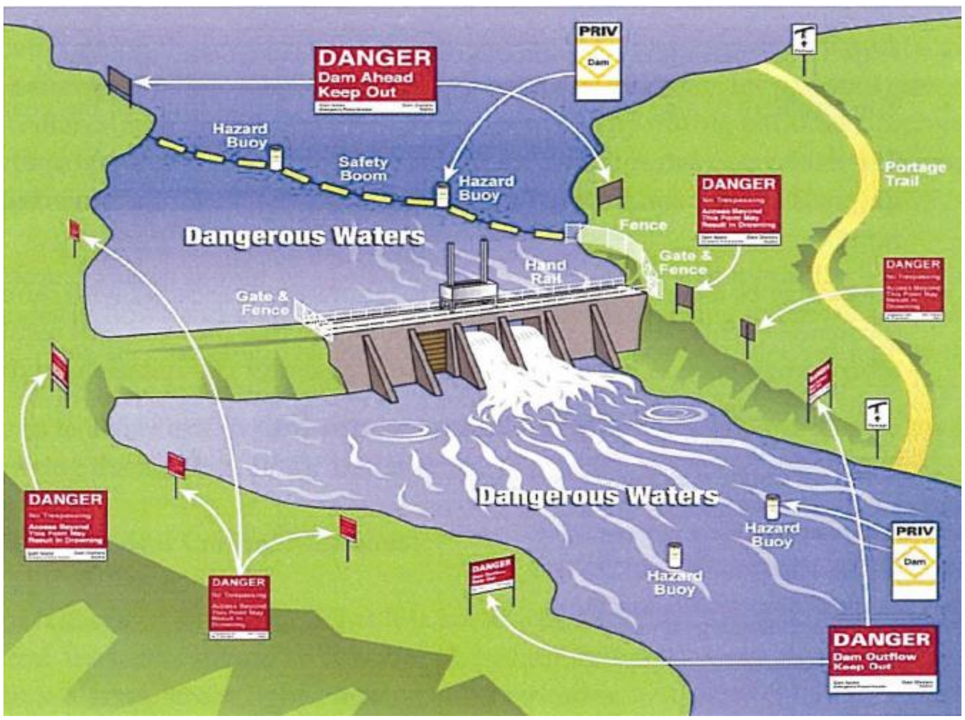

Figure 11.1 Example of Public Safety Control Measures at a Control Dam

Signage: It is recommended to install yellow warning signs at the entrance of the upstream and downstream areas to prevent the public from entering a hazardous zone. In the downstream area, the sign could be added downstream of the Highway 400 bridge and advise boaters to keep left to avoid the hazardous sections of the spillway. The onsite signage doesn't meet the current standards in terms of format, content and font size. Signage should be added on the shores where there is access to the structure to warn public about the hazard and prohibited activities. Example of adequate signage can be found in Appendix F of the Public Safety around Dams Risk Assessment Report (Reference 3).

Safety Booms and Buoys - Upstream: PCA should reconfigure the guard wire. Proper safety booms installed outside of the danger zone would be more visible and more effective at keeping public out of the hazardous zone and rescuing stranded boats/swimmers. An inverted V-shape promoting self-rescue is preferred as shown in

Figure 11.2. The current delineation of the headpond by the guard wire is not appropriate and should be reconfigured so that the guard wire is not within the danger zone but at the upstream limit where the flow velocities are lower.

Safety Booms and Buoys - Downstream: Even though the lock is closed when flow exceeds 90 m^3/s, warning buoys should be considered in the downstream area to direct boat traffic away from turbulence created by the discharge from the Main Dam spillway. Safety booms should also be considered but the velocity of the flow could make the design infeasible.

Fencing: Fencing should be added on Dam D to prevent access to the upstream area/headpond. The steel pipe railing on the downstream and upstream lock piers doesn't comply with current standard (References 46 and 42) and doesn't secure the entire pier. The chain link fence on the Main Dam deck is not deemed to be sufficient, considering that the gap at the bottom of the fence is larger than 100 mm (see Photo M5 of Appendix B of the Public Safety Risk Assessment Report, Reference 3).

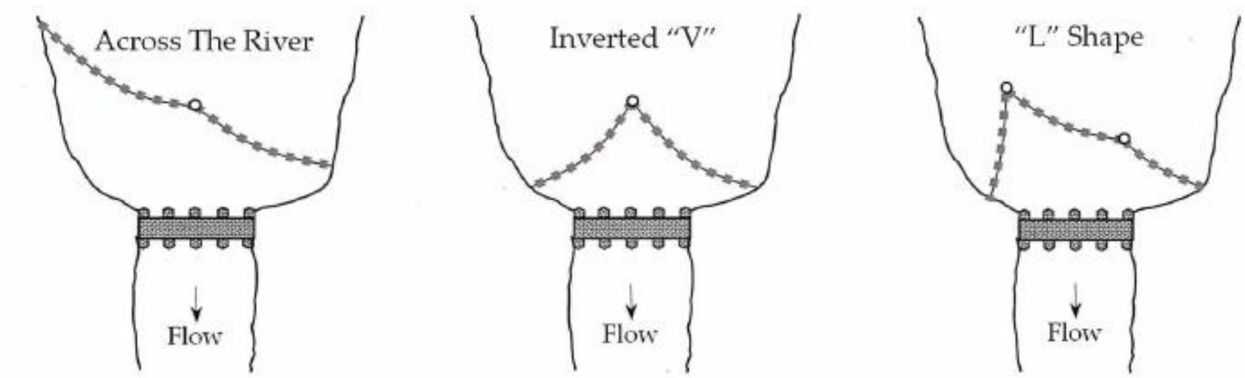

Figure 11.2 Typical Safety Booms Shapes

Life Rings: There should be at least one life ring added on the left bank near Dam D and another in the middle of the dam for the headpond. This is required given the length of the dam.

Portage Route: A sign should encourage canoe/kayaks to use the lock facility for portage instead of looking for an alternative route. This exposes the public to hazards in the spillways of the dams. In addition, the existing leakage from the lock wall should be resolved before inviting use of the stairs. This is because the stairs are slippery and dangerous for canoers/kayakers to use. It would be preferred to use the right shore giving access directly to the PCA Visitor Center. This portage route would be safer and would avoid public using boats in the dangerous zones. It will also avoid portage on the lock gates which is dangerous.

Pedestrian Access: The pedestrian crossing of the river should be reconfigured at Main Dam. A cantilever platform/footbridge reserved for the public, outside of the operation area, could be installed to secure the public crossing the dam. Also, the signalling of the traffic lights could include a pedestrian phase.

Mechanical / Electrical Equipment: Stoplog gain covers should be secured so they cannot be lifted by the public.

Public Safety Incident Log: An individual Public Safety Incident Report log should be started as soon as possible. The same log can be used for all Port Severn Dams. A template is available in Appendix D of the Public Safety Risk Assessment Report.

Audible/Visible Signaling Device: It is suggested that the PCA evaluates the possibility of installing an audible signaling device to alert the public present in the spillway/downstream location to changing flow conditions.

Ladders: There is no ladder to climb back on the shore/structure from the water making self-rescue almost impossible. Ladders should be installed at the end of the safety booms to help stranded swimmer/boater to climb back on the concrete walls/piers.

11.6 Closure of the Reservoir Rim

To ensure that the reservoir is safe under the IDF condition, there is one option for closing the reservoir near the wetland area, south of Dam A. For Areas 1, 2 and 3 there are no works to be recommended, however, there is the need to address the issues in emergency preparedness and response plans, in addition to the informing of issues to Authorities. Associated costs are defined in the condition assessment report of Dam A.

11.6.1 Wetland Area

There is an obvious drainage problem in the area south of Dam A. This will have to be solved to avoid threatening the roads and surrounding population.

There are several possible solutions to the drainage problem in the area south of Dam A. One is the construction of new structures in suitable locations. Rebuilding Dam A to the original crest elevation would once again protect the lowlands agains flooding from the reservoir. However, this does not alleviate the issue with water accumulation in the wetland area.

Parks Canada is constrained with regard to property ownership at Port Severn and cannot build outside of currently owned land unless it proceeds to purchasing additional land. Given the considerable costs of property acquisition for PCA and the fact that the issue of water accumulation stems from the urban development, this is an issue that must be remedied by the municipalities in the area.

To avoid the ponding of water in the wetland area, a ditch should be constructed along the yellow path in Figure 8.7. Also, the culverts of St-Amant Road and Port Severn Road should be lowered to allow the water to drain from the captive wetland.

Therefore, in conjunction with the rehabilitation of Dam A, AECOM recommends to inform the municipalities of the deficiencies in the wetland area and suggest that these deficiencies are remedied.

The rehabilitation of Dam A is presented in detail in the Review of the Structure Status and Dam Classification Report for Dam A, Dam B, Dam B1, Dam F and Little Go Home Bay Dam.

11.6.2 Areas 1, 2 and 3

There is no need to construct or alter the existing structures in this area. However, there is a risk to the local roads, highway and population at the maximum water level under IDF conditions. As discussed in Section 8.4.2, this is due to the roads acting as water-retaining structures which could lead to failure.

Therefore, it is recommended that the Emergency Preparedness Plan (EPP) and Emergency Response Plan (ERP) address the risks associated with the IDF water level in this area. Ministry of Transportation Ontario (MTO) and Emergency Management Ontario (EMO) should be notified of the potential hazards during elevated water levels at Port Severn.

During an elevated water level, roads and Highway 400 should be closely monitored to ensure that there is no significant difference in water elevation on either side of the roadway. If there is a difference, a significant risk to failure of the roadway exists.

11.7 Cost Estimates and Scheduling

Table 11.1 presents a Class D estimate of the costs associated with the recommended remedial measures. A Class D estimate is an “indicative” estimate, which is defined by the Treasury Board of PWGSC (Reference 52) as:

> *This estimate provides an indication of the total cost of the project, based on the user’s functional requirements to the degree known at the time. It is based on historical cost data for similar work, suitably adjusted for such factors as: effect of inflation, location, risk, quality, size, and time. All related factors affecting cost are considered to the extent possible. Such an estimate is strictly an indication (rough order of magnitude) of the project total cost and completion date. This estimate is used to establish the indicative estimate required by the Treasury Board for Preliminary Project Approval.*

The Treasury Board (Reference 53) further mentions that an indicative estimate is usually based on “an operational statement of requirement (SOR), a market assessment of products and technological availability that would meet the requirement and other considerations such as implementation, life cycle costs and operational savings”. More to the point, the Class D estimate must be in unit cost analysis format based upon a comprehensive list of project requirements (i.e. scope) and assumptions.

For each significant issue concerning the dam safety, a recommendation is detailed. The recommendations previously detailed are summarized and prioritized in Table 11.1. Priority is given based on:

- Risk of occurrence.
- Significance of potential negative impacts.
- Resources (cost, time, effort) required to implement.

Recommendations that are prioritized as “Short term” should be done within the next two years, “Medium term” within the next five, while “Long term” does not have a definite time line. All recommendations should be implemented. Table 11.1 also presents a preliminary costing. The degree of accuracy is approximately 20% (Reference 52) and as such these costs should not be used for tender documents.

For Short term, preliminary costs for the recommendations amount to approximately $ 1,965,400 including engineering design, construction management and contingency.

For Medium term, preliminary costs for the recommendations amount to approximately $ 38,000 including engineering design, construction management and contingency.

Table 11.1 Cost Estimation of Remedial Measures

Item	Description	Unit	Quantity	Unit Price	Total Estimated Price		
					Short Term	Medium Term	Long Term
1	**REMEDIAL WORKS**				**$1,196,800**	**$27,000**	
1.1	**Mobilization**				**$100,000**	**$10,000**	
1.1.1	Mobilization and demobilization	unit	1	$100,000	$100,000	$10,000	
1.2	**Structural / Civil**				**$551,100**	**$17,000**	
1.2.1	**Concrete repairs**						
1.2.1.2	Crack and joint repairs *[1]	m	200	$985	$197,000		
1.2.1.3	Repairing deteriorated concrete *[2]	m²	25	$8,200	$205,000		
1.2.1	**Lock wall stability rehabilitation**						
1.2.1.7	Passive anchors - 35M bars	kg	300	$5	$1,500		
1.2.1.8	Drilling and Sealing	m	40	$90	$3,600		
1.2.3	**Pier stability rehabilitation**						
1.2.3.1	Drilling machine & drilling hole	hours	40	$750	$30,000		
1.2.3.2	Crane and installation	unit	1	$80,000	$80,000		
1.2.3.3	Post-tensioning bar (Dywidag) - fabrication	unit	8	$1,500	$12,000		
1.2.3.4	Post-tensioning bar (Dywidag) - transportation	unit	1	$5,000	$5,000		
1.2.3.5	Injection grout anchor	unit	8	$625	$5,000		
1.2.3.6	Post-tensioning bar tests	unit	3	$4,000	$12,000		
1.2.4	**Dam A concrete repairs (closure of reservoir rim)**						
1.2.4.1	Cleaning the upstream face and work area	unit	1	$5,000		$5,000	
1.2.4.2	Concrete pump	unit	1	$10,000		$10,000	
1.2.4.3	Formwork and new concrete	m^3	2	$1,000		$2,000	
1.3	**Geotechnical**				**$4,500**		
1.3.1	Excavate and fill eroded soil material at Dam D	unit	1	$4,500	$4,500		
1.4	**Mechanical**				**$541,200**		
1.4.1	**Replacement of dam deck rails**						
1.4.1.1	Rails, including installation	m	225	$80	$18,000		
1.4.1.2	Anchors every 2 m	unit	115	$80	$9,200		
1.4.2	**Replacement of log lifter**						
1.4.2.1	Purchase of new Atlas Polar log lifter	unit	1	$500,000	$500,000		
1.4.3	**Gains and stoplogs**						
1.4.3.1	Replacement of steel angles in gains	unit	8	$1000	$8,000		
1.4.3.2	Increase rate of replacement of logs to 6/year*[3]	/year	6	$1000	$6,000		
2	**OPERATOR SAFETY**				**$168,000**		
2.1	**Lighting**						
2.1.1	**4x450 W LED light tower w/ 3,000VA generator**	unit	2	$12,000	$24,000		
2.2	**Fall-arrest systems**						
2.2.1	Installation of engineered anchors	unit	8	$17,000	$136,000		
2.3	**Protective railings**						
2.3.1	Steel 3x31mm tube handrails w/ primer, installed	m	40	$200	$8,000		

Item	Description	Unit	Quantity	Unit Price	Total Estimated Price		
					Short Term	Medium Term	Long Term
2.4	**Safety Awareness**				**TBD**		
2.4.1	Ensure operators are properly attached when operating				no add. cost		
2.4.2	Provide training (rescue, safety)				TBD		
2.4.3	Restrict public access during operations				no add. cost		
2.4.4	Periodically audit working procedures				no add. cost		
2.5	**Manual Winches**						
2.5.1	Investigate alternative backup systems				TBD		
3	**PUBLIC SAFETY**				**TBD**		
3.1	Perform Risk Treatment (including the identification and assessment of options), as per the CDA Risk Management Process.				TBD		
3.2	Implement mitigation measures identified and retained following Risk Treatment				TBD		
4	**OPERATIONS**				**TBD**		
4.1	Provide training				TBD		
4.2	Update TSW Standing Orders				no add. cost		
5	**MAINTENANCE, SURVEILLANCE, PLANNING**				**TBD**		
5.1	Prepare EPP, ERP and OMS manual				TBD		
5.2	Update DSR every 10 years						TBD
5.3	Perform engineering inspection at least every 3 years					no add. cost	
5.4	Perform routine inspections at least every 4 months				no add. cost		
6	**ENGINEERING AND MANAGEMENT**				**$273,000**	**$5,000**	
6.1	Engineering Design and Construction Management				$273,000	$5,000	
TOTAL							
				Subtotal	$1,637,800	$32,000	$0
				Total contingency (20 %)	$327,600	$6,000	$0
Total with contingency (taxes are not included)					**$1,965,400**	**$38,000**	**$0**

NOTES

* [1]	Including cleaning, marking, injection, labour and turbidity curtain
* [2]	Including removal of old concrete, cleaning marking, formwork, rebar, labour, new concrete and turbidity curtain
* [3]	Suggested rate of replacement is 30 logs/year in the sector, which comes down to 6 logs/year for Port Severn
TBD	To be determined by PCA, through further cost estimations or by requesting proposals
no add. cost	No additional cost expected as this can be done by PCA full-time employees as part of their regular work

12. Conclusions

This section summarizes and concludes the Dam Safety Report for the Port Severn Main Dam. The conclusion reviews all aspects covered in the DSR.

Currently, the Port Severn Main Dam, including Lock 45, the Upstream Shoreline Wall and Dam D, does not meet the regulatory requirements for the following reasons:

- The dam was shown to be unstable under ice loading scenarios only. It is structurally sound under all other loading scenarios.
- While Port Severn Main Dam can theoretically be operated to discharge the IDF, reliability in operation of certain sluices poses a risk.
- Improvements should be made to provide a safer working environment.

12.1 Dam Classification

The Main Dam is classified as a Significant dam. This is due to the fact that there are zero fatalities to permanent population in the event of a dam break and a maximum of 12 transient people. However, there will be damages not exceeding $12 Million to property and unquantified economic losses from impacted tourism activities. There are no anticipated losses to cultural and heritage sites or natural habitat.

The resulting IDF for the Main Dam is the 1,000 year flood. This corresponds to a flow of 502 m^3/s at a maximum water level of 181.35 m in Gloucester Pool. The water level under IDF conditions can be reduced to 180.57 m by operating all sluices, which is recommended to reduce flood damage in the reservoir.

The 500-year earthquake is chosen as the DBE. The low end of the range is selected to reflect the transient use of 12 people.

12.2 Current Condition of Structures

From the site inspection it was seen that the Main Dam is in a fair condition. There are cracks and damage to concrete in several places that require repair. There is no erosion, seepage or ground settlement at the Main Dam.

Lock 45 and the Upstream Shoreline Wall are in a fair condition. As with the Main Dam, there are localized cracks and concrete damage which will require repair though some visible repairs have been done on the concrete. Most notably, water leaks through a large crack at the top of the stairs on the left side and a large concrete piece is detached from the left wall of the lock. Sinkholes have been observed in the past near the Upstream Shoreline Wall and were recently filled with sand and gravel. These sinkholes suggest underground erosion due to deterioration of the concrete wall.

Dam D is in a poor condition, but stable. There is some damage to the concrete and visible sinkholes downstream. This is likely a result of piping due to deterioration of the upstream concrete wall.

Continued surveillance and monitoring of the facilities must be performed.

A Recapitalization Study would quantify the cost and benefit of refurbishing the Port Severn Main Dam to a condition where it will provide another 40+ years of useful life. While not necessary to bring the structures to a standard recommended by the CDA, PWGSC/PCA could consider including this study in a package with the recommended works and remedial measures to benefit from economies of scale. Such economies include, but are not limited to, administrative and legal fees associated to tender and contracting, mobilization and demobilization, on-site and off-site works supervision, quality control and more.

12.3 Structural Integrity of the Structures

In general, all structures are stable (Dam D, Lock 45 and Upstream Shoreline Wall), except for the Main Dam. The case where stability is a problem is during the winter, when there is ice cover in the reservoir and ice pressure is being exerted on the Main Dam piers.

The installation of post-tensioned anchors through the dam and into the bedrock will provide protection against overturning. Temporarily, staff must break the forming ice on the entire length of the upstream face of the dam during winter to reduce ice forces. Water agitators could also be considered, yet are quite expensive for a temporary solution.

12.4 State of the Mechanical Equipment

Several issues were noted during the DSR regarding the mechanical aspects of the dam and lock. It was noted that gears, winches, frames and wheels are in good condition and are well maintained.

A functional test was performed on Lock 45 and it was demonstrated that the structure is in an acceptable working condition.

It is recommended to replace the hydraulic log lifter. While it is maintained in an acceptable working condition and can be relied upon to remove sufficient logs such as to allow discharge of the IDF, its use poses an OHS risk. It cannot be used to press the logs tightly together so as to cut off the water flow. The operators must resort to jacking the stoplogs, which requires entering the sluices.

The manual winches used over sluices 1 and 9 and as a backup for the hydraulic log lifter are functional but very old. They are neither reliable nor efficient. It is recommended to study alternatives. This is also a safety concern.

The rail tracks on the dam require replacement. These also have reached the end of their useful life.

Continued surveillance and monitoring of the equipment must be performed.

12.5 Freeboard Adequacy

For the Main Dam, there is adequate freeboard under both the normal and minimum freeboard cases. For Lock 45, Dam D and the Upstream Shoreline Wall, normal freeboard criteria are met, but not minimum freeboard as there is overtopping by waves during the IDF. However, this wave overtopping is not likely to be an issue as the ground in these areas is flat.

12.6 Operation

The Main Dam was shown to have been designed to pass the IDF safely. However, it requires that the sluices, logs and log lifter be well-maintained and remain fully functional. Currently, the bottom four logs of all sluices cannot be relied upon for added discharge capacity as the operators have great difficulty removing them under considerable water depth. While the Port Severn Dams can still be operated to safely discharge the IDF, relying on the operation of the top eight logs of sluices 1 to 8 and/or Dam E and Dam G, it is highly recommended that measures be implemented to allow full operation of all logs.

Assuming that three teams of two operators are available, that operators are well-trained, facilities well maintained and sluice opening has been anticipated, the sluices can be opened under the required time to pass the IDF. The current staffing (four full-time experienced operators) is insufficient to operate safely the dams during significant floods as the Sector Manager is forced under these conditions to join the operating staff to form a third team. At least two more operators should be available to complete the teams and free the Sector Manager to pursue managerial duties. Nonetheless, the lack of redundancy (no back-up crews) in staff numbers also poses a risk. Should an operator not be able to perform his duties during a significant flood, it is likely that the dams could not be operated in time to safely discharge. Part-time or back-up crews should also be available.

Since the Port Severn Main Dam has sufficient capacity to safely discharge floods (given that the recommendations on maintenance and sluice operation be implemented), gate mechanization is not recommended for dam safety purposes. PCA/PWGSC may consider it for asset recapitalization purposes should a Recapitalization Study be performed.

12.7 Safety

12.7.1 Occupational Health and Safety

With regard to the operation of the Main Dam and Lock 45 several safety issues were identified.

In summary, it was seen that safety rules are not observed diligently, mostly as there is currently no secure way for the operators to attach themselves during operations on the Main Dam. In addition, the log jacking operation is hazardous and lighting is absent, making night operation difficult.

Use of the manual winches is physical demanding and hazardous. While this can be tolerated as they are mostly used as a backup system, it is nonetheless recommended to investigate alternative backup systems.

On Lock 45, manoeuvring around the locks is hazardous as the railings do not meet the standard. There are no capacity markings on the manual winches. These should be added.

Finally, a comprehensive training program should be given to operators. This is required to ensure that safe and efficient operation can occur in fair and adverse weather conditions.

12.7.2 Public Safety

A Public Safety Risk Assessment was completed at the site of the Port Severn Dams. This assessment was carried out using the guidance presented in the Canadian Dam Association 2011 Guidelines for Public Safety around Dams. It is recommended that PCA pursue the Risk Management Process and engage in Risk Treatment.

Although not included in the Risk Assessment step according to CDA Guidelines, additional risk treatment measures have also been suggested for each dam based on the CDA Guidelines and Technical Bulletin. This Risk Assessment report should be followed by an identification and assessment of the risk mitigation options, prioritization and implementation being the Risk Treatment. The Risk Assessment Tools in Appendix C of the Public Safety Around Dams Report (Reference 3) are meant to assist PCA in this process.

Considering the general safety measures, it was seen that more public education should be given, whether through the distribution of brochures or other campaigns. PCA staff should also be trained to recognise potential risks and site hazards as well as deal with situations where members of the public may find themselves in danger. The practice of performing reviews and audits and keeping incident logs should be introduced. Furthermore, a public safety plan and an emergency response plan should be prepared and implemented.

With regard to the public use of the area around Main Dam and Lock 45 several safety issues were identified. The site presents significant hazards, which could result in risk of serious injuries and fatalities when combined with the high degree of public activities and interaction. The existing onsite risk reduction measures such as signs, guard wires, railings and procedures do not meet present day standards. These items need to be reviewed, upgraded and adjusted in location, numbers, format, designs and construction.

More specifically, signage meeting current standards should be added, life rings, ladders and signaling devices should be installed, guard wire upstream of the dam should be reconfigured, buoys should be added downstream, pedestrian crossing of the river should be reconfigured and stoplog gain covers should be secured. Even though the lock is closed when flow exceeds 90 m^3/s, warning buoys should be considered in the downstream area to direct boat traffic away from turbulence created by the discharge from the Main Dam spillway.

Finally, a proper Risk Treatment analysis should be undertaken as the public safety review performed for this DSR aimed more at identifying issues rather than analyzing risk and evaluating the added value of implementing measures.

12.8 Impoundment Review

With water at the normal maximum operating level of 180.50 m, there are no major issues with the impoundment of the reservoir. There is some minor flow through culverts in areas northwest of Main Dam, under Highway 400, though this is not considered to be a hazard to public and property under normal operating level.

At the maximum water level under IDF conditions of 181.35 m, water can exit the reservoir though culverts. These are located in the areas described above, and also in the wetland area south of Dam A. Here, water is ponding due to an incorrectly located outlet under Port Severn Road. This poses a risk to the public and property as the roads may begin to behave as dams under elevated water levels.

To close the reservoir rim and provide protection from flooding under the IDF condition, Dam A should be rebuilt to its original crest level. However, there is an obvious drainage problem in the area south of Dam A. This will have to be solved to avoid threatening the roads and surrounding population. This is an issue which must be remedied by the municipalities in the area. There is a need to address these issues in emergency preparedness and response plans, in addition to informing municipalities and authorities of the deficiencies. Associated costs are defined in the condition assessment report of Dam A.

12.9 Summary of Recommendations

To ensure the safety at the Port Severn Main Dam, it is recommended that PCA:

- Implements the remedial measures detailed in Section 11.2 - Rehabilitation / Repair of Assets within the time frame prescribed.
- Immediately implements all OHS recommendations detailed in Section 11.5 - Safety.
- Reviews the TSW Standing Orders and insert the modifications listed in Section 9.1 - Standing Orders Review.

- Continues the maintenance and surveillance of the Port Severn Dams and implements the recommendations detailed in Section 11.4 - Maintenance. This includes the periodic review of the DSR and engineering and routine inspections, as outlined in Section 6.16 - Future Dam Safety Review Program.
- Provides the required training to its staff, including operation of the sluices under emergency situations.
- Considers ordering a Recapitalization Study (not part of DSR recommendations) while proceeding with the implementation of the recommendations and remedial measures.
- Proceed with the completion of the following documents: EPP, ERP and OMS. Each document should cover all the Port Severn Dams. These should be distributed to concerned parties and implemented as required.

Preliminary costs for the recommendations amount to approximately $ 2,003,400 including engineering design, construction management and contingency.

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60. US Army Corps of Engineers (USACE), The Shore Protection Manual, Coastal Engineering Centre, 1984.
61. USBR, Hydrodynamic Pressures on Dams due to Horizontal Earthquake Effects, 1952.
62. USBR, A Procedure for Estimating Loss of Life Caused by Dam Failure, 1999.
63. Watson & Associates Economists Ltd, Severn DC Report, 2009.

Appendix A
Drawings

- 001 – General Location Plan
- 002 – Location Plan - Main Dam
- 005 – Main Dam - Plan, Elevation, Sections and Detail
- 006 – Main Dam Sections
- 007 – Lock 45 and Upstream Shoreline Wall - Plan, Elevation and Details
- 008 – Lock 45 and Upstream Shoreline Wall – Sections
- 010 – Dams C, D, F - Plan, Elevations and Sections

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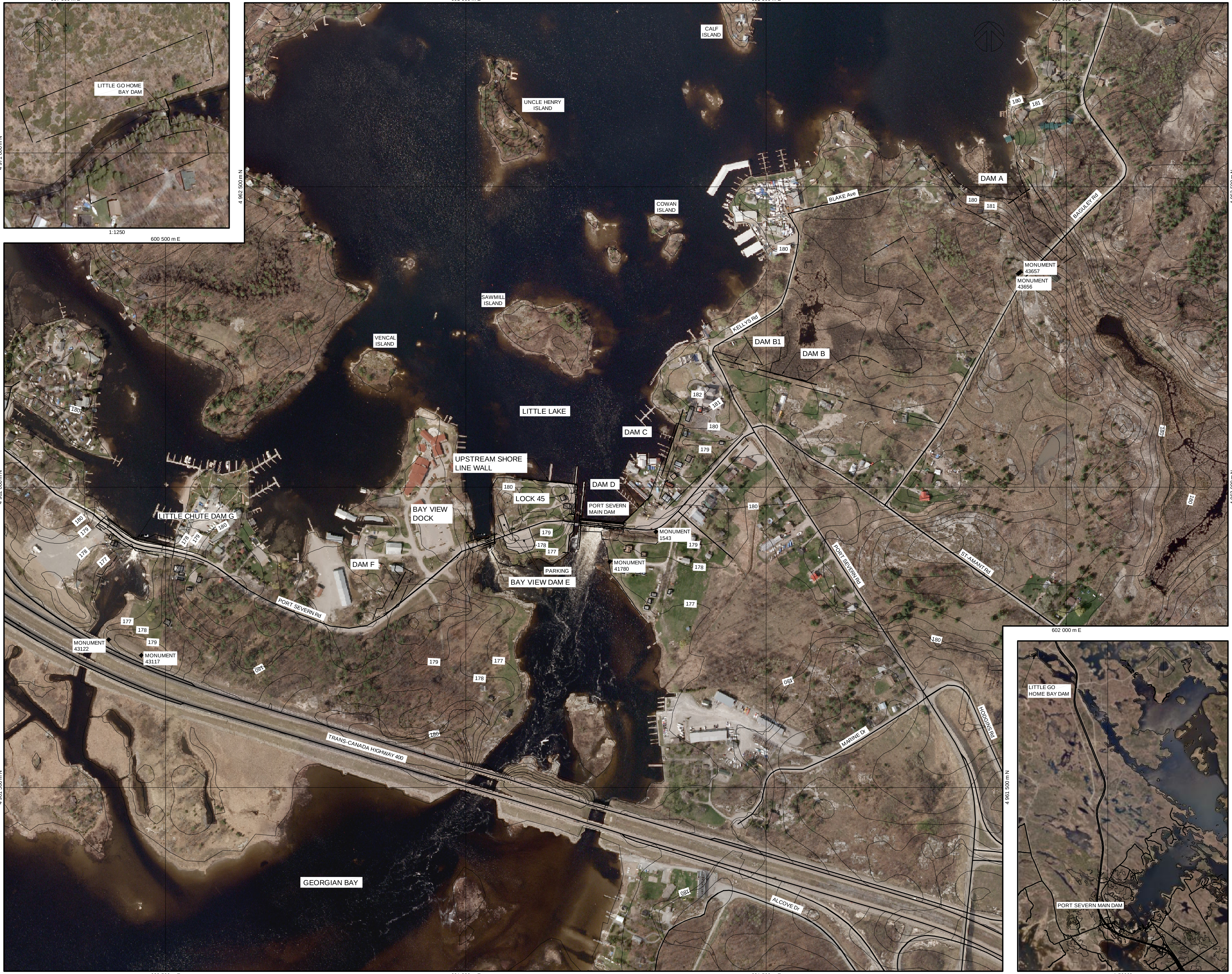

LOCATION PLAN
1:2500

PCA PROPERTIES

CONTOUR

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DAM SAFETY REVIEW

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GENERAL
LOCATION PLAN

Drawn by / Dessiné par: D. DESLAURIERS
Designed by / Conçu par:
Approved by / Approuvé par:
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MAIN DAM PLAN
1:400

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DAM SAFETY REVIEW

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LOCATION PLAN
UPSTREAM SHORE LINE WALL
LOCK 45, MAIN DAM
DAM D AND DAM C

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Drawing Number/ Numéro du Dessin: 002
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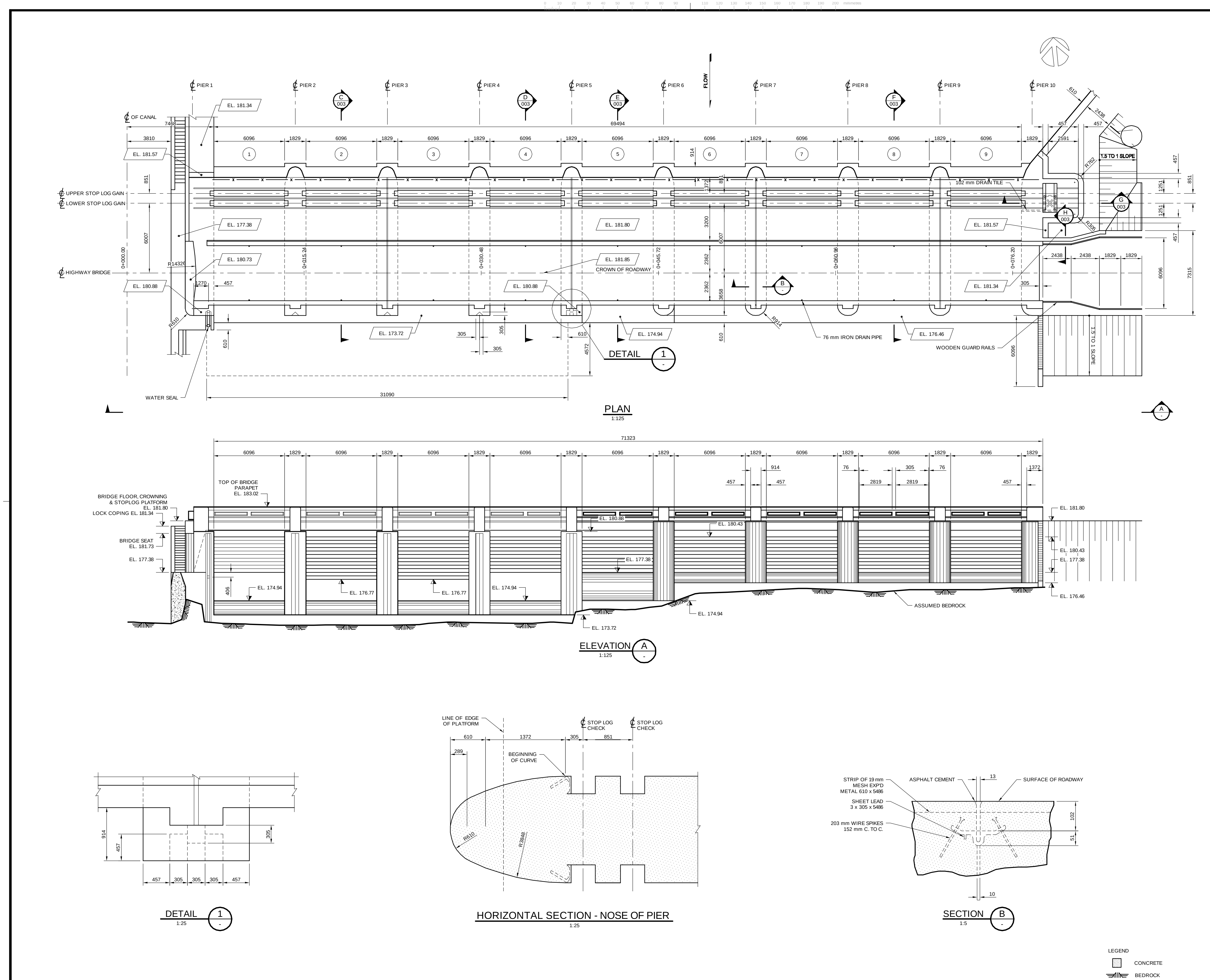

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2-THIS DRAWING HAS BEEN PREPARED USING SURVEYS DONE IN MAY 2012 ALONG WITH DRAWING C-11-3379 TRENT CANAL, SEVERN DIVISION, FINAL PLAN OF DAM AND HIGHWAY BRIDGE AT PORT SEVERN, AS BUILT DATED FROM FEBRUARY 19, 1918.

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DAM SAFETY REVIEW

Drawing title / Titre du dessin
MAIN DAM
PLAN, ELEVATION, SECTIONS AND DETAIL

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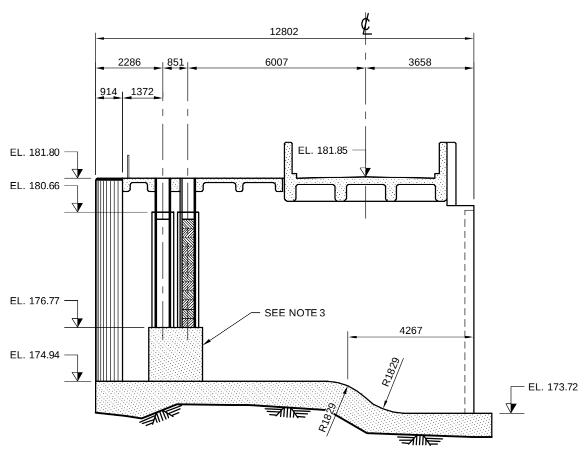
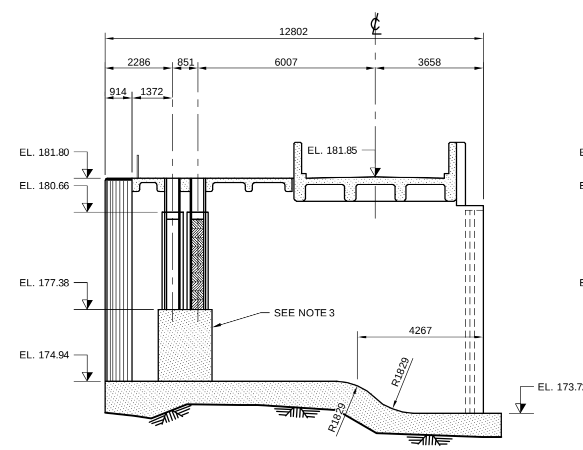
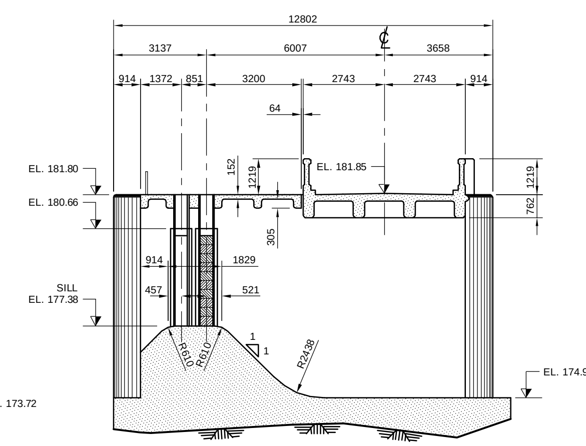


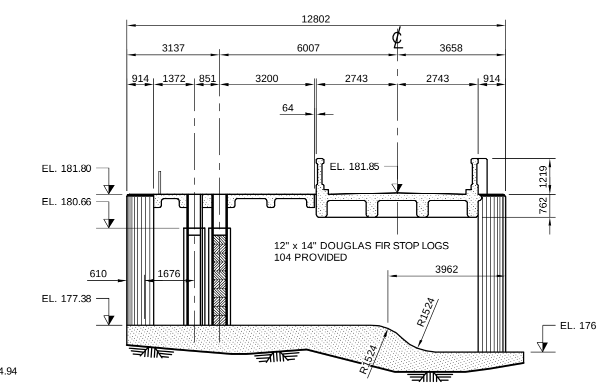
MAIN DAM - SLUICE 2 AND 3 (C / 005)
1:100



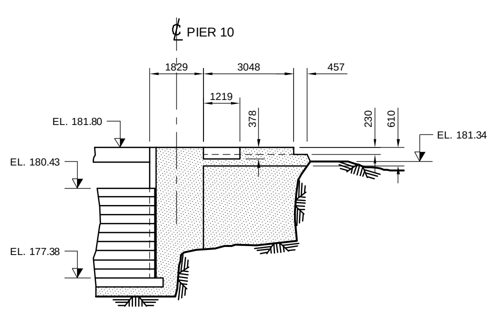
MAIN DAM - SLUICE 1 AND 4 (D / 005)
1:100



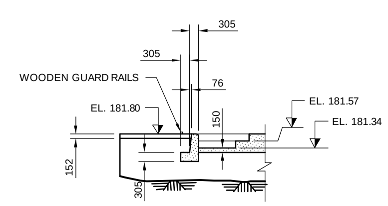
MAIN DAM - SLUICE 5 (E / 005)
1:100



MAIN DAM - SLUICE 6 TO 9 (F / 005)
1:100



MAIN DAM (G / 005)
1:100



MAIN DAM (H / 005)
1:100

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3-TEMPORARY WALL BUILT AS SHOWN IN SLUICES 1, 2, 3 & 4 MOULDING TO BE PLACED AT ENDS AND UNDER SAME TO PREVENT BENDING WITH PIERS AND SILLS, 1" x 4' REINFORCING RODS TO PREVENT SHEARING LAID HORIZONTALLY AND PLACED SO THAT ENDS PROJECT INTO GAINS 9", SPACED VERTICALLY EVERY 6".

LEGEND

- CONCRETE
- BEDROCK
- WOODEN STOPLOG

No.	Description	Dwn.By Des.Par	Date
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PORT SEVERN DAMS
DAM SAFETY REVIEW

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MAIN DAM
SECTIONS

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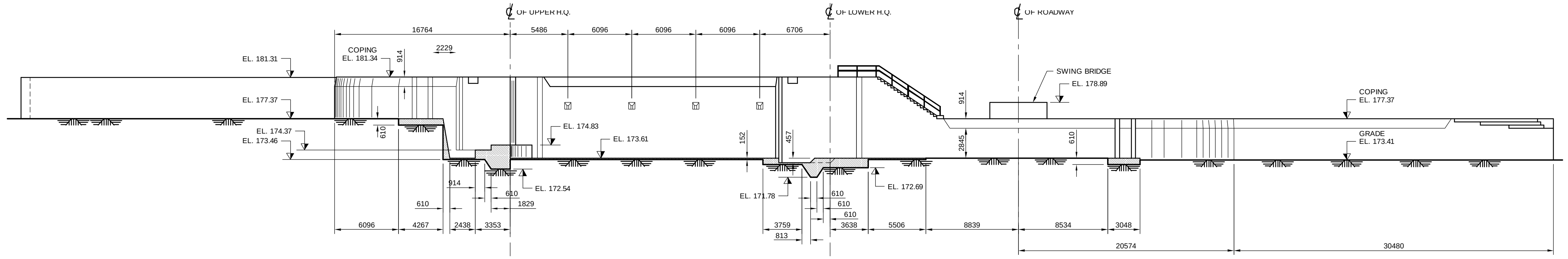
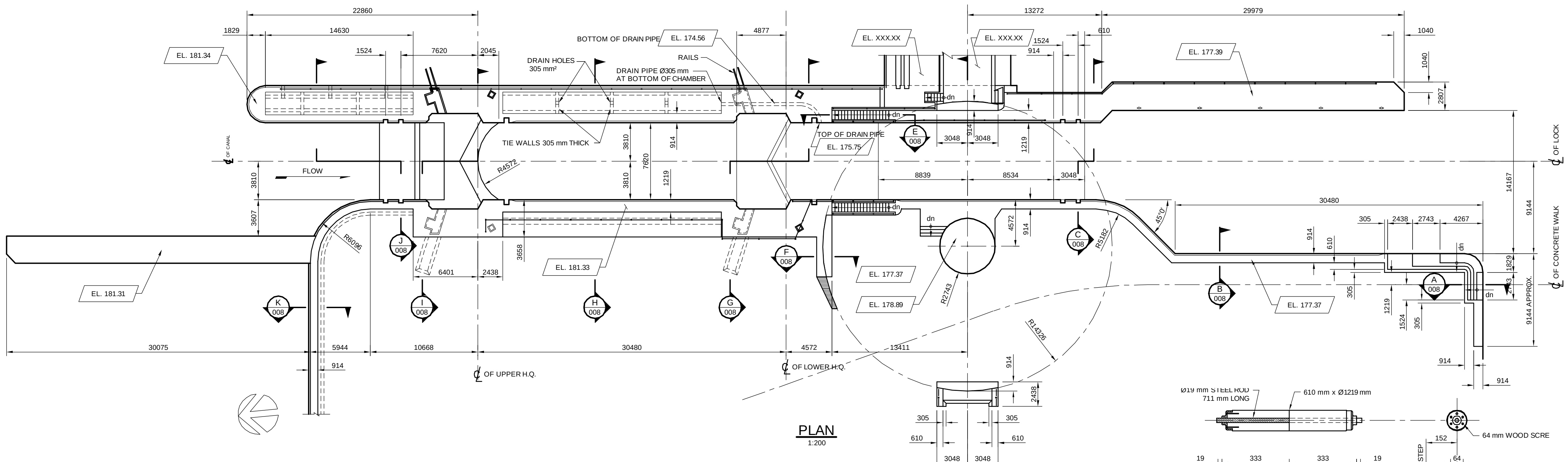
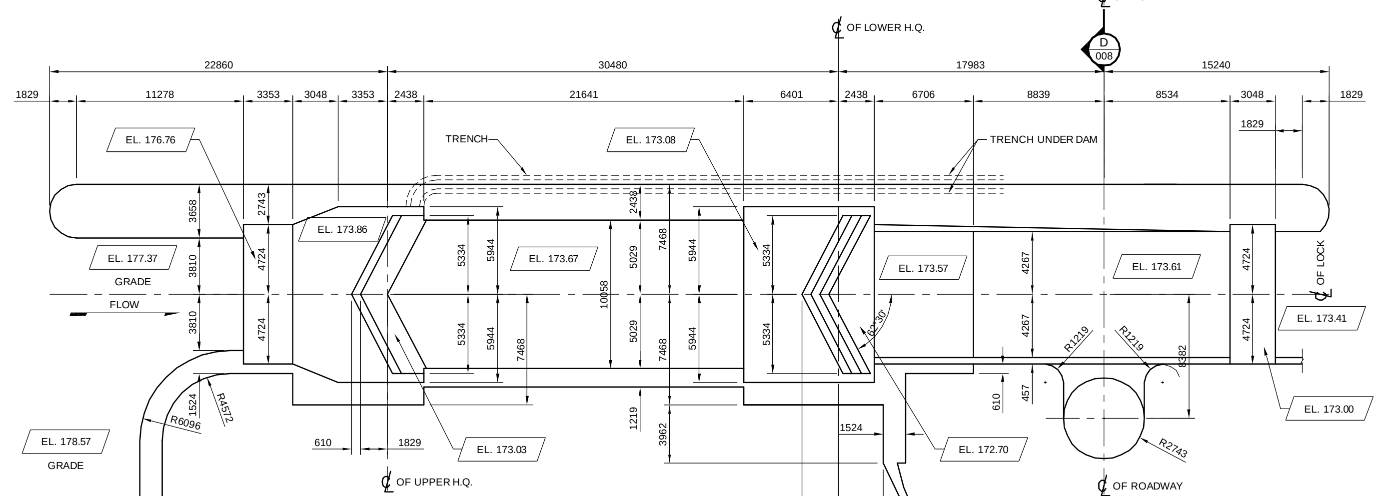


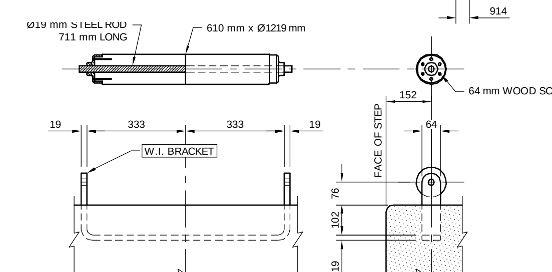
SECTIONAL ELEVATION ON CENTRE LINE OF LOCK
1:200



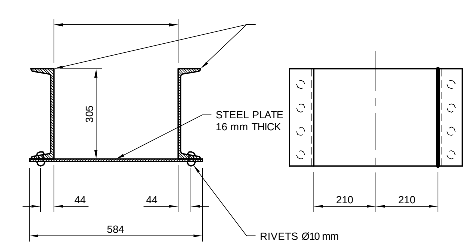
PLAN
1:200



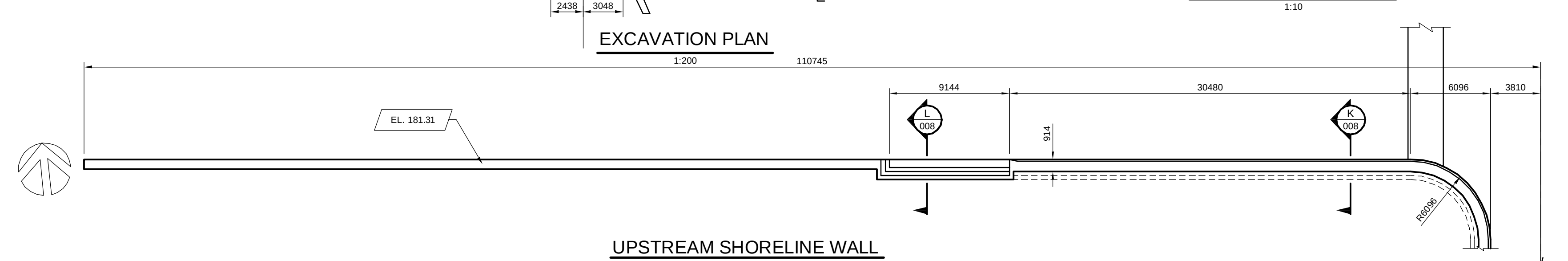
EXCAVATION PLAN
1:200



DETAIL OF ROLLER AND BRACKET
1:10



CROSS SECTION ELEVATION OF LOWER END
STEEL STOPLOG GAIN
1:10



UPSTREAM SHORELINE WALL
PLAN
1:200

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LEGEND

CONCRETE

BEDROCK

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PORT SEVERN DAMS
DAM SAFETY REVIEW

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LOCK 45 AND
UPSTREAM SHORELINE WALL
PLAN, ELEVATION
AND DETAILS

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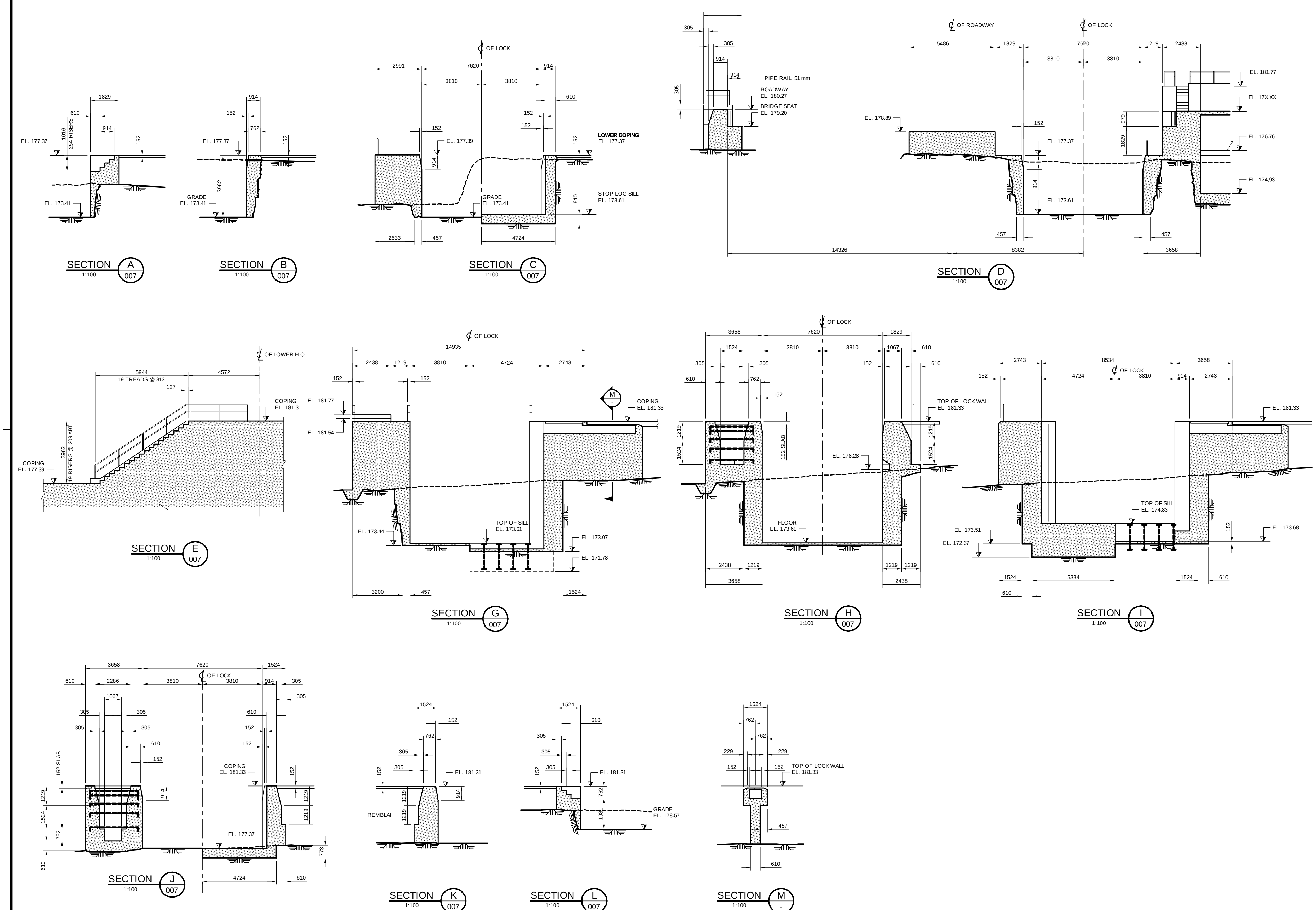

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LEGEND

CONCRETE

BEDROCK

No.	Description	Dwn.By Des.Par	Date
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PORT SEVERN DAMS
DAM SAFETY REVIEW

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LOCK 45 AND
UPSTREAM SHORELINE WALL
SECTIONS

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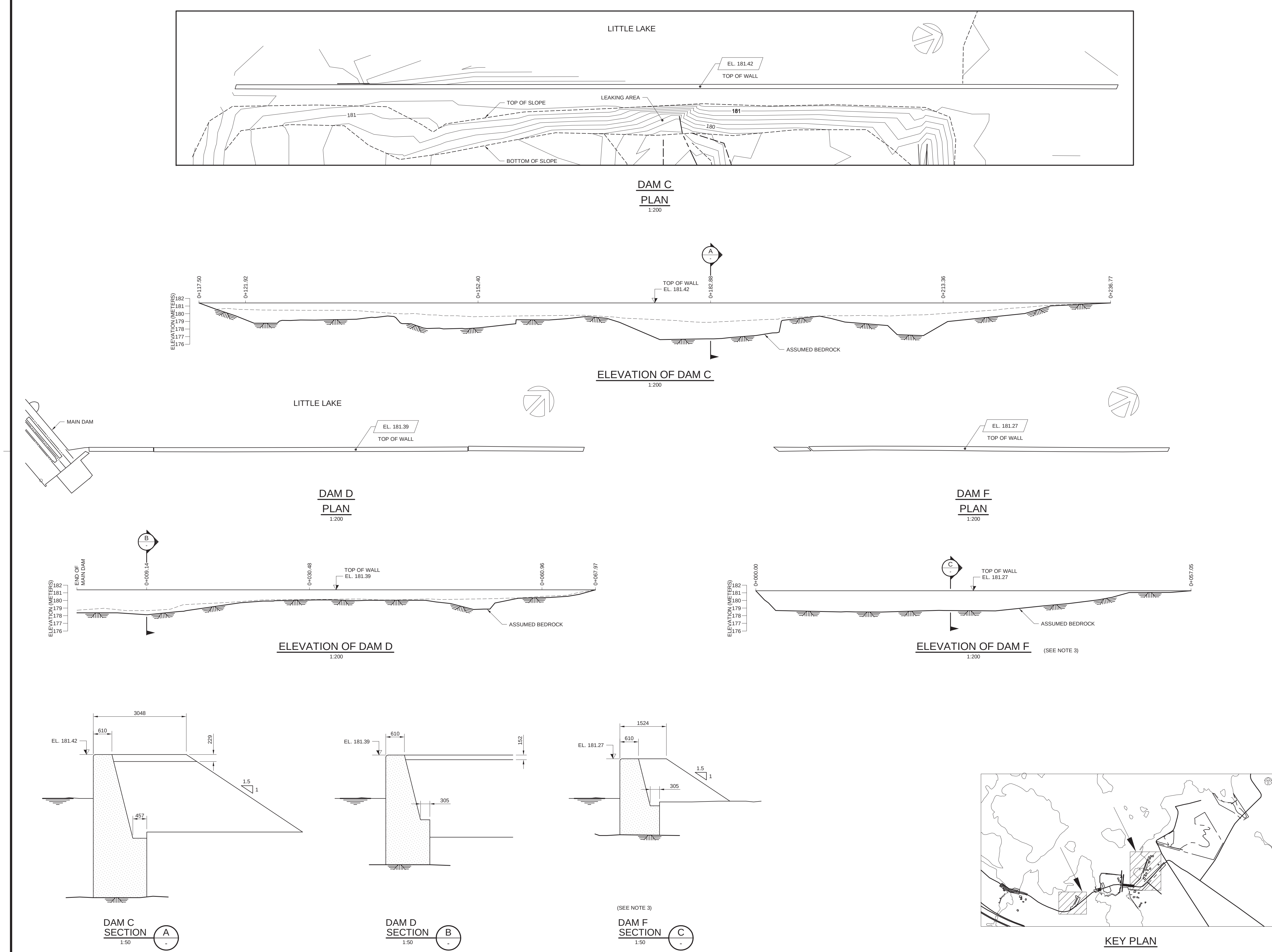

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3-DAM F IS NO LONGER REQUIRED DUE TO LAND RECLAMATION.

LEGEND

CONCRETE

BEDROCK

No.	Description	Dwn.By Des.Par	Date
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PORT SEVERN DAMS
DAM SAFETY REVIEW

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DAMS C, D, F
PLAN, ELEVATIONS
AND SECTIONS

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Project Number / Numéro du projet: 0522157-2000
Drawing Number / Numéro du Dessin: 010
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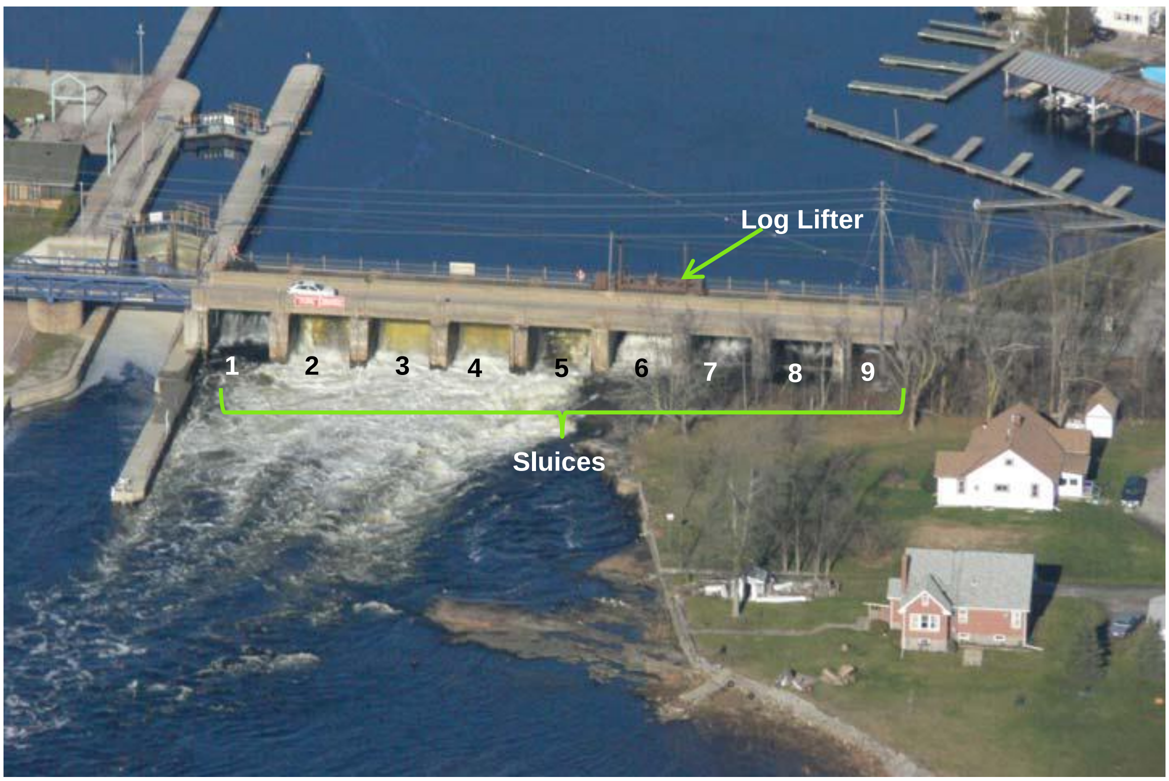
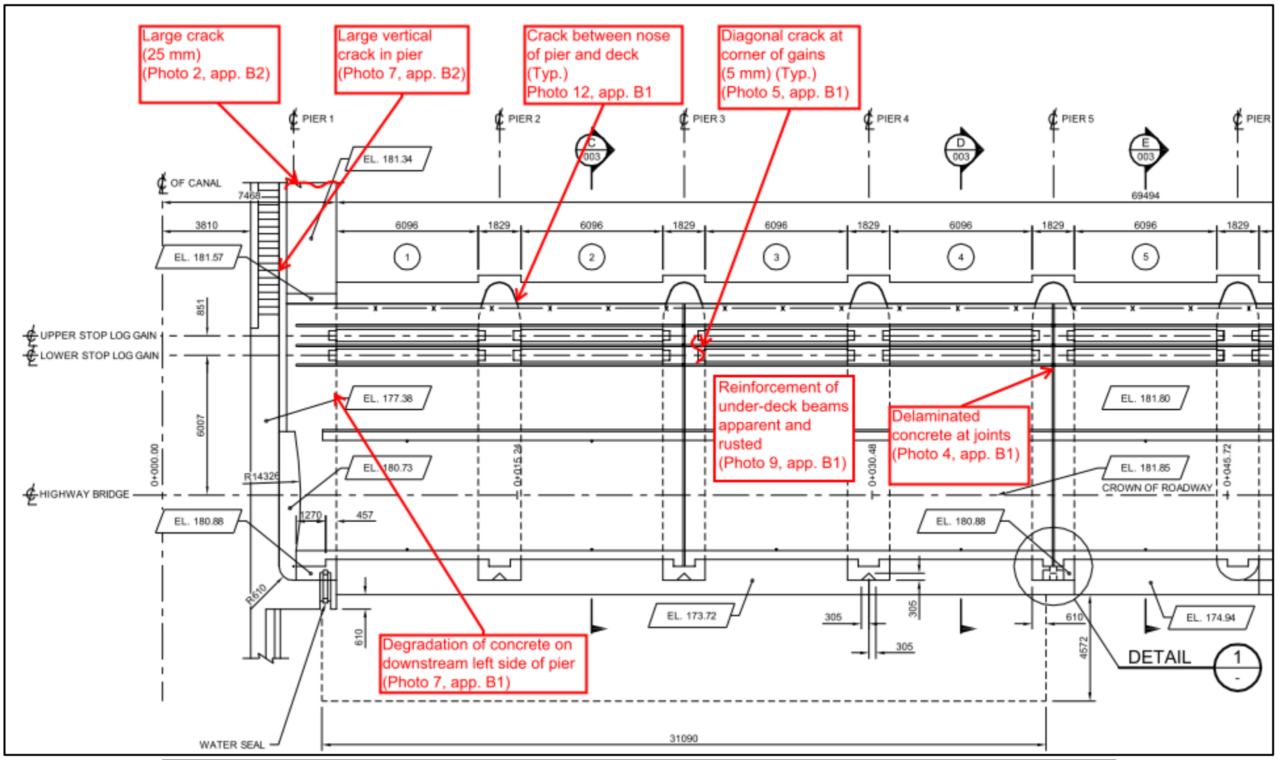
Appendix B
Site Photographs

- B1 – Main Dam
- B2 – Lock 45 and Upstream Shoreline Wall
- B3 – Dam D
- B4 – Reservoir Rim

Appendix B1
Pictures of Main Dam




Large crack
(25 mm)
(Photo 2, app. B2)
Large vertical
crack in pier
(Photo 7, app. B2)
Crack between nose
of pier and deck
(Typ.)
Photo 12, app. B1
Diagonal crack at
corner of gains
(5 mm) (Typ.)
(Photo 5, app. B1)
Reinforcement of
under-deck beams
apparent and
rusted
(Photo 9, app. B1)
Delaminated
concrete at joints
(Photo 4, app. B1)
Degradation of concrete on
downstream left side of pier
(Photo 7, app. B1)
PIER 1
PIER 2
PIER 3
PIER 4
PIER 5
OF CANAL
UPPER STOP LOG GAIN
LOWER STOP LOG GAIN
HIGHWAY BRIDGE
CROWN OF ROADWAY
WATER SEAL
DETAIL
EL. 181.34
EL. 181.57
EL. 177.38
EL. 180.73
EL. 180.88
EL. 173.72
EL. 181.80
EL. 181.85
EL. 174.94
31090

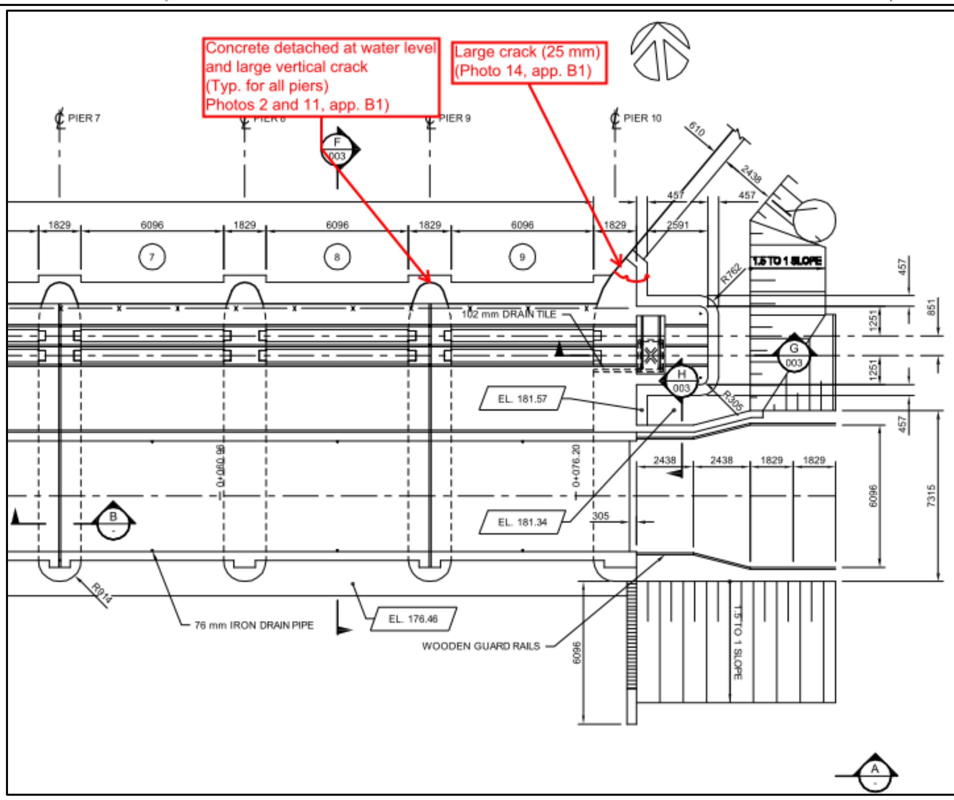

Concrete detached at water level
and large vertical crack
(Typ. for all piers)
Photos 2 and 11, app. B1)
Large crack (25 mm)
(Photo 14, app. B1)
PIER 7
PIER 9
PIER 10
1.5 TO 1 SLOPE
102 mm DRAIN TILE
EL. 181.57
EL. 181.34
EL. 176.46
76 mm IRON DRAIN PIPE
WOODEN GUARD RAILS

Photo 1: General view

Photo 2: Upstream view

Photo 3: Downstream view

Photo 4: Evidence of repair of construction joint at pier 5

Photo 5: Crack on the deck at sluice 3

Photo 6: Moss along the rails at sluice 6

Photo 7 : Damaged concrete on gains and downstream wall at sluice 1

Photo 8: Under-deck concrete in good condition at sluice 2, except for a few apparent stirrups

Photo 9: Apparent rusted rebar at sluice 3 under-deck

Photo 10: Under-deck concrete in good condition at sluice 7

Photo 11: Deteriorated concrete at the piers 7 and 8

Photo 12: Crack at the nose of pier 2

Photo 13: Crack at the nose of pier 7

Photo 14: Large crack on left bank abutment

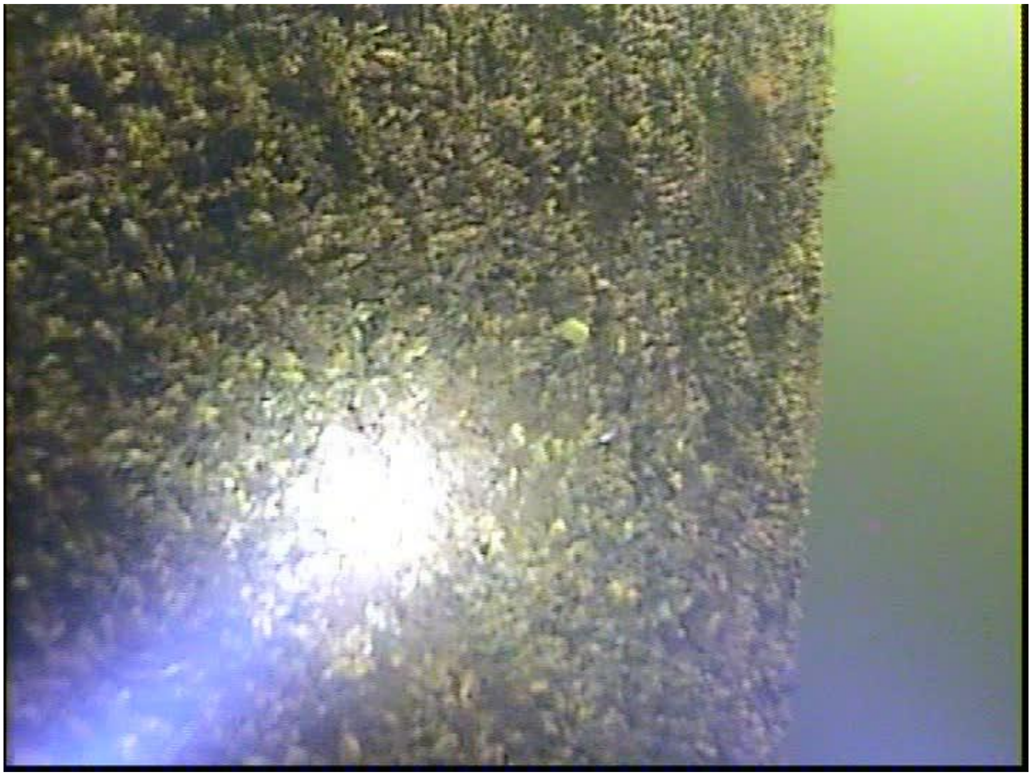


Photo 15: Zebra mussels on concrete surface

Photo 16: Left side of pier 2, with deteriorated concrete

Photo 17: Open construction joint on the left side of pier no. 2

Photo 18: Open construction joint near embedded parts of gains, pier 2

Photo 19: Concrete of pier 3 right face

Photo 20: Crest of downstream pier 3

Photo 21: Sluice 2 wooden stop logs

Photo 22: Wooden stop logs

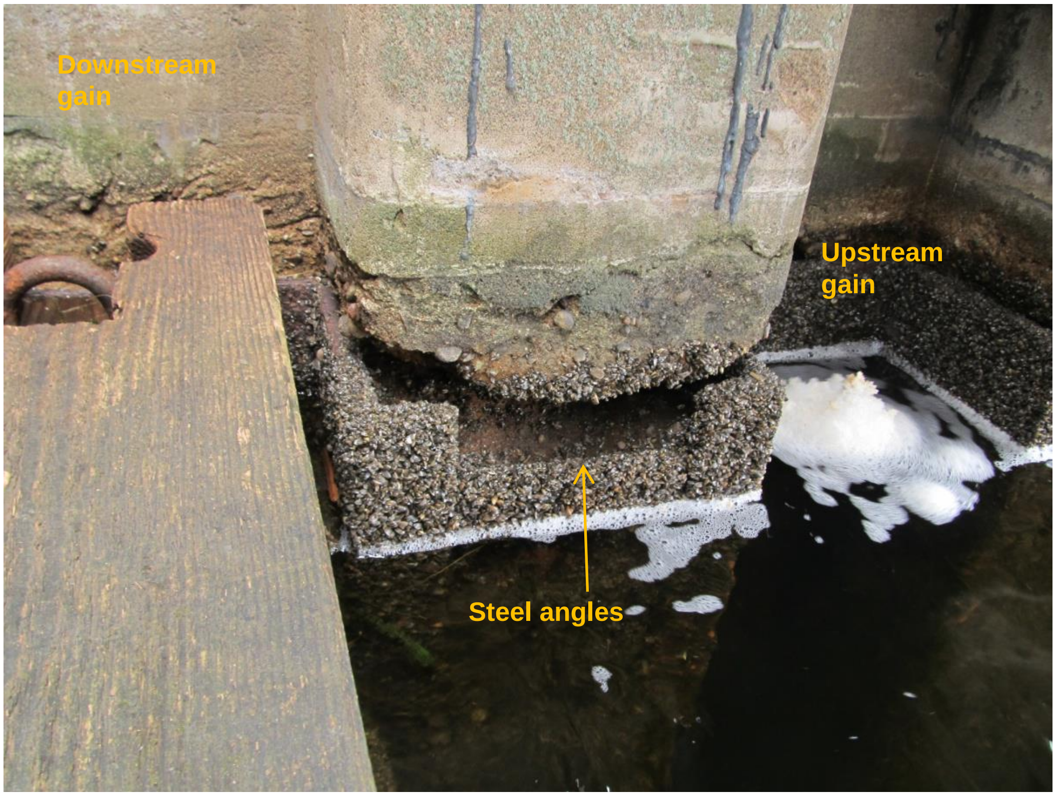


Photo 23: Extensively damaged concrete at junction with embedded steel angles covered in zebra mussels

Photo 24: Secondary concrete losing its integrity

Photo 25: Unlocked wooden gain covers (covers in good condition)

Photo 26: Hydraulic log lifter

Photo 27: Guided hook

Photo 28: Log lifter

Photo 29: Engine

Photo 30: Sideway adjusting rails in poor condition

Photo 31: Used rails and damaged concrete due to "jacking"

Photo 32: "Jacking" operation

Photo 33: Manual winches

Photo 34: Well-maintained deck in wintertime (Photo provided by PCA)

Photo 35: Problematic ice buildup (Photo provided by PCA)

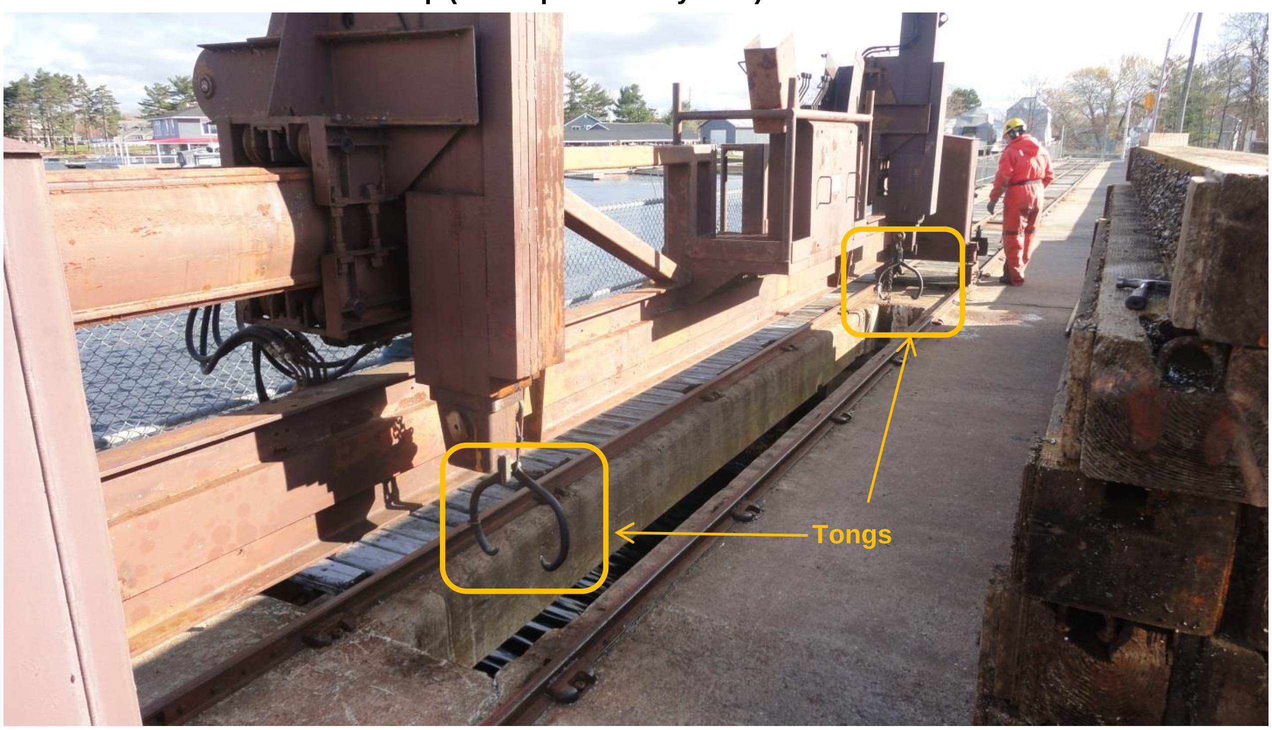


Photo 36: Use of tongs with log lifter

Photo 37: Manual use of tongs; operators not properly tied-off

Photo 38: Surface ground immediately downstream of sluices 7, 8 and 9

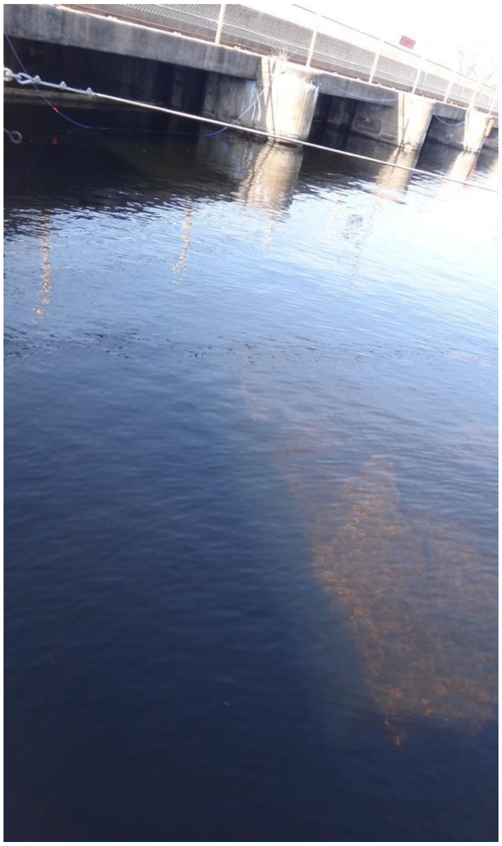


Photo 39: Presence of rocks or high river bed immediately upstream of sluice 9

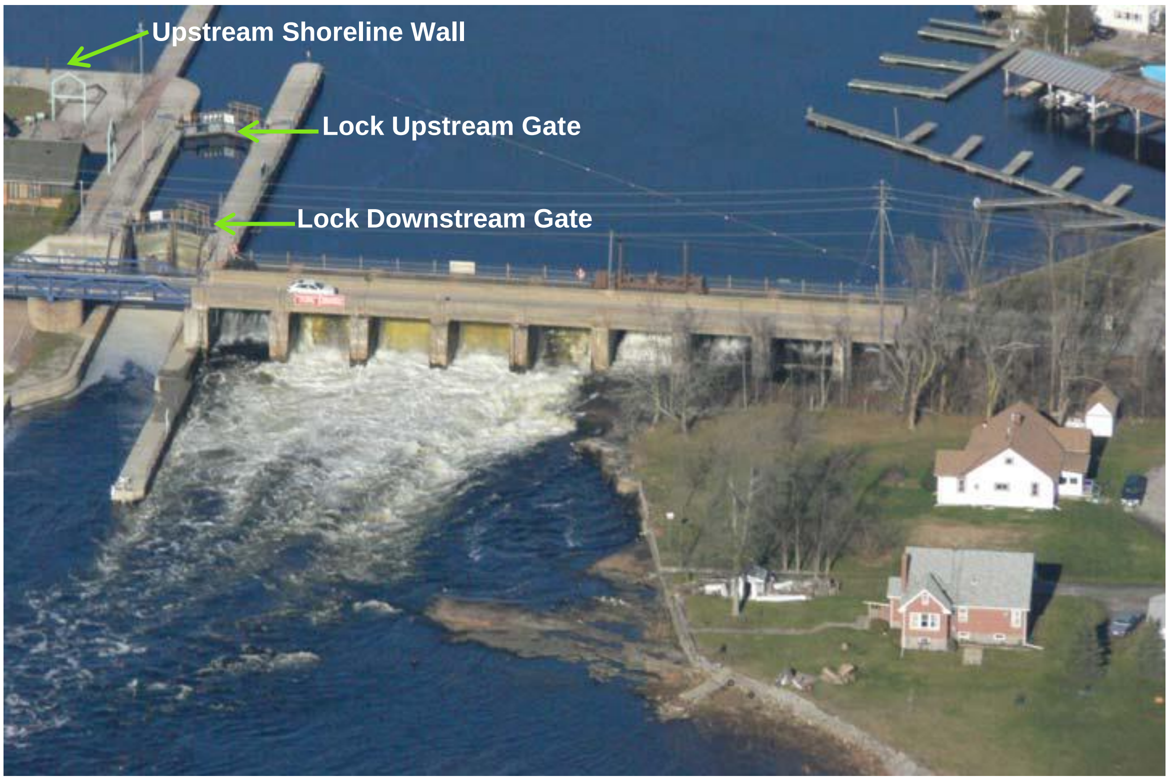
Photo 40: Leakage through a damaged stop log

Photo 41: Hydraulic log lifter and manual winch on the rails

Photo 42: Hydraulic log lifter hooked onto log dee

Photo 43: Use of tongs to remove a log with a broken dee

Appendix B2
Pictures of Lock 45 and Upstream Shoreline Wall



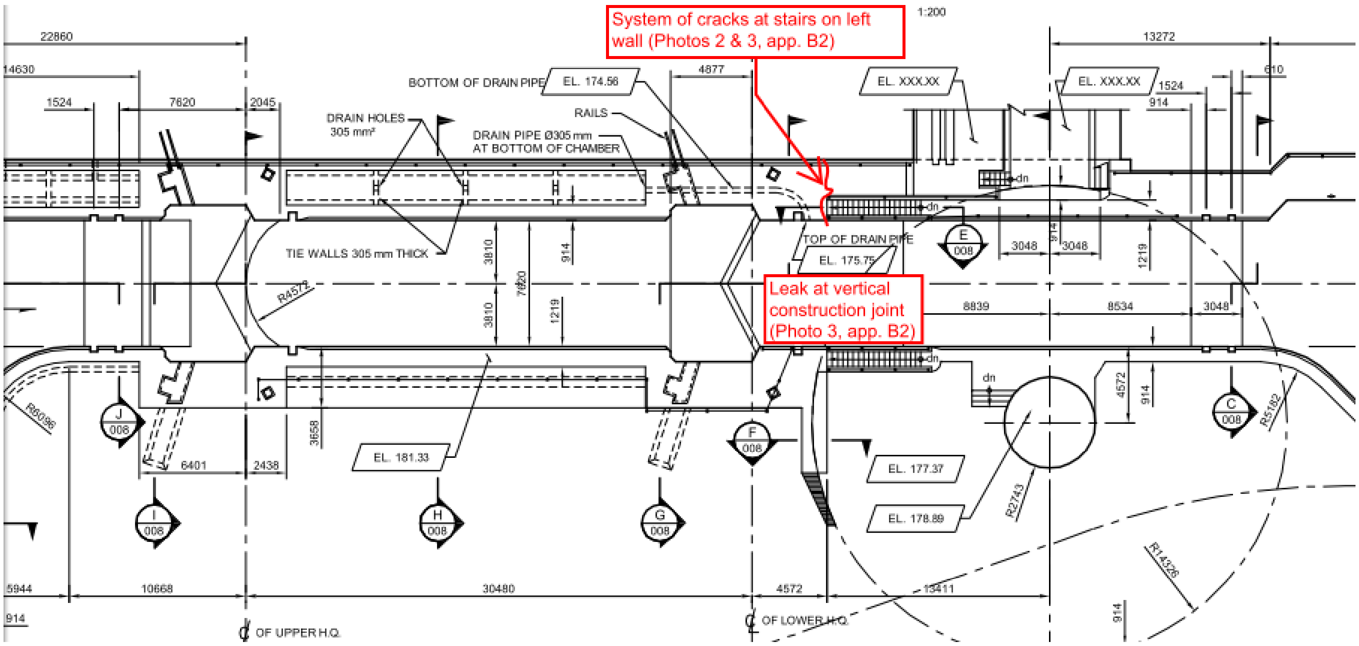

System of cracks at stairs on left wall (Photos 2 & 3, app. B2)
Leak at vertical construction joint (Photo 3, app. B2)
1:200
BOTTOM OF DRAIN PIPE
EL. 174.56
DRAIN HOLES
305 mm²
RAILS
DRAIN PIPE Ø305 mm
AT BOTTOM OF CHAMBER
TIE WALLS 305 mm THICK
TOP OF DRAIN PIPE
EL. 175.75
EL. XXX.XX
EL. 181.33
EL. 177.37
EL. 178.89
OF UPPER H.Q.
OF LOWER H.Q.

Photo 1: General view

Photo 2: Crack at the top of the stairs

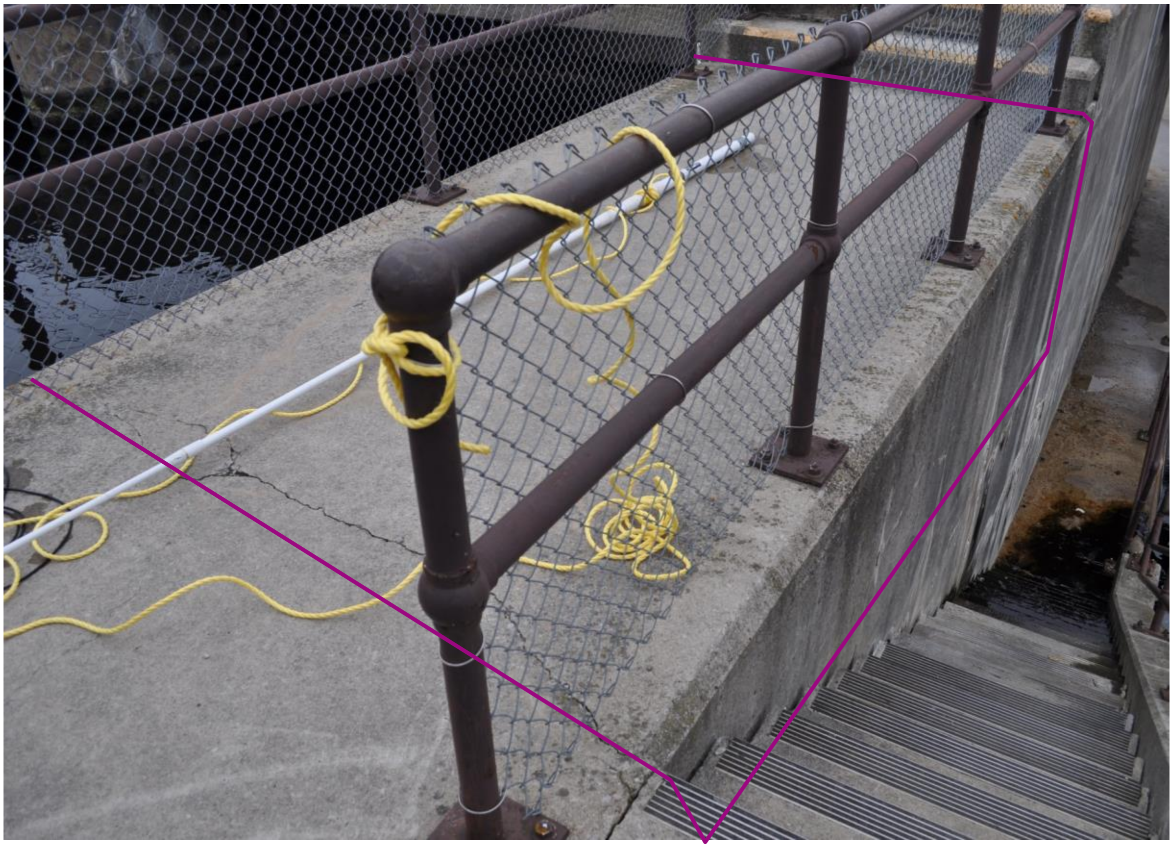


Photo 2a: System of cracks and joints at the top of the stairs

Photo 2b: System of cracks and joints at the top of the stairs

Photo 3: Left wall and leaking crack

Photo 4: Junction of horizontal and vertical open construction joints on dam side of left lock wall

Photo 5: Vertical crack under water (open construction joint) on dam side of left lock wall

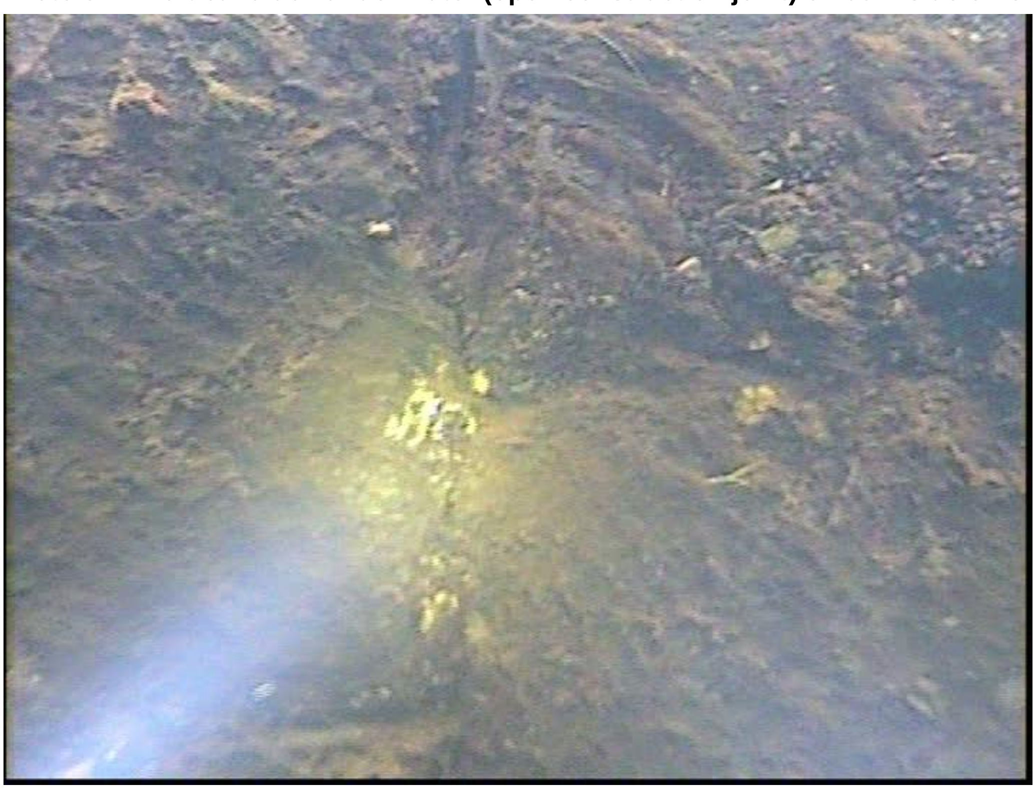


Photo 6: Vertical crack (open construction joint) on dam side of left lock wall

Photo 7: Large vertical crack at junction with Main Dam on reservoir side

Photo 8: Caulked vertical crack at slab level

Photo 9: Vertical crack at water level on reservoir side (open construction joint)

Photo 10: Open horizontal open construction joint dam side of left lock wall

Photo 11: Right wall and handrails

Photo 12: Beginning of Upstream Shoreline Wall and added pier at junction with lock canal

Photo 13: Left portion of wall with boarding dock

Photo 14: Left portion of wall

Photo 15: Right portion of wall (toward Bay View)

Photo 16: Right portion of wall (view toward lock canal)

Photo 17: Right end of wall

Photo 18: Old and additional walkways

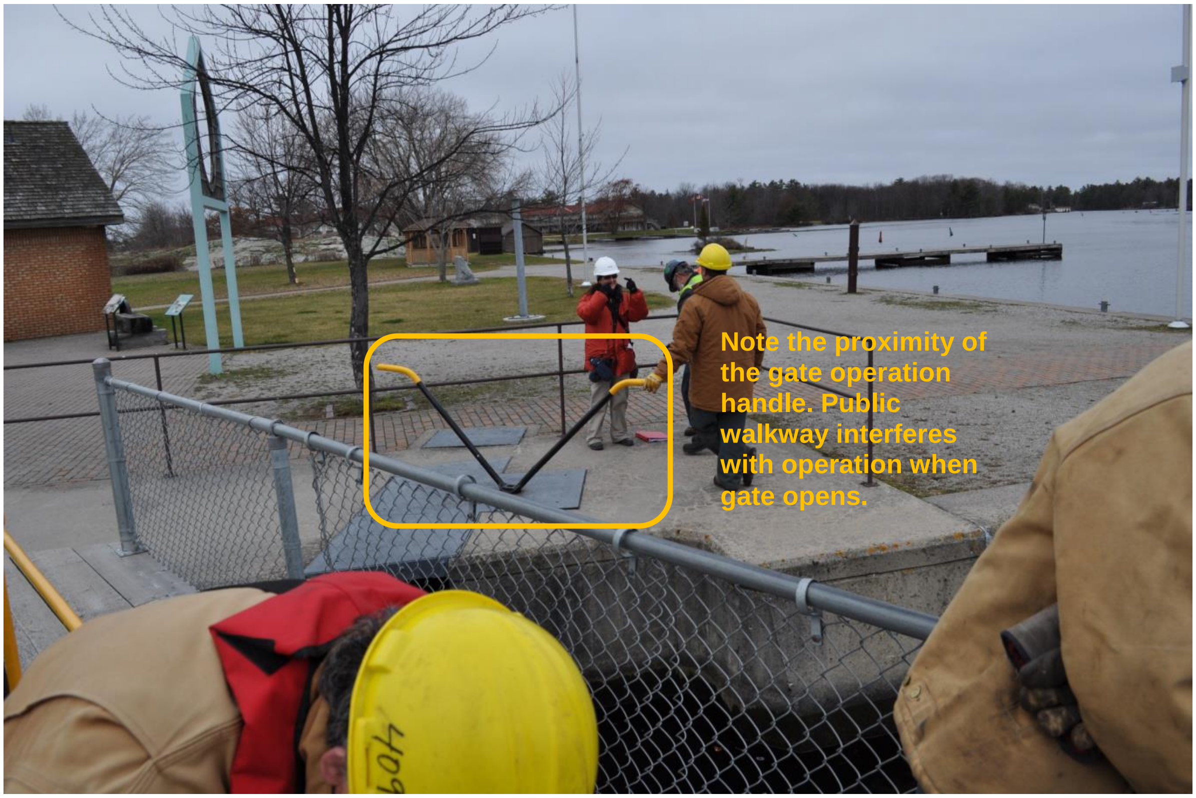


Photo 19: Upstream lock gate

Photo 20: Downstream lock gate

Photo 21: Downstream lock gate, left arm bent

Photo 22: Filled sinkholes near the upstream shoreline wall

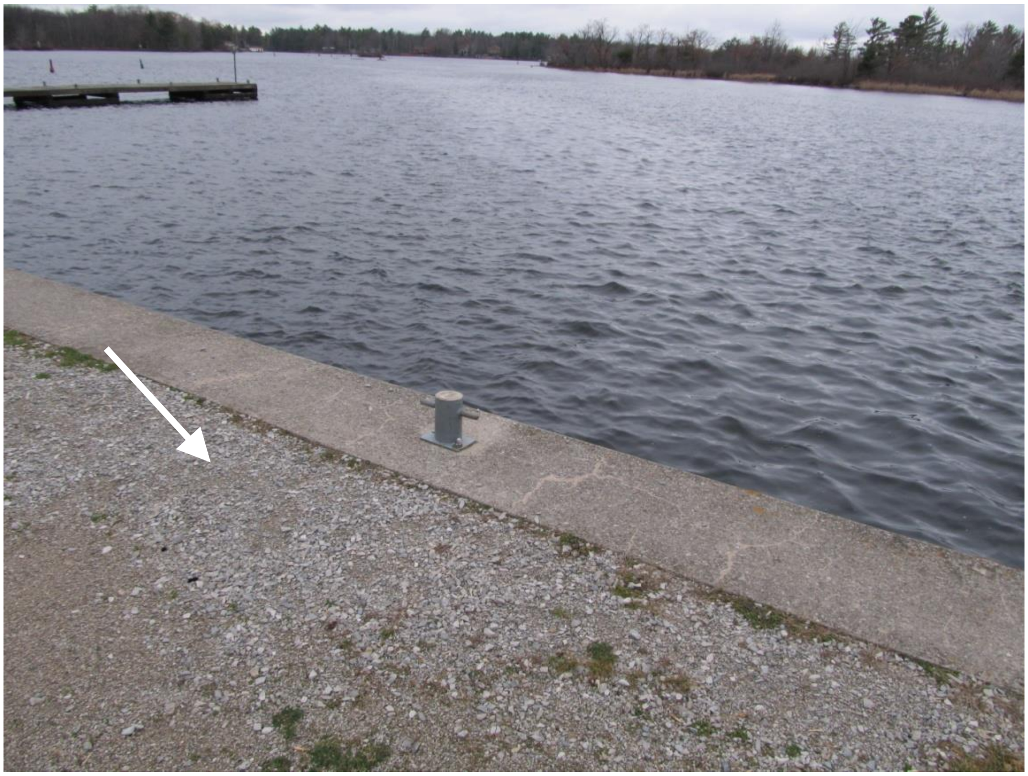
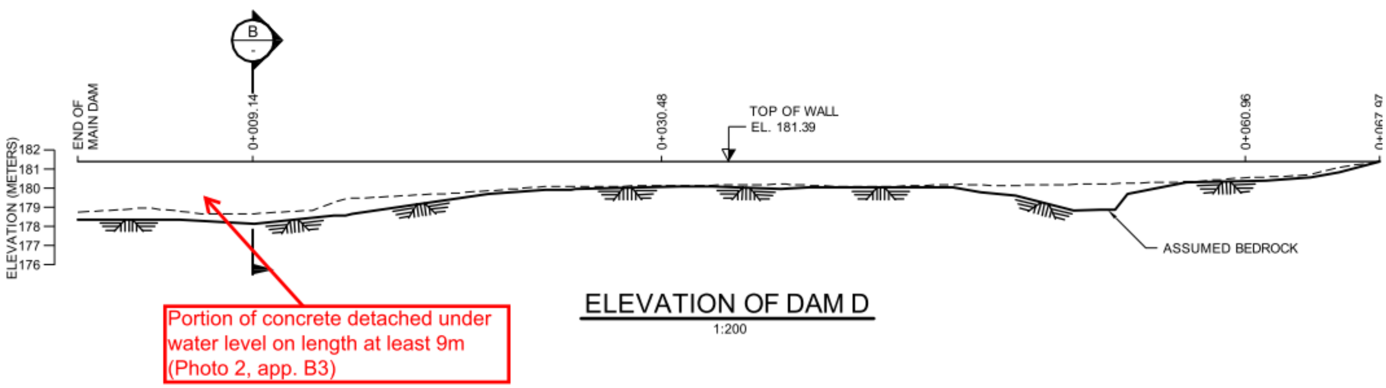
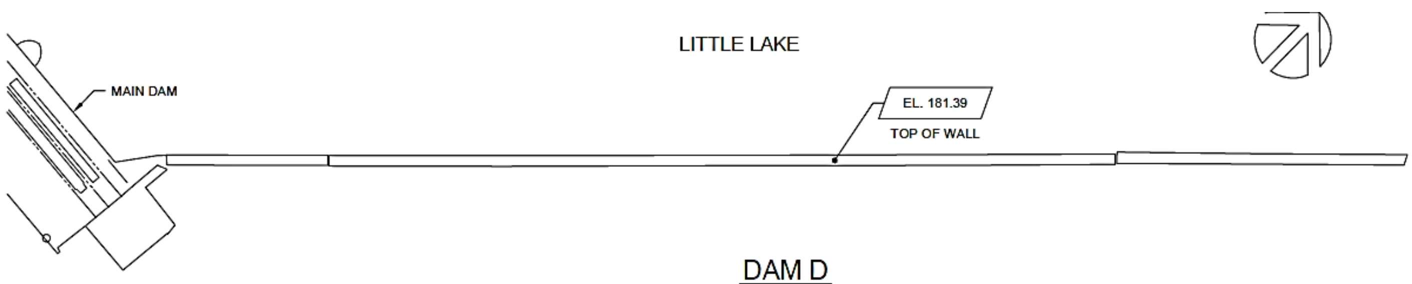


Photo 23: Filled sinkholes near the upstream shoreline wall

Appendix B3
Pictures of Dam D



ELEVATION OF DAM D
1:200



DAM D
PLAN
1:200

Photo 1: Minor crack on wall

Photo 2: Detached pieces of concrete in the drawdown zone

Photo 3: Large cracks at junction with Main Dam

Photo 4: Degradation of submerged concrete

Photo 5: Presence of algae on submerged concrete

Photo 6: Drainage pipe at the base of dam

Photo 7: Small cracks and minor efflorescence on concrete

Photo 8: Sinkhole #1 observed near Dam D

Photo 9: Sinkhole #1 observed near Dam D

Photo 10: Sinkhole #2 observed near Dam D

Photo 11: Upstream view of concrete deterioration of Dam D

Photo 12: Upstream view of concrete deterioration of Dam D

Appendix B4
Pictures of the Reservoir Rim

Photo 1 Culvert 3 and device to circulate flow

Photo 2 Culvert 4

Photo 3 **View from Culvert 5 looking north-west toward Highway 400**

Photo 4 **Culvert 5**

Photo 5 **Culvert 1 looking north-west**

Appendix C
Severn River Watershed

C1 – General Location Plan

C2 – SSARR Flow Chart

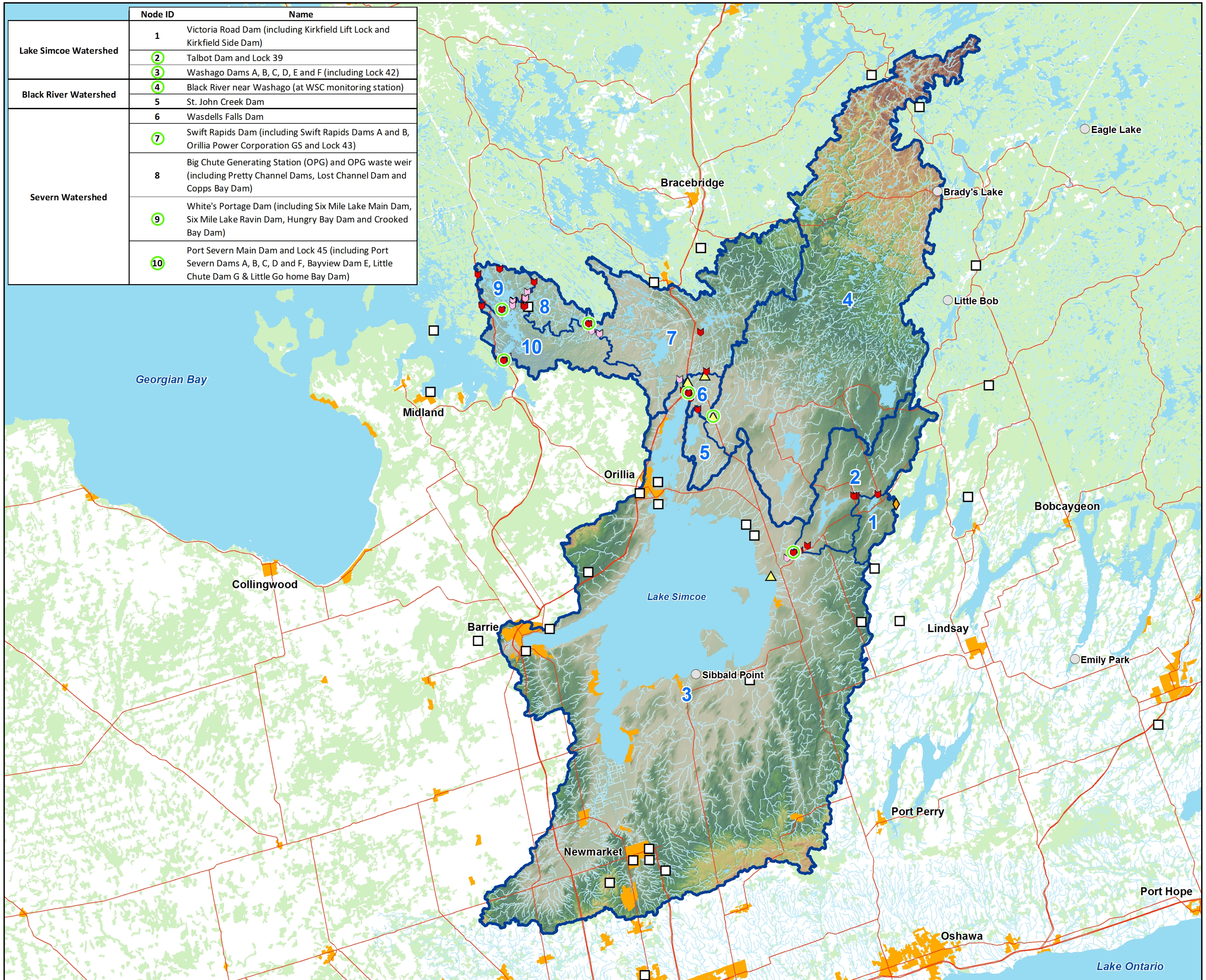
Travaux publics et Services gouvernementaux Canada

Public Works and Government Services Canada

Parcs Canada

Parks Canada

SEVERN RIVER WATERSHED HYDRO-TECHNICAL STUDY



	Node ID	Name
Lake Simcoe Watershed	1	Victoria Road Dam (including Kirkfield Lift Lock and Kirkfield Side Dam)
	2	Talbot Dam and Lock 39
	3	Washago Dams A, B, C, D, E and F (including Lock 42)
Black River Watershed	4	Black River near Washago (at WSC monitoring station)
	5	St. John Creek Dam
Severn Watershed	6	Wasdells Falls Dam
	7	Swift Rapids Dam (including Swift Rapids Dams A and B, Orillia Power Corporation GS and Lock 43)
	8	Big Chute Generating Station (OPG) and OPG waste weir (including Pretty Channel Dams, Lost Channel Dam and Copps Bay Dam)
	9	White's Portage Dam (including Six Mile Lake Main Dam, Six Mile Lake Ravin Dam, Hungry Bay Dam and Crooked Bay Dam)
	10	Port Severn Main Dam and Lock 45 (including Port Severn Dams A, B, C, D and F, Bayview Dam E, Little Chute Dam G & Little Go home Bay Dam)

Regulatory Dam

Non-Regulatory Dam / Lock

Guard Dam

Study Node

Hydrometric Station

Climatological Station

Snow Station

Sub-basin

Watershed

Terrain Elevation

High : 462 m

Low : 174 m

0 10 20 30 km

1 : 650 000

UTM 17
NAD 83 Datum

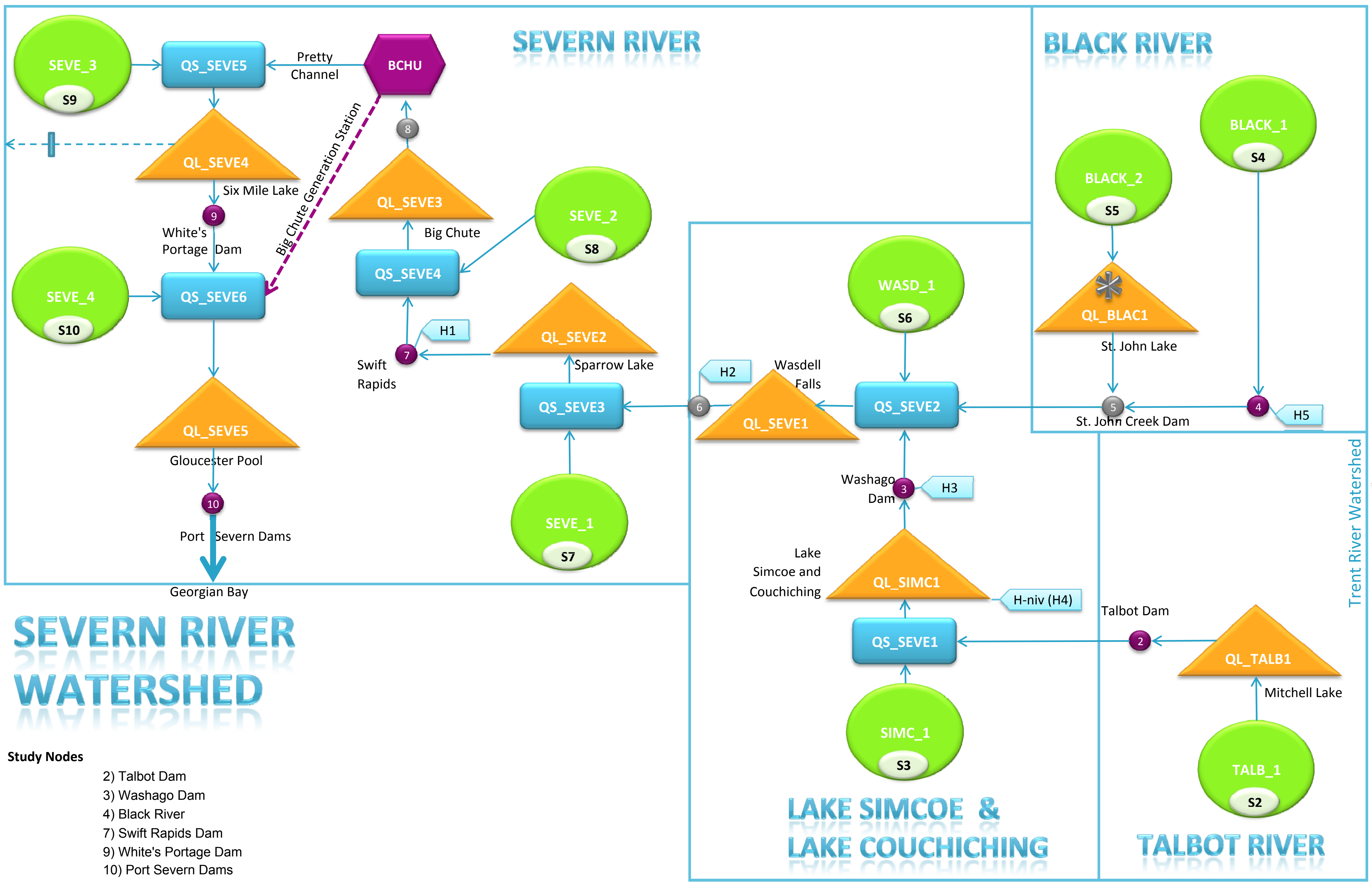
GENERAL LOCATION PLAN

January 2012 *Project: 0522157-1000*

AECOM

Travaux publics et Services gouvernementaux Canada
Public Works and Government Services Canada
Parcs Canada
Parks Canada

SEVERN RIVER WATERSHED
HYDRO-TECHNICAL STUDY

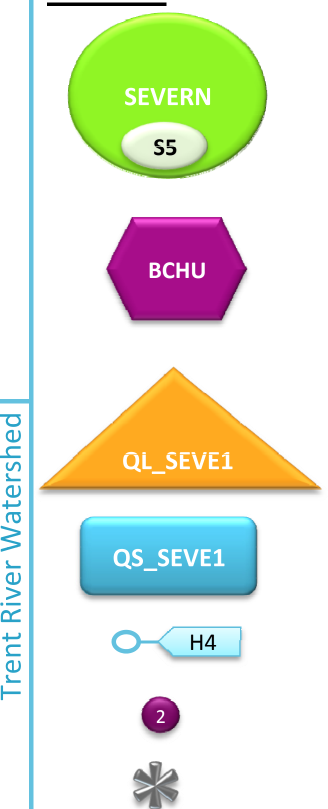


SEVERN RIVER WATERSHED

Study Nodes

2) Talbot Dam
3) Washago Dam
4) Black River
7) Swift Rapids Dam
9) White's Portage Dam
10) Port Severn Dams

LEGEND



Sub-Basin
Diversion
Reservoir Regulation
Junction Node (Sum of Hydrographs)
Hydrometric Station
Study Node
Backwater effect
During high peak flows, the Black River tends to back up into St John Creek as a result of overtopping the existing control structure and thus reverses the typical flow direction.

Figure A2 - SSARR Flow Chart

January 2012

Appendix D
Dam Classification

D1 – Calculations for People at Risk (PAR) at Port Severn

D2 – Flood Hydrographs

Appendix D1
Calculations for People at Risk (PAR) at Port Severn

Appendix D - Calculations for People at Risk (PAR) at Port Severn PAR Buffer = 0.5 m (No Evac) Significanant Wave = 0.1 m

REVISED (from DEM 177.24)

1 = Evacuated

1 = Significant Wave from Dam Break

Number = Type of Building

					Dam Break									Evacuated					Significant					PAR				
OBJECTID	Type	ZONE	ID	ELEV	SD_WB	100_NB	100_WB	1K_NB	1K_WB	10K_N	10K_W	PMF_N	PMF_WB	SD	100	1K	10K	PMF	SD	100	1K	10K	PMF	SD	100	1K	10K	PMF
0	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
1	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
2	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
3	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
4	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
5	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
6	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
7	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
8	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
9	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
10	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
11	Boatshed	5	1	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
12	Commercial	2	2	180.5419	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
13	Commercial	2	2	178.1413	0.00	0.00	0.00	0.00	0.00	0.04	0.33	0.53	0.90	0	0	0	0	1	0	0	0	1	1				2	
14	Commercial	2	2	180.5321	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
15	Commercial	2	2	181.9006	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
16	Commercial	2	2	180.6844	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
17	Commercial	4	2	178.4124	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
18	Commercial	2	2	181.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
19	Commercial	2	2	180.6250	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
20	Commercial	2	2	999.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
21	Commercial	2	2	179.5309	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
22	Commercial	2	2	181.4000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
23	Commercial	2	2	181.4000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
24	Commercial	2	2	999.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
25	Commercial	5	2	179.1045	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
26	Commercial	4	2	179.8775	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
27	Commercial	5	2	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
28	Vacant	1	3	178.0948	0.00	0.00	0.00	0.00	0.00	0.12	0.41	0.65	1.03	0	0	0	0	1	0	0	0	1	1				3	
29	Vacant	4	3	182.3802	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
30	Mixed	2	4	179.0716	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
31	Mixed	2	4	179.7145	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
32	Mixed	2	4	178.7162	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.32	0	0	0	0	0	0	0	0	0	1					4
33	Mixed	5	4	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
34	Houses	5	5	178.5072	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
35	Houses	5	5	178.3110	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
36	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
37	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
38	Houses	5	5	177.6613	0.00	0.00	0.00	0.00	0.00	0.16	0.18	0.19	0.21	0	0	0	0	0	0	0	0	0	0					
39	Houses	3	5	181.2495	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
40	Houses	3	5	181.5762	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
41	Houses	1	5	177.4906	0.00	0.00	0.04	0.27	0.46	0.72	1.01	1.25	1.63	0	0	0	1	1	0	0	1	1	1			5		
42	Houses	3	5	179.2307	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
43	Houses	2	5	181.7981	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
44	Houses	1	5	177.8105	0.00	0.00	0.00	0.00	0.14	0.40	0.69	0.93	1.31	0	0	0	0	1	0	0	1	1	1			5	5	
45	Houses	4	5	182.7573	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
46	Houses	2	5	178.4250	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.24	0.61	0	0	0	0	0	0	0	0	0	1					5
47	Houses	3	5	182.0571	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
48	Houses	3	5	178.9501	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					

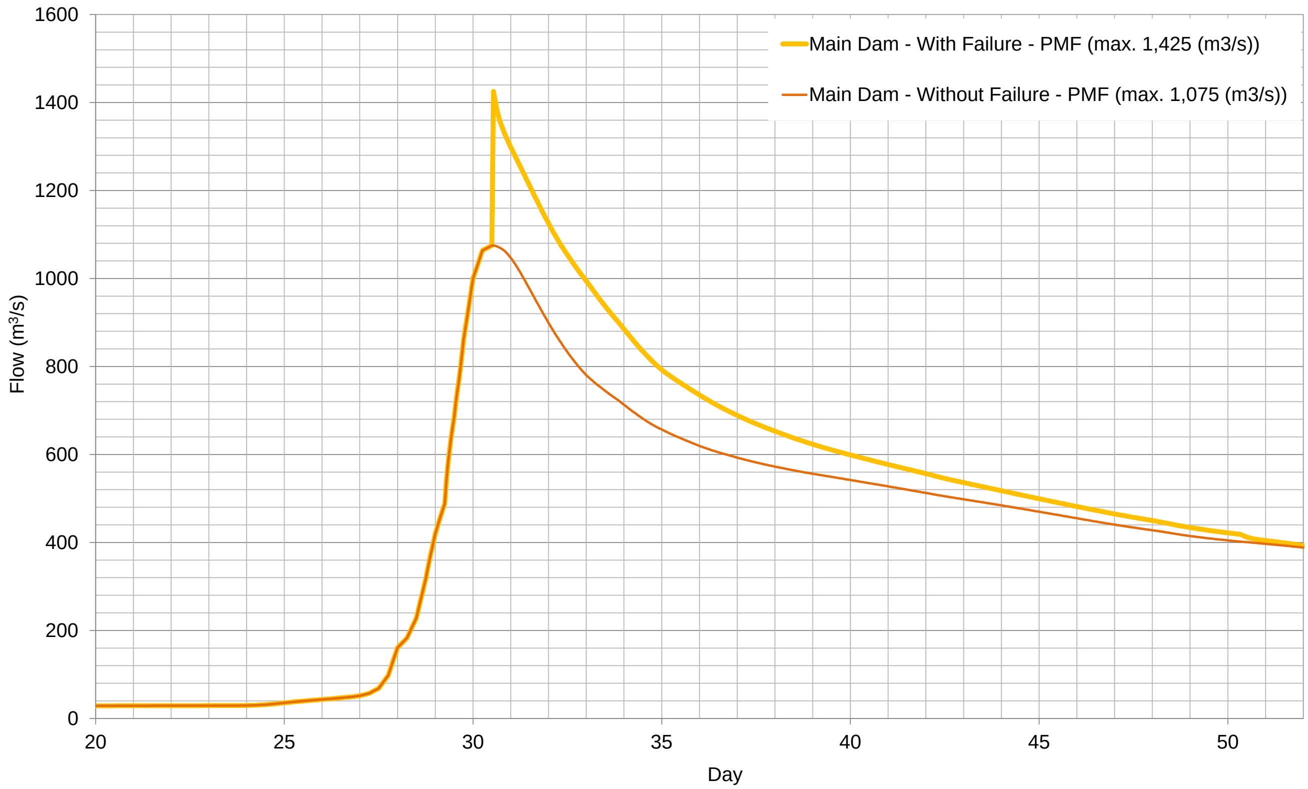
49	Houses	1	5	178.5285	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.21	0.59	0	0	0	0	0	0	0	0	0	1					5
50	Houses	1	5	179.2086	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
51	Houses	4	5	182.5960	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
52	Houses	1	5	178.9381	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.18	0	0	0	0	0	0	0	0	0	1					5
53	Houses	2	5	178.7121	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.33	0	0	0	0	0	0	0	0	0	1					5
54	Houses	2	5	180.0877	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
55	Houses	3	5	177.9045	0.00	0.00	0.00	0.00	0.00	0.23	0.50	0.69	1.04	0	0	0	0	1	0	0	0	1	1				5	
56	Houses	3	5	179.7092	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
57	Houses	2	5	178.6010	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.44	0	0	0	0	0	0	0	0	0	1					5
58	Houses	2	5	177.7952	0.00	0.00	0.00	0.00	0.14	0.38	0.67	0.87	1.24	0	0	0	0	1	0	0	1	1	1			5	5	
59	Houses	4	5	177.4575	0.00	0.00	0.00	0.08	0.11	0.40	0.44	0.47	0.54	0	0	0	0	0	0	0	0	0	0					
60	Houses	1	5	178.2085	0.00	0.00	0.00	0.00	0.00	0.00	0.29	0.53	0.91	0	0	0	0	1	0	0	0	1	1				5	
61	Houses	2	5	180.2881	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
62	Houses	1	5	179.2780	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
63	Houses	4	5	177.2400	0.00	0.02	0.04	0.30	0.33	0.62	0.66	0.69	0.76	0	0	0	1	1	0	0	0	0	0					
64	Houses	4	5	177.2400	0.00	0.02	0.04	0.30	0.33	0.62	0.66	0.69	0.76	0	0	0	1	1	0	0	0	0	0					
65	Houses	2	5	181.0531	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
66	Houses	4	5	179.9850	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
67	Houses	3	5	182.4536	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
68	Houses	2	5	177.7617	0.00	0.00	0.00	0.00	0.18	0.42	0.71	0.91	1.28	0	0	0	0	1	0	0	1	1	1			5	5	
69	Houses	4	5	181.0439	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
70	Houses	2	5	178.8629	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.18	0	0	0	0	0	0	0	0	0	1					5
71	Houses	5	5	180.7663	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
72	Houses	3	5	182.1221	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
73	Houses	3	5	178.3943	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.20	0.55	0	0	0	0	0	0	0	0	0	1					5
74	Houses	5	5	178.0197	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
75	Houses	2	5	181.0706	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
76	Houses	2	5	181.3492	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
77	Houses	5	5	177.2746	0.00	0.00	0.00	0.25	0.26	0.55	0.57	0.58	0.60	0	0	0	1	1	0	0	0	0	0					
78	Houses	2	5	180.2013	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
79	Houses	2	5	181.3000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
80	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
81	Houses	2	5	179.5668	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
82	Houses	4	5	177.2400	0.00	0.02	0.04	0.30	0.33	0.62	0.66	0.69	0.76	0	0	0	1	1	0	0	0	0	0					
83	Houses	5	5	177.6487	0.00	0.00	0.00	0.00	0.00	0.17	0.19	0.20	0.22	0	0	0	0	0	0	0	0	0	0					
84	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
85	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
86	Houses	2	5	178.0216	0.00	0.00	0.00	0.00	0.00	0.16	0.45	0.65	1.02	0	0	0	0	1	0	0	0	1	1				5	
87	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
88	Houses	5	5	177.5837	0.00	0.00	0.00	0.00	0.00	0.24	0.26	0.27	0.29	0	0	0	0	0	0	0	0	0	0					
89	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
90	Houses	5	5	179.6059	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
91	Houses	5	5	178.3936	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
92	Houses	5	5	179.4324	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
93	Houses	5	5	178.1108	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
94	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
95	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
96	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
97	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
98	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
99	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
100	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
101	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					

102	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
103	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
104	Houses	5	5	177.6452	0.00	0.00	0.00	0.00	0.00	0.17	0.19	0.20	0.22	0	0	0	0	0	0	0	0	0	0					
105	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
106	Houses	5	5	180.4290	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
107	Houses	5	5	178.8790	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
108	Houses	5	5	178.3300	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
109	Houses	5	5	178.1885	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
110	Houses	5	5	177.3559	0.00	0.00	0.00	0.16	0.17	0.46	0.48	0.49	0.51	0	0	0	0	0	0	0	0	0	0					
111	Houses	5	5	177.7060	0.00	0.00	0.00	0.00	0.00	0.11	0.13	0.14	0.16	0	0	0	0	0	0	0	0	0	0					
112	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
113	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
114	Houses	5	5	178.8709	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
115	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
116	Houses	5	5	177.4820	0.00	0.00	0.00	0.04	0.05	0.34	0.36	0.37	0.39	0	0	0	0	0	0	0	0	0	0					
117	Houses	5	5	178.6985	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
118	Houses	5	5	180.1869	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0					
119	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
120	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
121	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
122	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
123	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
124	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
125	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
126	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
127	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
128	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
129	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
130	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
131	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
132	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
133	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
134	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
135	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
136	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
137	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
138	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
139	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
140	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
141	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
142	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
143	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
144	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
145	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
146	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
147	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
148	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
149	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
150	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
151	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
152	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
153	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					
154	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0					

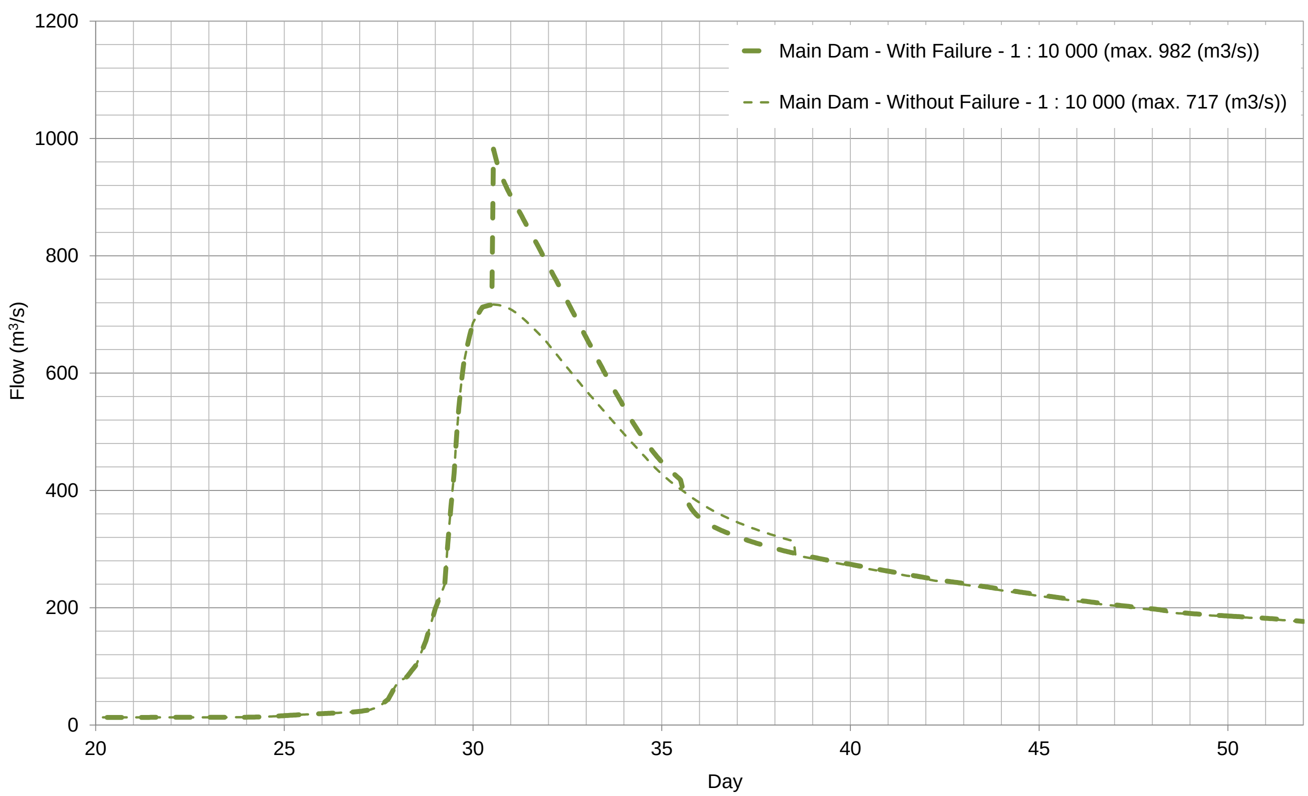
155	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
156	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
157	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
158	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
159	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
160	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
161	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
162	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
163	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
164	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
165	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
166	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
167	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
168	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
169	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
170	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
171	Houses	5	5	177.6185	0.00	0.00	0.00	0.00	0.00	0.20	0.22	0.23	0.25	0	0	0	0	0	0	0	0	0	0
172	Houses	5	5	177.5357	0.00	0.00	0.00	0.00	0.00	0.28	0.30	0.31	0.33	0	0	0	0	0	0	0	0	0	0
173	Houses	5	5	177.7048	0.00	0.00	0.00	0.00	0.00	0.12	0.14	0.15	0.17	0	0	0	0	0	0	0	0	0	0
174	Houses	5	5	177.7878	0.00	0.00	0.00	0.00	0.00	0.03	0.05	0.06	0.08	0	0	0	0	0	0	0	0	0	0
175	Houses	5	5	177.7567	0.00	0.00	0.00	0.00	0.00	0.06	0.08	0.09	0.11	0	0	0	0	0	0	0	0	0	0
176	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
177	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
178	Houses	5	5	179.6539	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
179	Houses	5	5	177.9851	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
180	Houses	5	5	178.0323	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
181	Houses	5	5	177.7098	0.00	0.00	0.00	0.00	0.00	0.11	0.13	0.14	0.16	0	0	0	0	0	0	0	0	0	0
182	Houses	5	5	177.4860	0.00	0.00	0.00	0.03	0.04	0.33	0.35	0.36	0.38	0	0	0	0	0	0	0	0	0	0
183	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
184	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
185	Houses	5	5	177.2620	0.00	0.00	0.00	0.26	0.27	0.56	0.58	0.59	0.61	0	0	0	1	1	0	0	0	0	0
186	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
187	Houses	5	5	177.3826	0.00	0.00	0.00	0.14	0.15	0.44	0.46	0.47	0.49	0	0	0	0	0	0	0	0	0	0
188	Houses	5	5	179.4254	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
189	Houses	5	5	178.7851	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
190	Houses	5	5	178.3706	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
191	Houses	5	5	182.0295	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0	0	0	0	0	0	0	0	0	0
192	Houses	5	5	177.2400	0.00	0.00	0.01	0.28	0.29	0.58	0.60	0.61	0.63	0	0	0	1	1	0	0	0	0	0
193	Houses	4	5	177.6900	0.00	0.00	0.00	0.00	0.00	0.17	0.21	0.24	0.31	0	0	0	0	0	0	0	0	0	0
*Note: House in zone 0 have elevation of 999 to indicate that they are above the dam and not included in the calculations																							
					0	3	97	104	107	124	126	128	132										

T	0	0	4	8	8
1 Boatshed	0	0	0	0	0
2 Commercial	0	0	0	1	0
3 Vacant	0	0	0	1	0
4 Mixed	0	0	0	0	1
5 Houses	0	0	4	6	7
C	0	0	4	8	8
	Y	Y	Y	Y	Y

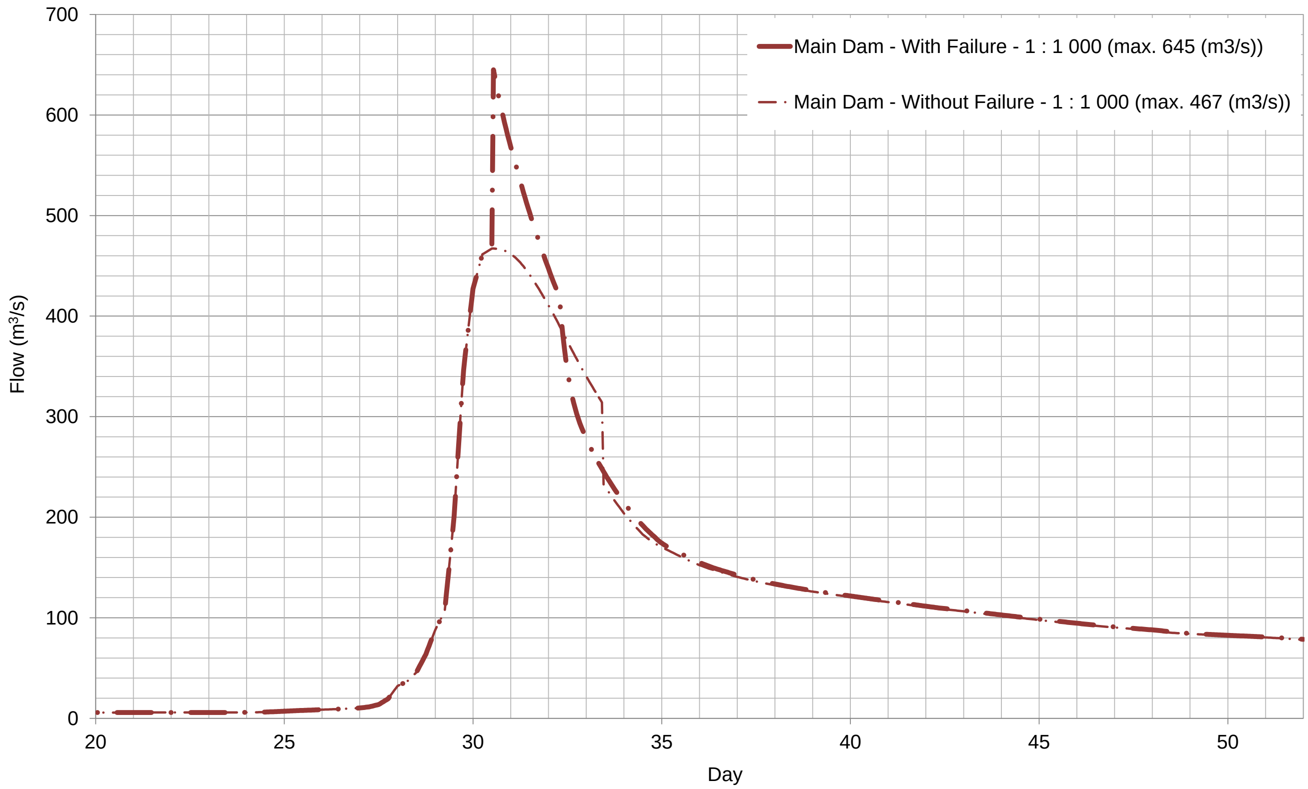
Appendix D2
Flood Hydrographs



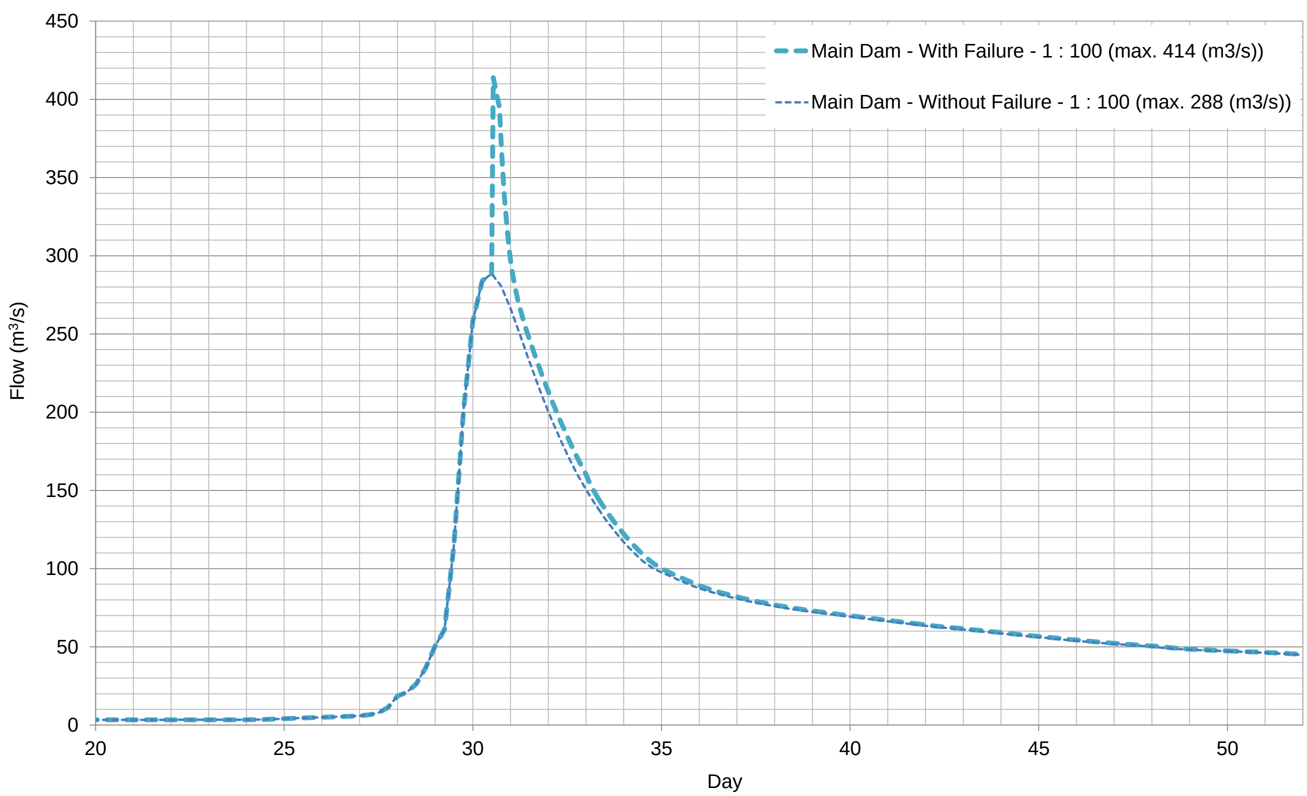
D 1 Main Dam – PMF Flood Hydrograph



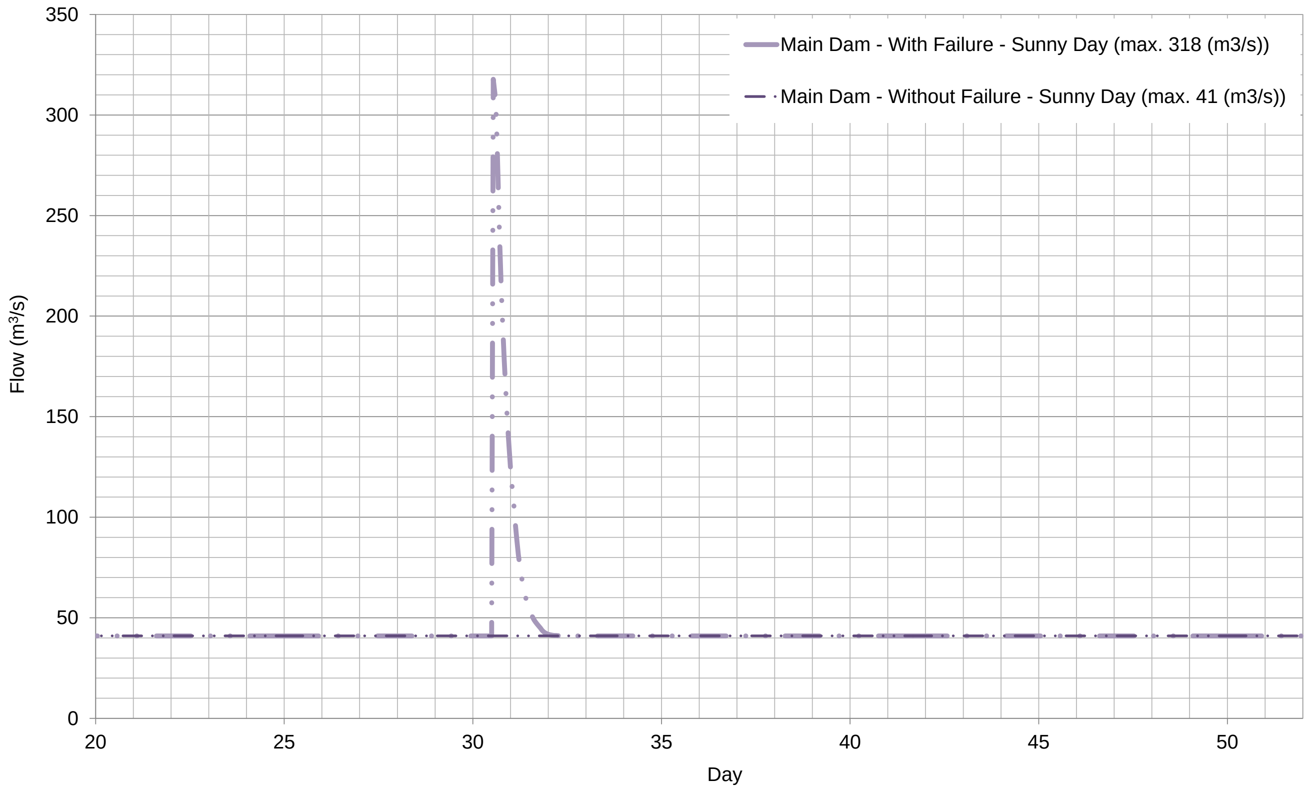
D 2 Main Dam – 1 :10 000 Year Flood Hydrograph



D 3 Main Dam – 1 :1 000 Year Flood Hydrograph



D 4 Main Dam – 1 :100 Year Flood Hydrograph



D 5 Main Dam – Sunny Day Break Hydrograph

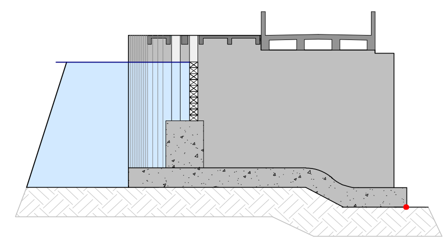
Appendix E
Dam and Lock Stability Analysis

- E1 – Loading Combinations Results
- E2 – Loading Combinations Sketches

Appendix E1
Loading Combinations Results

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Safety Factors (SF): Uplift	Safety Factors (SF): Peak Sliding: C = 350 kPa	Safety Factors (SF): Peak Sliding: C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa) *before cracking*	Stress at the base: Downstream of the rotation point (kPa) *f'c = 20 MPa*	Position of the Force Resultant *length of the crack (m)*
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**0 <**	**< 6 000**	**Middle third**
N1: Maximum oprating level in summer	180.50	176.48	1.50	17.03	43	0 *-3.91*	55	median half *Fissure : 1.02*
N2: Maximum operating level in winter with normal static ice load	180.40	176.48	1.41	6.35	141	0 *-23*	108	within the base *Fissure : 7.84*
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**0 <**	**< 10 000**	**Median half**
I1 - Maximum operating level in winter with unusual ice load	180.40	176.48	1.51	5.28	96	0 *-33*	138	within the base *Fissure : 8.38*
I2 - Inflow Design Flood level (IDF)	181.35	177.66	1.55	19.18	17	2.58	64	middle third
I3 - Post-Seismic analysis of N1	180.50	176.48	1.50	16.90	5	0.46	55	median half *Fissure : 1.12*
I4 - Post-Seismic analysis of N2	180.40	176.48	1.41	14.25	15	0 *-22*	108	within the base *Fissure : 7.84*
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**0 <**	**< 18 000**	**Within the base**
E1 - Earthquake - 1:500	180.50	176.48	1.50	16.33	6	-5.13	56	median half *fissure : 1.12*
E2 - Maximum oprating level in winter with extreme static ice load	180.40	176.48	1.22	0.39	4 694	0 *-56*	443 317	within the base *Fissure : 13.31*

Base characteristics	
Length of the base	13.412 m
Width of the base	7.925 m
Crest level	181.80 m
Rotation point level	172.91 m
Height of the section	8.892 m



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Public Works and Government Services Canada

Port Severn Bundle

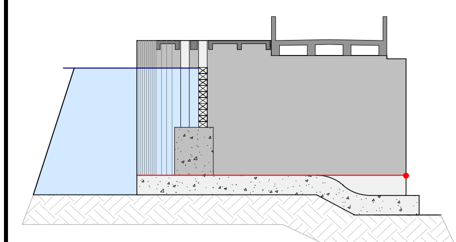
Main Dam - Pier # 3 - concrete-rock interface (existing condition)

Projected by : Herns Jean-Baptiste eng.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 1.1

Load Combinations	Water level: *Upstream of the rotation point*	Water level: *Downstream of the rotation point*	Safety Factors (SF): *Uplift*	Safety Factors (SF): *Peak Sliding* C = 300 kPa	Safety Factors (SF): *Peak Sliding* C_{min} required (kPa)	Stress at the base: *Upstream of the rotation point (kPa)* ft = 0.05 · f'c	Stress at the base: *Downstream of the rotation point (kPa)* f'c = 20 MPa	Position of the Force Resultant *length of the crack (m)*
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**- 1 000 <**	**< 6 000**	**Middle third**
N1 - Maximum oprating level in summer	180.50	176.48	5.62	10.82	0	36	213	middle third
N2 - Maximum operating level in winter with normal static ice load	180.40	176.48	6.04	7.14	32	2.5	264	middle third
						considering a stress of 80 kPa at the interface		
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**- 1 000 <**	**< 10 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	180.40	176.48	6.42	6.15	0	2.8	284	middle third
						considering a stress of 100 kPa at the interface		
I2 - Inflow Design Flood level (IDF)	181.35	177.66	3.06	9.05	0	1.9	203	middle third
						considering a stress of 60 kPa at the interface		
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A *N/A*
I4 - Post-Seismic analysis of N2	180.40	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A *N/A*
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**- 1 000 <**	**< 18 000**	**Within the base**
E1 - Earthquake	180.50	176.48	5.61	9.79	0	33	215	middle third
E2 - Maximum oprating level in winter with extreme static ice load	180.40	176.48	7.37	4.74	0	3.1	334	middle third
						considering a stress of 151 kPa at the interface		

Base characteristics

Length of the base	12.802 m
Width of the base	1.829 m
Crest level	181.80 m
Rotation point level	174.93 m
Height of the section	6.867 m



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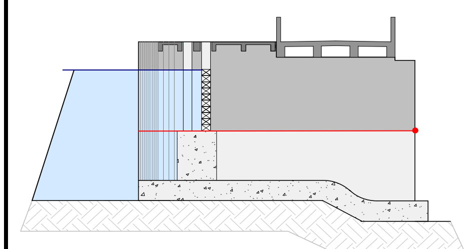
Public Works and Government Services Canada

Port Severn Bundle

Main Dam - Pier # 3 - concrete-concrete interface

Projected by : Herns Jean-Baptiste eng.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 1.2

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Safety Factors (SF): Uplift	Safety Factors (SF): Peak Sliding, C = 300 kPa	Safety Factors (SF): Peak Sliding, C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa), ft = 0.05 · f'c	Stress at the base: Downstream of the rotation point (kPa), f'c = 20 MPa	Position of the Force Resultant, *length of the crack (m)*	Base characteristics
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**-1 000 <**	**< 6 000**	**Middle third**	**Length of the base** 12.802 m
N1 - Maximum oprating level in summer	180.50	176.48	10.08	20.05	0	54	156	middle third	**Width of the base** 1.829 m
N2 - Maximum operating level in winter with normal static ice load	180.40	176.48	10.36	9.24	3	6	201	middle third	**Crest level** 181.80 m
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**-1 000 <**	**< 10 000**	**Median half**	**Rotation point level** 176.77 m
I1 - Maximum operating level in winter with unusual static ice load	180.40	176.48	10.80	6.95	0	2	215	middle third	**Height of the section** 5.029 m
						considering a stress of 62 kPa at the interface			
I2 - Inflow Design Flood level (IDF)	181.35	177.66	3.62	13.86	0	21	146	middle third	
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
I4 - Post-Seismic analysis of N2	180.40	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**-1 000 <**	**< 18 000**	**Within the base**	
E1 - Earthquake - 1:500	180.50	176.48	10.07	18.69	0	52	157	middle third	
E2 - Maximum oprating level in winter with extreme static ice load	180.40	176.48	12.17	5.02	0	2	245	middle third	
						considering a stress of 91 kPa at the interface			



AECOM

Public Works and Government Services Canada

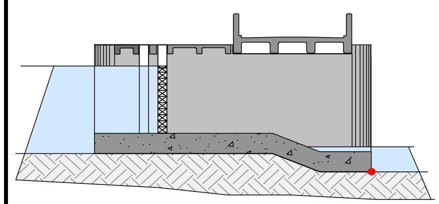
Port Severn Bundle

Main Dam - Pier # 3 - concrete-concrete interface

Projected by : Herns Jean-Baptiste eng.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 1.3

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Safety Factors (SF): *Uplift*	Safety Factors (SF): *Peak Sliding* C = 350 kPa	Safety Factors (SF): *Peak Sliding* C_{min} required (kPa)	Stress at the base: *Upstream of the rotation point (kPa)* *before cracking*	Stress at the base: *Downstream of the rotation point (kPa)* f'c = 20 MPa	Position of the Force Resultant *length of the crack (m)*
<u>Normal loading cases</u>			**SF > 1.20**	**SF > 3.00**		**0 <**	**< 6 000**	**Middle third**
N1 - Maximum oprating level in summer	180.50	176.48	1.90	34.80	13	12	34	middle third
N2 - Maximum operating level in winter with normal static ice load	180.40	176.48	1.91	21.96	31	0 *-0.39*	47	median half *Fissure : 0.31*
<u>Unusual loading cases</u>			**SF > 1.10**	**SF > 2.00**		**0 <**	**< 10 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	180.40	176.48	1.56	11.34	42	0 *-6.85*	58	within the base *Fissure : 5.47*
I2 - Inflow Design Flood level (IDF)	181.35	177.66	1.30	30.38	14	14	15	middle third
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
I4 - Post-Seismic analysis of N2	180.40	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
<u>Extreme loading cases</u>			**SF > 1.10**	**SF > 1.30**		**0 <**	**< 18 000**	**within the base**
E1 - Earthquake - 1:500	180.50	176.48	1.90	33.55	0	11	34	middle third
E2 - Maximum oprating level in winter with extreme static ice load	180.40	176.48	1.22	0.35	4 039	0 *-22.25*	120 170	within the base *Fissure : 13.32*

Base characteristics	
Length of the base	13.413 m
Width of the base	7.925 m
Crest level	181.80 m
Rotation point level	175.47 m
Height of the section	6.334 m



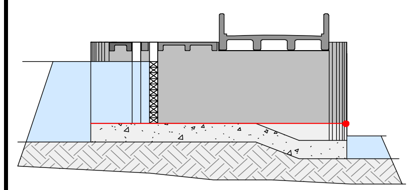
AECOM

Public Works and Government Services Canada

Port Severn Bundle

Main Dam - Pier # 8 - concrete-rock interface (existing condition)

Projected by : Herns Jean-Baptiste eng.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 2.1

Load Combinations	Water level		Safety Factors (SF)			Stress at the base		Position of the Force Resultant	Base characteristics
	Upstream of the rotation point	*Downstream of the rotation point*	*Uplift*	*Peak Sliding*		*Upstream of the rotation point (kPa)*	*Downstream of the rotation point (kPa)*		
				C = 300 kPa	C_{min} required (kPa)	ft = 0.05 · f'c	f'c = 20 MPa	*length of the crack (m)*	
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**- 1 000 <**	**< 6 000**	**Middle third**	**Length of the base** 12.802 m
N1 - Maximum oprating level in summer	180.50	176.48	10.82	27.61	0	59	139	middle third	**Width of the base** 1.829 m
N2 - Maximum operating level in winter with normal static ice load	180.40	176.48	11.18	10.28	0	17	179	middle third	**Crest level** 181.80 m
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**- 1 000 <**	**< 10 000**	**Median half**	**Rotation point level** 177.38 m
I1- Maximum operating level in winter with unusual static ice load	180.40	176.48	11.32	7.78	0	0.9	196	middle third	**Height of the section** 4.420 m
						considering a stress of 50 kPa at the interface			
I2 - Inflow Design Flood level (IDF)	181.35	177.66	4.00	17.20	0	28	134	middle third	
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
I4 - Post-Seismic analysis of N2	180.40	176.48	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**- 1 000 <**	**< 18 000**	**within the base**	
E1 - Earthquake 1:500	180.50	176.48	10.80	25.61	0	57	140	middle third	
E2 - Maximum oprating level in winter with extreme static ice load	180.40	176.48	12.62	5.09	0	0.6	222	median half	
						considering a stress of 70 kPa at the interface			

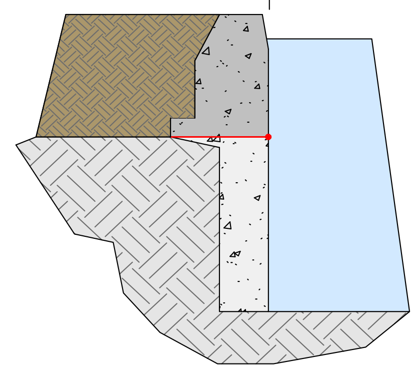
AECOM		**Public Works and Government Services Canada**	
		Port Severn Bundle	
Projected by : Herns Jean-Baptiste eng.	March 2013	Main Dam - Pier # 8 - concrete-concrete interface	
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 2.2

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Safety Factors (SF): Uplift	Safety Factors (SF): Peak Sliding, C = 300 kPa	Safety Factors (SF): Peak Sliding, C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa), ft = 0.05 · f'c	Stress at the base: Downstream of the rotation point (kPa), f'c = 14 MPa	Position of the Force Resultant, *length of the crack (m)*
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**-700 <**	**< 4 200**	**Middle third**
N1 - Maximum operating level in summer	178.09	180.50	4.93	1 074.36	0.00	35.15	58	middle third
N2 - Maximum operating level in winter with normal static ice load	178.09	180.40	8.07	13.08	0.00	159	1.8	middle third
						considering a stress of 95 kPa at the interface		
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**-700 <**	**< 7 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	178.09	180.40	10.49	9.39	0.00	214	1.1	middle third
						considering a stress of 151 kPa at the interface		
I2 - Inflow Design Flood level (IDF)	178.09	181.35	3.64	41.01	0.00	69	15	middle third
I3 - Post-Seismic analysis of N1	178.09	180.40	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
I4 - Post-Seismic analysis of N2	178.09	180.40	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**-700 <**	**< 12 600**	**Within the base**
E1 - Earthquake - 1:500	178.09	180.40	5.13	175.05	0.00	41	53	middle third
E2 - Maximum operating level in winter with extreme static ice load	178.09	180.40	16.42	6.23	0.00	348	2	middle third
						considering a stress of 312 kPa at the interface		

Base characteristics

Length of the base	2.439 m
Width of the base	1.000 m
Crest level	181.33 m
Rotation point level	178.09 m
Height of the section	3.243 m

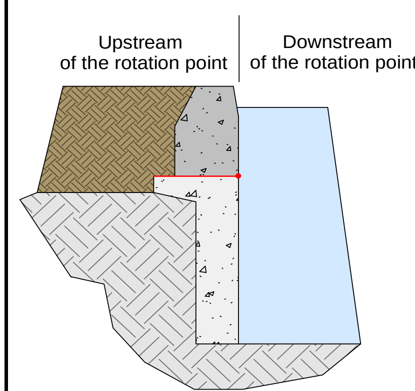
Upstream of the rotation point | Downstream of the rotation point



AECOM	Public Works and Government Services Canada
	Port Severn Bundle
Projected by : Mélanie Otis eng. jr. — March 2013	Lock 45 - Right wall - concrete-concrete interface
Approved by : Amélie Desrosiers eng. — March 2013	Contrat No. 6024 5720 — Table 3.1

Load Combinations	Water level: *Upstream of the rotation point*	Water level: *Downstream of the rotation point*		Safety Factors (SF): *Uplift*	Safety Factors (SF): *Peak Sliding* C = 300 kPa	Safety Factors (SF): *Peak Sliding* C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa) ft = 0.05 · f'c	Stress at the base: Downstream of the rotation point (kPa) f'c = 14 MPa	Position of the Force Resultant *length of the crack (m)*
Normal loading cases				**SF > 1.20**	**SF > 3.00**		**-700 <**	**< 4 200**	**Middle third**
N1 - Maximum operating level in summer	178.59	180.50		5.74	222.03	0.00	37.57	51	middle third
N2 - Maximum operating level in winter with normal static ice load	178.59	180.40		12.08	10.55	0.00	196	0.6	middle third
							considering a stress of 135 kPa at the interface		
Unusual loading cases				**SF > 1.10**	**SF > 2.00**		**-700 <**	**< 7 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	178.59	180.40		16.32	7.71	0.00	271	1.7	middle third
							considering a stress of 211 kPa at the interface		
I2 - Inflow Design Flood level (IDF)	178.59	181.35		3.97	42.05	0.00	79	2	middle third
I3 - Post-Seismic analysis of N1	178.59	180.40		N/A	N/A	N/A	N/A *N/A*	N/A	N/A
I4 - Post-Seismic analysis of N2	178.59	180.40		N/A	N/A	N/A	N/A *N/A*	N/A	N/A
Extreme loading cases				**SF > 1.10**	**SF > 1.30**		**-700 <**	**< 12 600**	**within the base**
E1 - Earthquake - 1:500	178.59	180.40		6.04	631.61	0.00	44	46	middle third
E2 - Maximum operating level in winter with extreme static ice load	178.59	180.40		26.40	5.28	0.00	450	1.5	middle third
							considering a stress of 391 kPa at the interface		

Base characteristics
Length of the base 1.829 m
Width of the base 1.000 m
Crest level 181.33 m
Rotation point level 178.59 m
Height of the section 2.743 m



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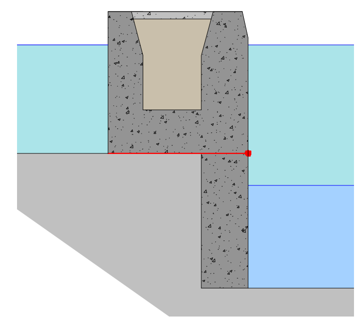
Public Works and Government Services Canada

Port Severn Bundle

Lock 45 - Right wall - concrete-concrete interface

Projected by : Mélanie Otis eng. jr.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 3.2

Load Combinations	Water level		Safety Factors (SF)			Stress at the base		Position of the Force Resultant	Base characteristics
	Upstream of the rotation point	*Downstream of the rotation point*	*Uplift*	*Peak Sliding*		*Upstream of the rotation point (kPa)*	*Downstream of the rotation point (kPa)*		
				C = 350 kPa	C_{min} required (kPa)	*before cracking*	*f'c = 20 MPa*	*length of the crack (m)*	
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**0 <**	**< 6 000**	**Middle third**	**Length of the base** 3.658 m
N1 - Maximum oprating level in summer	180.50	176.48	7.99	31.83	0	33	110	middle third	**Width of the base** 1.000 m
N2 - Maximum operating level in winter with normal static ice load	180.40	180.40	8.25	1 531 472	0	56	88	middle third	**Crest level** 181.33 m
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**0 <**	**< 10 000**	**Median half**	**Rotation point level** 177.37 m
I1 - Maximum operating level in winter with normal static ice load	180.40	180.40	8.25	1 531 472	0	56	88	middle third	**Height of the section** 3.962 m
I2 - Inflow Design Flood level (IDF)	181.35	177.66	6.28	19.80	0	1.71	136	middle third	
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A	N/A	N/A	
I4 - Post-Seismic analysis of N2	180.40	180.40	N/A	N/A	N/A	N/A	N/A	N/A	
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**0 <**	**< 18 000**	**Within the base**	
E1 - Earthquake - 1:500	180.50	176.48	7.98	29.68	0	31	112	middle third	
E2 - Maximum oprating level in winter with extreme static ice load	180.40	180.40	8.25	1 531 472	0	56	88	middle third	



AECOM

Projected by : Amélie Desrosiers eng. — July 2013

Approved by :

Public Works and Government Services Canada

Port Severn Bundle

Lock 45 - Left wall - concrete-rock interface

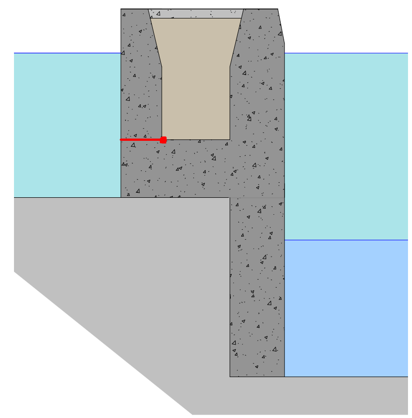
Contrat No. 6024 5720

Table 3.3

Load Combinations	Water level: *Upstream of the rotation point*	Water level: *Downstream of the rotation point*	Safety Factors (SF): *Uplift*	Safety Factors (SF): *Peak Sliding* C = 300 kPa	Safety Factors (SF): *Peak Sliding* C_{min} required (kPa)	Stress at the base: *Upstream of the rotation point (kPa)* ft = 0.05 · f'c	Stress at the base: *Downstream of the rotation point (kPa)* f'c = 20 MPa	Position of the Force Resultant length of the crack (m)
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**-1000 <**	**< 6 000**	**Middle third**
N1 - Maximum oprating level in summer	180.50	176.48	7.49	19.73	0	0.50	121	middle third
					considering a stress of 57 kPa at the interface			
N2 - Maximum operating level in winter with normal static ice load	180.40	180.40	47.32	8.16	0	0.50	823	middle third
					considering a stress of 766 kPa at the interface			
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**-1000 <**	**< 10 000**	**Median half**
I1 - Maximum operating level in winter with 97% of the unusual ice load	180.40	180.40	54.49	6.79	0	0.50	1 196	median half 0.105 m
					considering a stress of 1000 kPa at the interface			
I2 - Inflow Design Flood level (IDF)	181.35	177.66	10.04	11.61	0	0.50	245	middle third
					considering a stress of 190 kPa at the interface			
I3 - Post-Seismic analysis of N1	180.50	176.48	N/A	N/A	N/A	N/A	N/A	N/A
I4 - Post-Seismic analysis of N2	180.40	180.40	N/A	N/A	N/A	N/A	N/A	N/A
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**-1000 <**	**< 18 000**	**Within the base**
E1 - Earthquake - 1:500	180.50	176.48	7.64	19.32	0	0.50	124	middle third
					considering a stress of 60 kPa at the interface			
E2 - Maximum oprating level in winter with 52% of the extreme ice load	180.40	180.40	57.77	7.16	0	0.50	1 149	median half 0.059 m
					considering a stress of 1000 kPa at the interface			

Base characteristics

Base characteristics	
Length of the base	0.915 m
Width of the base	1.000 m
Crest level	181.33 m
Rotation point level	178.59 m
Height of the section	2.743 m



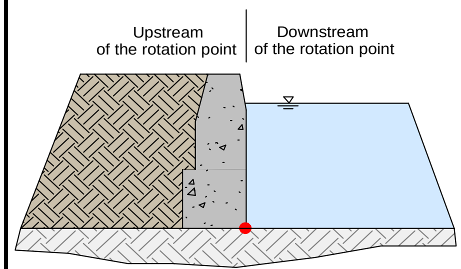
AECOM

Projected by : Amélie Desrosiers eng.	July 2013	**Public Works and Government Services Canada** Port Severn Bundle Lock 45 - Left wall - concrete-concrete interface	
Approved by :		Contrat No. 6024 5720	Table 3.4

Load Combinations	Water level: *Upstream of the rotation point*	Water level: *Downstream of the rotation point*	Safety Factors (SF): *Uplift*	Safety Factors (SF): *Peak Sliding* C = 350 kPa	Safety Factors (SF): *Peak Sliding* C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa) *before cracking*	Stress at the base: Downstream of the rotation point (kPa) f'c = 14 MPa	Position of the Force Resultant *length of the crack (m)*
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**0 <**	**< 4 200**	**Middle third**
N1 - Maximum operating level in summer	177.37	180.50	4.85	136.25	0	47	72	middle third
N2 - Maximum operating level in winter with normal static ice load (Kp = 2.61)	177.37	180.40	5.01	6.40	143	1.55	118	middle third
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**0 <**	**< 7 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load (Kp = 3.00)	177.37	180.40	5.01	6.96	68	181	-62	within the base *crack : 0.39*
I2 - Inflow Design Flood level (IDF) (Kp = 1.12)	177.37	181.35	3.82	32.06	0	0.65	109	middle third
I3 - Post-Seismic analysis of N1	177.37	180.50	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
I4 - Post-Seismic analysis of N2	177.37	180.40	N/A	N/A	N/A	N/A *N/A*	N/A	N/A
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**0 <**	**< 12 600**	**within the base**
E1 - Earthquake - 1:500	177.37	180.50	4.85	34.87	0	90	28	middle third
E2 - Maximum operating level in winter with extreme static ice load (Kp = 3.00)	177.37	180.40	5.01	67.83	0	884	-764	within the base *crack : 0.71*

Base characteristics

Length of the base	1.524 m
Width of the base	1.000 m
Crest level	181.31 m
Rotation point level	177.37 m
Height of the section	3.940 m



AECOM

Projected by : Mélanie Otis eng. jr.	March 2013	**Public Works and Government Services Canada**	
		Port Severn Bundle	
		Upstream Shoreline Wall - concrete-rock interface	
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 4.1

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Safety Factors (SF): Uplift	Safety Factors (SF): Peak Sliding – C = 300 kPa	Safety Factors (SF): Peak Sliding – C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa) ft = 0.05 · f'c	Stress at the base: Downstream of the rotation point (kPa) f'c = 14 MPa	Position of the Force Resultant (length of the crack (m))
Normal loading cases			**SF > 1.20**	**SF > 3.00**		**-700 <**	**< 4 200**	**Middle third**
N1 - Maximum operating level in summer	178.87	180.50	5.96	123.49	0.00	21	58	middle third
N2 - Maximum operating level in winter with normal static ice load	178.87	180.40	21.46	6.15	0.00	305	1.2	middle third
					considering a stress of 252 kPa at the interface			
Unusual loading cases			**SF > 1.10**	**SF > 2.00**		**-700 <**	**< 7 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	178.87	180.40	30.63	6.37	0.00	443	0.8	middle third
					considering a stress of 390 kPa at the interface			
I2 - Inflow Design Flood level (IDF)	178.87	181.35	4.46	34.11	0.00	83	2	middle third
					considering a stress of 34 kPa at the interface			
I3 - Post-Seismic analysis of N1	178.87	180.50	N/A	N/A	N/A	N/A	N/A	N/A
I4 - Post-Seismic analysis of N2	178.87	180.50	N/A	N/A	N/A	N/A	N/A	N/A
Extreme loading cases			**SF > 1.10**	**SF > 1.30**		**-700 <**	**< 12 600**	**Within the base**
E1 - Earthquake - 1:500	178.87	180.50	5.95	352.73	0.00	34	45	middle third
E2 - Maximum operating level in winter with extreme static ice load	178.87	180.40	21.38	4.29	0.00	669	-364	within the base *crack : 0.43*
					considering a stress of 700 kPa at the interface			

Base characteristics

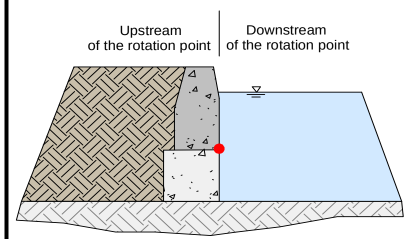
Length of the base
1.219 m

Width of the base
1.000 m

Crest level
181.31 m

Rotation point level
178.87 m

Height of the section
2.438 m



AECOM

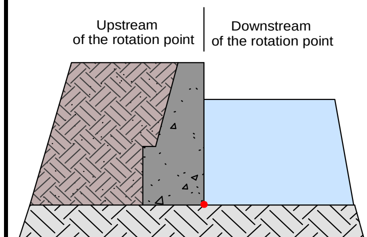
Public Works and Government Services Canada

Port Severn Bundle

Upstream Shoreline Wall - concrete-concrete interface

Projected by : Mélanie Otis eng. jr.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 4.2

Load Combinations	Water level: Upstream of the rotation point	Water level: Downstream of the rotation point	Uplift	Peak Sliding: C = 350 kPa	Peak Sliding: C_{min} required (kPa)	Stress at the base: Upstream of the rotation point (kPa) *before cracking*	Stress at the base: Downstream of the rotation point (kPa) f'c = 12 MPa	Position of the Force Resultant *length of the crack (m)*	Base characteristics
<u>Normal loading cases</u>			**SF > 1.20**	**SF > 3.00**		**0 <**	**< 3 600**	**Middle third**	**Length of the base** 1.445 m
N1 - Maximum operating level in summer	177.80	180.50	4.99	1 906	0	21	85	middle third	**Width of the base** 1.000 m
N2 - Maximum operating level in winter with normal static ice load (Kp = 2.35)	177.80	180.40	5.18	11.09	65	104	2.62	middle third	**Crest level** 181.39 m
<u>Unusual loading cases</u>			**SF > 1.10**	**SF > 2.00**		**0 <**	**< 6 000**	**Median half**	**Rotation point level** 177.80 m
I1 - Maximum operating level in winter with unusual static ice load (Kp = 3.00)	177.80	180.40	5.18	10.11	35	219	-112	within the base *crack : 0.49*	**Height of the section** 3.590 m
I2 - Inflow Design Flood level (IDF) (Kp = 1.05)	177.80	181.35	3.80	42.09	0	2.28	95	middle third	
I3 - Post-Seismic analysis of N1	177.80	180.50	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
I4 - Post-Seismic analysis of N2	177.80	180.40	N/A	N/A	N/A	N/A *N/A*	N/A	N/A	
<u>Extreme loading cases</u>			**SF > 1.10**	**SF > 1.30**		**0 <**	**< 10 800**	**within the base**	
E1 - Earthquake - 1:500	177.80	180.50	4.98	54.86	0	58	48	middle third	
E2 - Maximum operating level in winter with extreme static ice load (Kp = 3.00)	177.80	180.40	5.18	14.07	0	877	-771	within the base *crack : 0.68*	



AECOM

Public Works and Government Services Canada

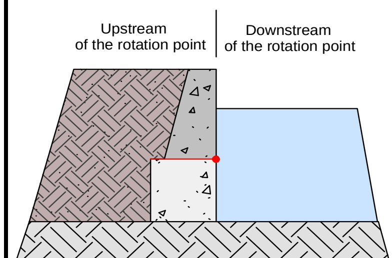
Port Severn Bundle

Dam D - concrete-rock interface

Projected by : Mélanie Otis eng. jr.	March 2013		
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 5.1

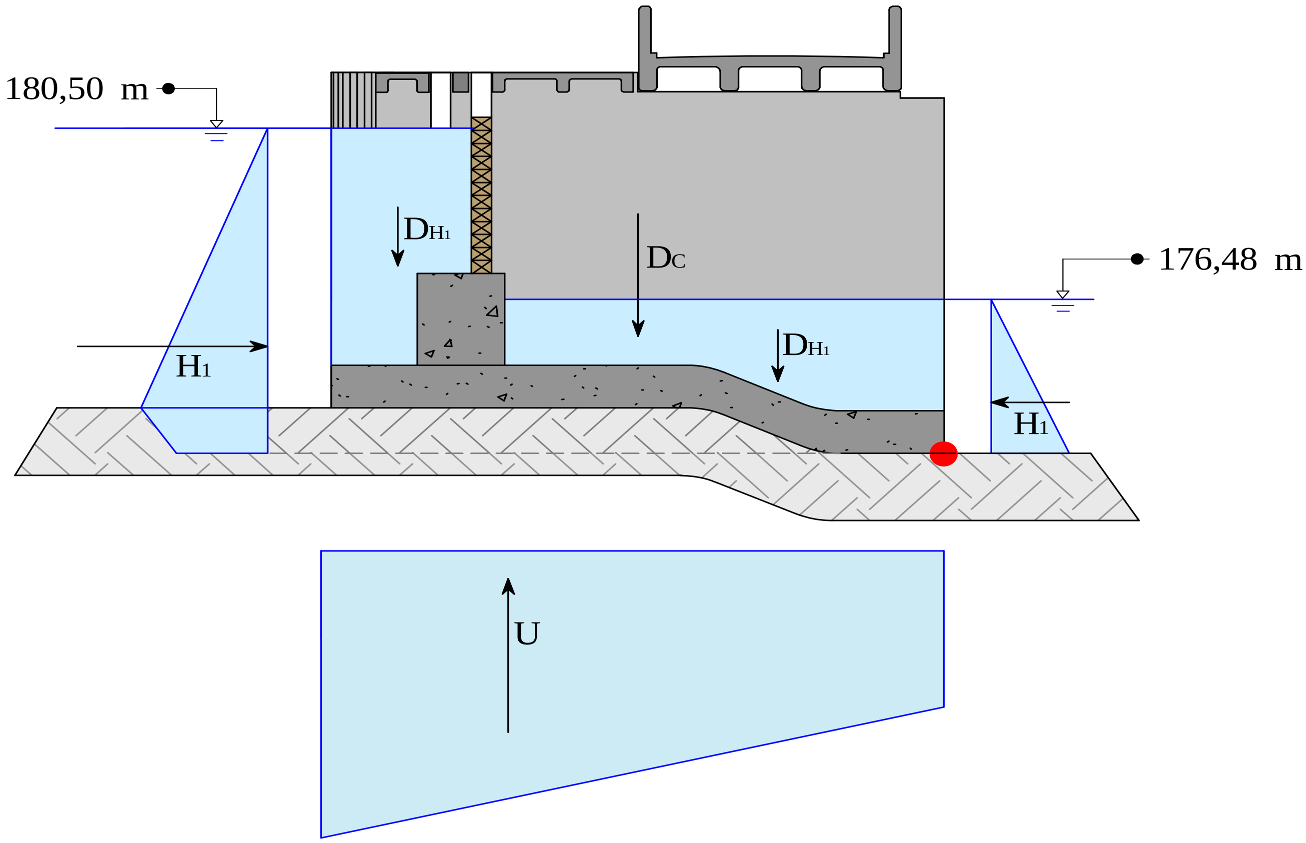
Load Combinations	Water level		Safety Factors (SF)			Stress at the base		Position of the Force Resultant
	Upstream of the rotation point	*Downstream of the rotation point*	*Uplift*	*Peak Sliding* C = 300 kPa	*Peak Sliding* C_{min} required (kPa)	Upstream of the rotation point (kPa) ft = 0.05 · f'c	Downstream of the rotation point (kPa) f'c = 12 MPa	*length of the crack (m)*
<u>Normal loading cases</u>			**SF > 1.20**	**SF > 3.00**		**-600 <**	**< 3 600**	**Middle third**
N1 - Maximum operating level in summer	179.28	180.50	6.48	78.61	0.00	2.50	63	middle third
N2 - Maximum operating level in winter with normal static ice load	179.28	180.40	21.65	6.95	0.00	227	0.5	middle third
						considering a stress of 183 kPa at the interface		
<u>Unusual loading cases</u>			**SF > 1.10**	**SF > 2.00**		**-600 <**	**< 6 000**	**Median half**
I1 - Maximum operating level in winter with unusual static ice load	179.28	180.40	31.22	5.23	0.00	333	0.3	middle third
						considering a stress of 288 kPa at the interface		
I2 - Inflow Design Flood level (IDF)	179.28	181.35	3.83	47.13	0.00	56	1	middle third
I3 - Post-Seismic analysis of N1	179.28	180.50	N/A	N/A	N/A	N/A	N/A	N/A
I4 - Post-Seismic analysis of N2	179.28	180.40	N/A	N/A	N/A	N/A	N/A	N/A
<u>Extreme loading cases</u>			**SF > 1.10**	**SF > 1.30**		**-600 <**	**< 10 800**	**within the base**
E1 - Earthquake - 1:500	179.28	180.50	6.47	267.36	0.00	12	54	middle third
E2 - Maximum operating level in winter with extreme static ice load	179.28	180.40	54.11	3.79	0.00	586	0.4	middle third
						considering a stress of 541 kPa at the interface		

Base characteristics
Length of the base 1.140 m
Width of the base 1.000 m
Crest level 181.39 m
Rotation point level 179.28 m
Height of the section 2.115 m
Height of the section 2.115 m



AECOM		Public Works and Government Services Canada	
		Port Severn Bundle	
Projected by : Mélanie Otis eng. jr.	March 2013	Dam D - concrete-concrete interface	
Approved by : Amélie Desrosiers eng.	March 2013	Contrat No. 6024 5720	Table 5.2

Appendix E2
Loading Combinations Sketches



N1 - MAXIMUM OPERATING LEVEL IN SUMMER

$D + H_1 + U$

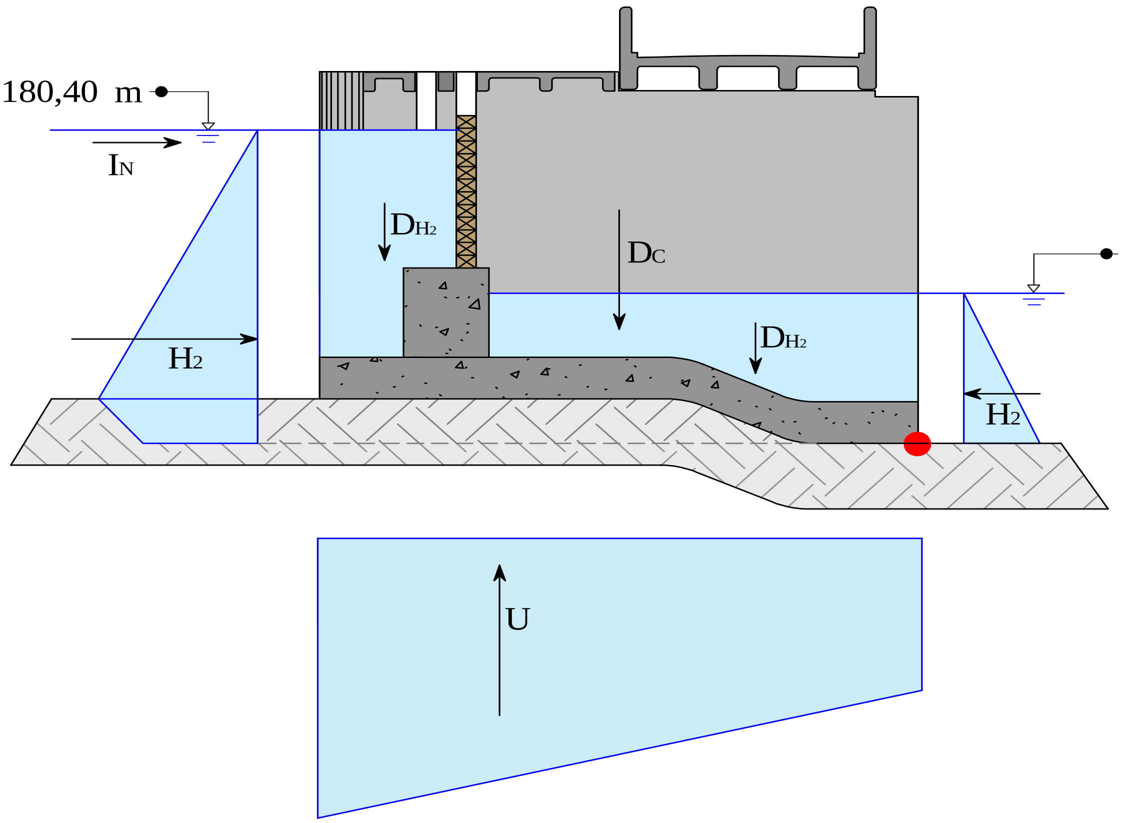
D_C : WEIGHT OF THE CONCRETE

D_{H1} : WEIGHT OF THE WATER

H_1 : HYDROSTATIC PRESSURE IN SUMMER

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada — Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	SPILLWAY NORMAL LOADING CASE - N1
AECOM	PROJECT : 6024-5720 | DATE : March 2013 | figure 1



N2- MAXIMUM OPERATING LEVEL IN WINTER WITH NORMAL STATIC ICE LOAD

$D + H_2 + I_N + U$

D_C : WEIGHT OF THE CONCRETE

D_{H2} : WEIGHT OF THE WATER

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_N : NORMAL STATIC ICE LOAD

U : UPLIFT PRESSURE

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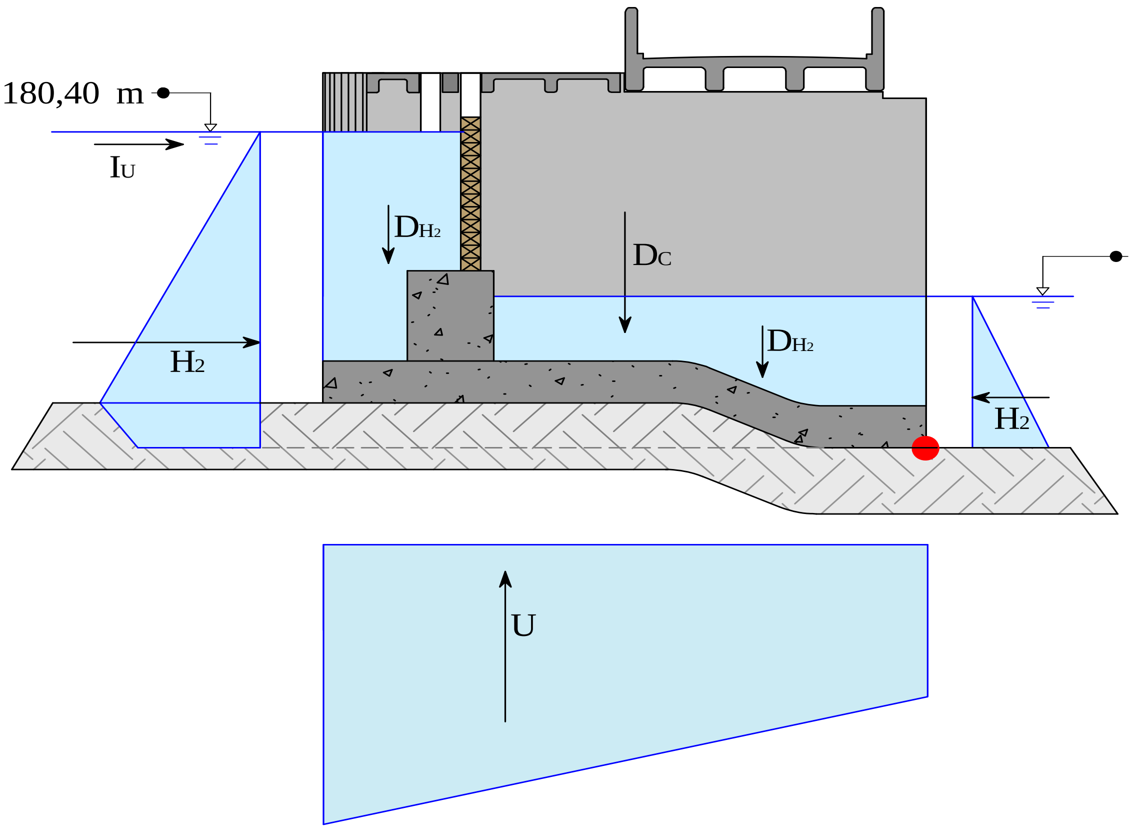
AECOM

PORT SEVERN BUNDLE

SPILLWAY

NORMAL LOADING CASE - N2

PROJECT : 6024-5720	DATE : March 2013	figure 2



I1- MAXIMUM OPERATING LEVEL IN WINTER WITH UNUSUAL STATIC ICE LOAD

$D + H_2 + I_U + U$

D_C : WEIGHT OF THE CONCRETE

D_{H2} : WEIGHT OF THE WATER

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_U : UNUSUAL STATIC ICE LOAD

U : UPLIFT PRESSURE

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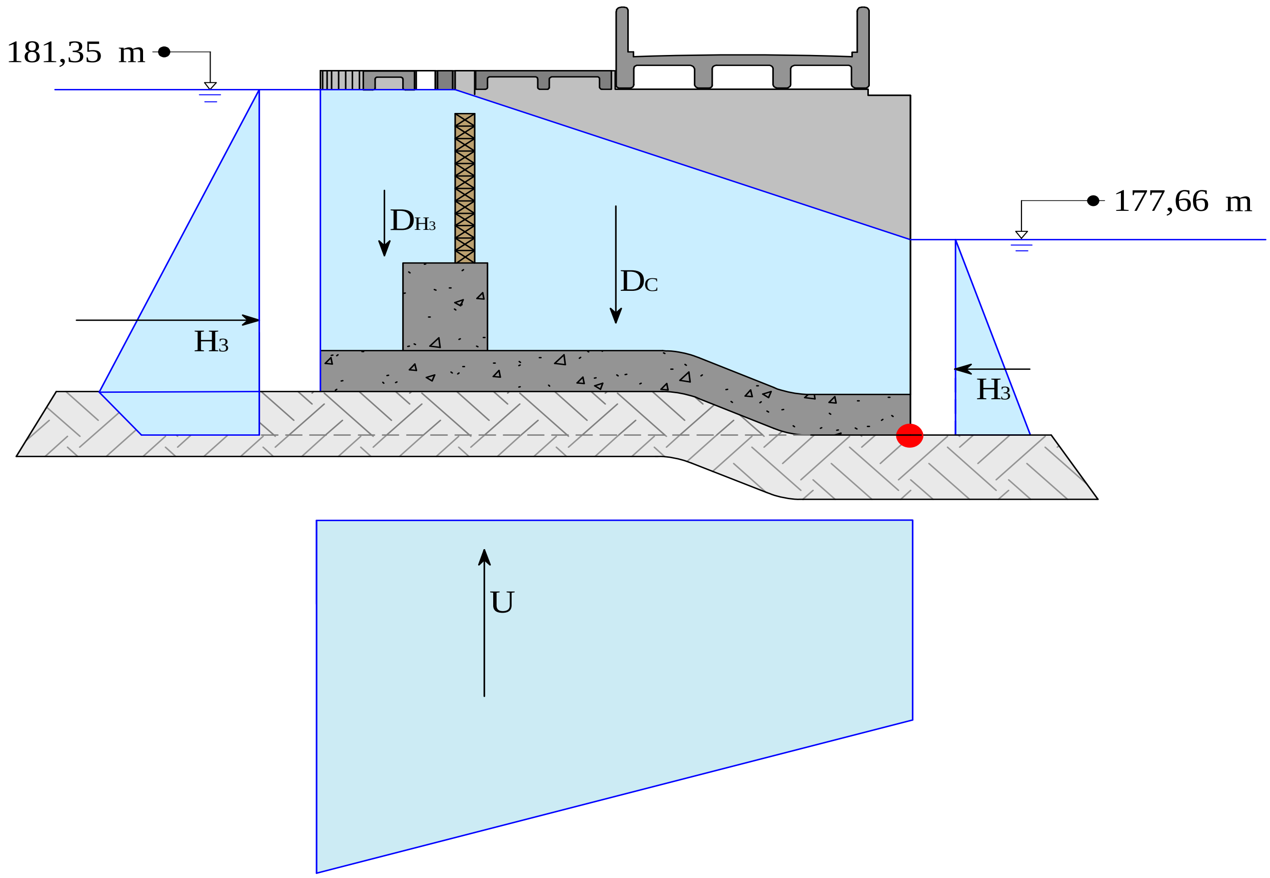
AECOM

PORT SEVERN BUNDLE

SPILLWAY

UNUSUAL LOADING CASE - I1

PROJECT : 6024-5720	DATE : March 2013	figure 3



I2 - INFLOW DESIGN FLOOD LEVEL (IDF)

$D + H_3 + U$

D_C : WEIGHT OF THE CONCRETE

D_{H3} : WEIGHT OF THE WATER

H_3 : HYDROSTATIC PRESSURE WITH IDF

U : UPLIFT PRESSURE

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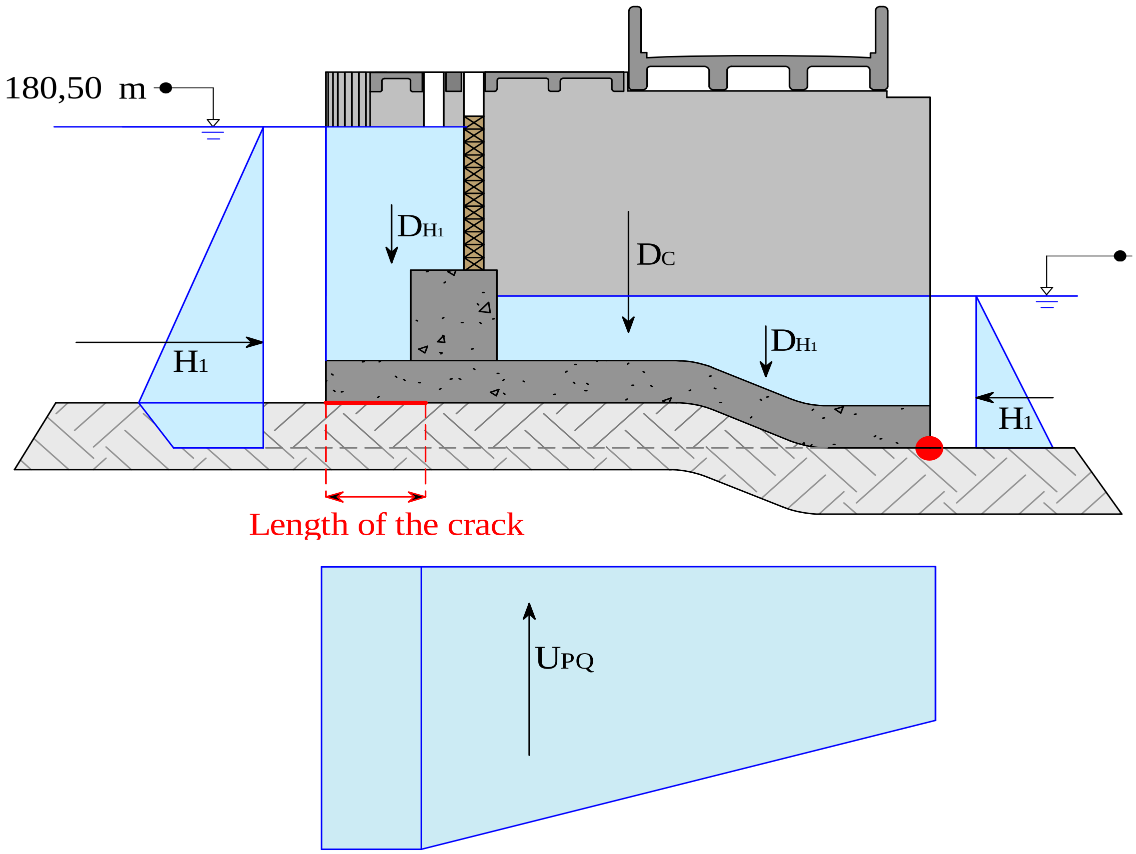
AECOM

PORT SEVERN BUNDLE

SPILLWAY

UNUSUAL LOADING CASE - I2

PROJECT : 6024-5720 | DATE : March 2013 | figure 4



I3 - POST-SEISMIC ANALYSIS OF N1

$D + H_1 + U_{PQ}$

D_C :	WEIGHT OF THE CONCRETE
D_{H1} :	WEIGHT OF THE WATER
H_1 :	HYDROSTATIC PRESSURE IN SUMMER
U_{PQ} :	UPLIFT PRESSURE (POST-SEISMIC)

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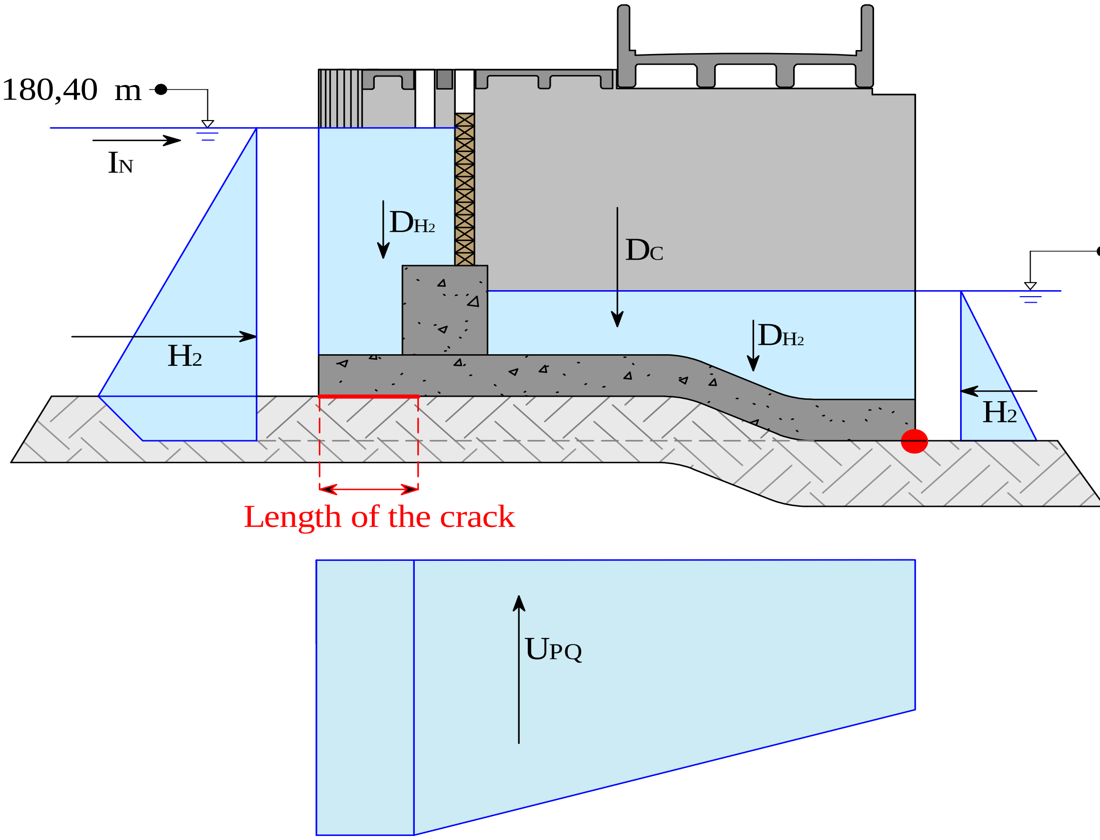
AECOM

PORT SEVERN BUNDLE

SPILLWAY

UNUSUAL LOADING CASE - I3

PROJECT : 6024-5720	DATE : March 2013	figure 5



I4 - POST-SEISMIC ANALYSIS OF N2

$D + H_2 + I_N + U_{PQ}$

D_C : WEIGHT OF THE CONCRETE

D_{H2} : WEIGHT OF THE WATER

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_N : NORMAL STATIC ICE LOAD

U_{PQ} : UPLIFT PRESSURE (POST-SEISMIC)

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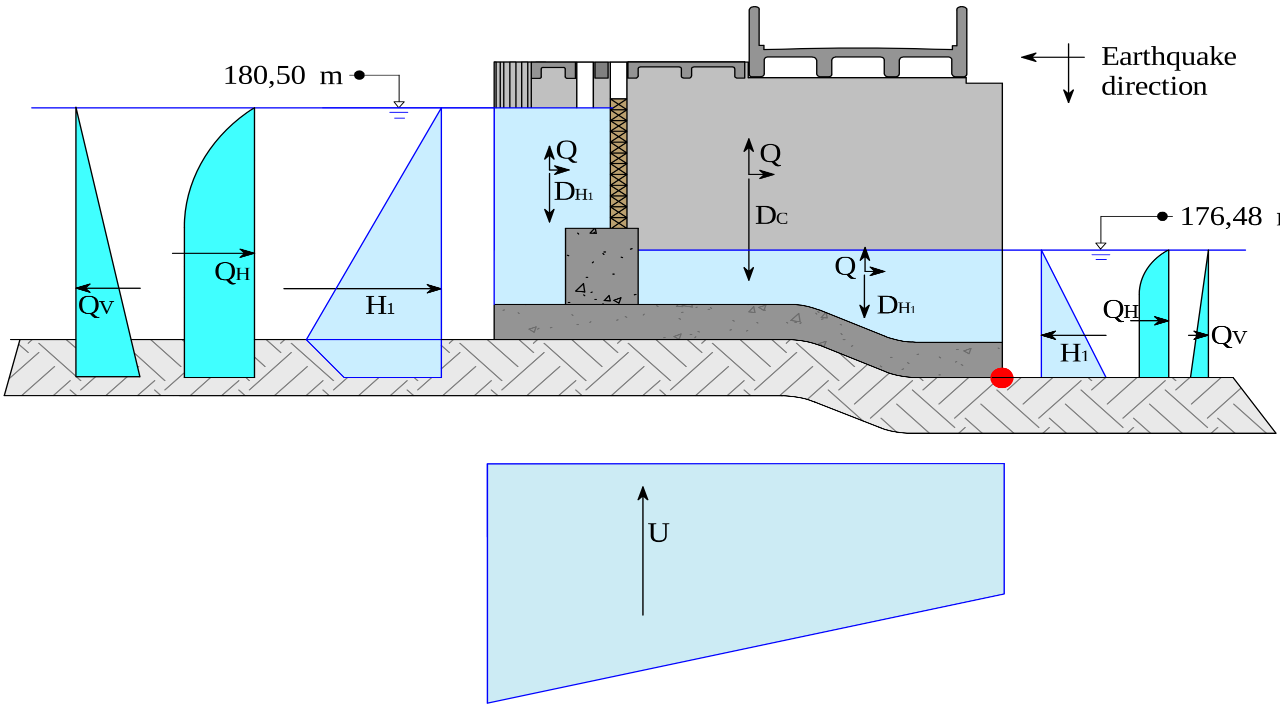
AECOM

PORT SEVERN BUNDLE

SPILLWAY

UNUSUAL LOADING CASE - I4

PROJECT : 6024-5720	DATE : March 2013	figure 6



E1 - EARTHQUAKE

$D + H_1 + U + Q$

D_C :	WEIGHT OF THE CONCRETE
D_{H1} :	WEIGHT OF THE WATER
H_1 :	HYDROSTATIC PRESSURE IN SUMMER
U :	UPLIFT PRESSURE
Q :	EARTHQUAKE LOAD

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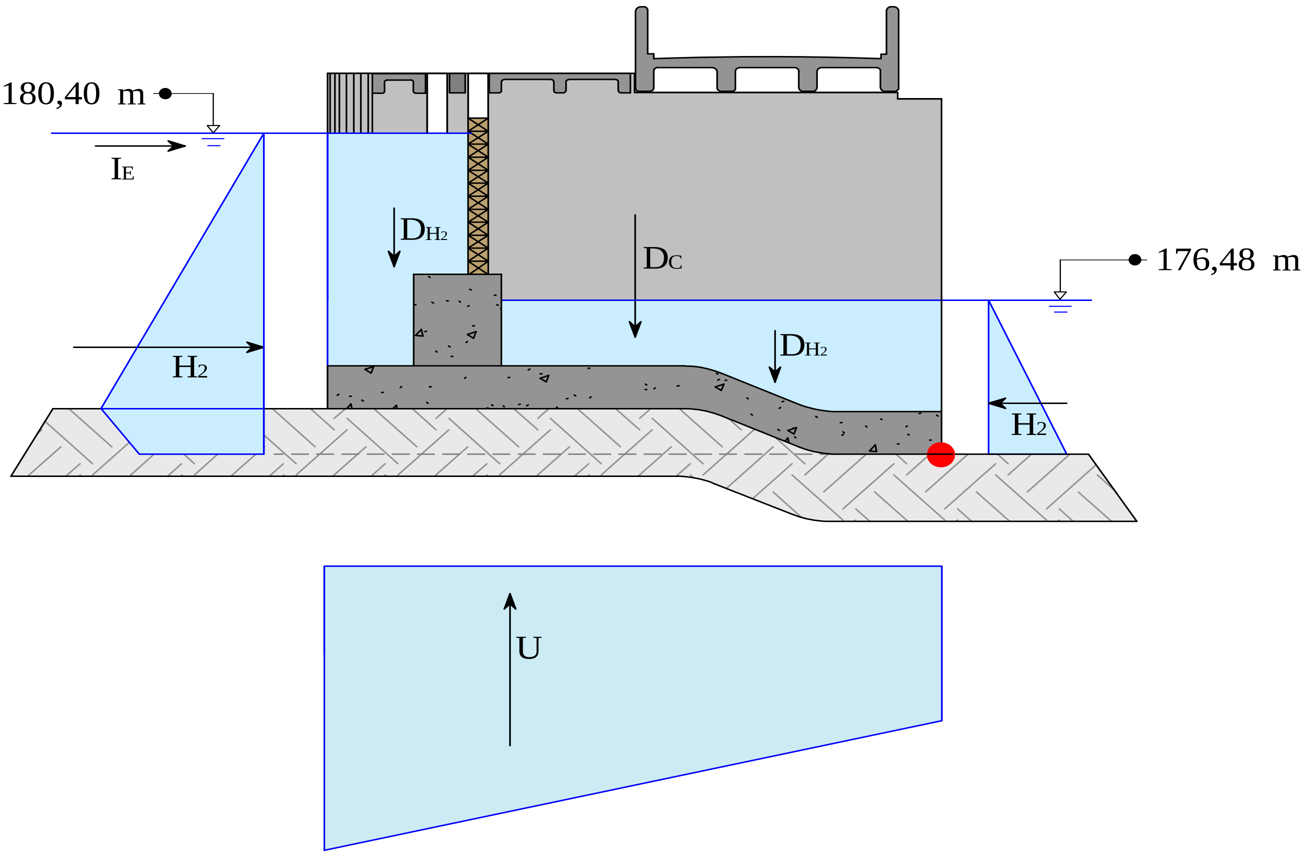
AECOM

PORT SEVERN BUNDLE

SPILLWAY

EXTREME LOADING CASE - E1

PROJECT : 6024-5720	DATE : March 2013	figure 7



E2 - MAXIMUM OPERATING LEVEL IN WINTER WITH EXTREME STATIC ICE LOAD

$D + H_2 + I_E + U$

D_C : WEIGHT OF THE CONCRETE
D_{H2} : WEIGHT OF THE WATER
H_2 : HYDROSTATIC PRESSURE IN WINTER
I_E : EXTREME STATIC ICE LOAD
U : UPLIFT PRESSURE

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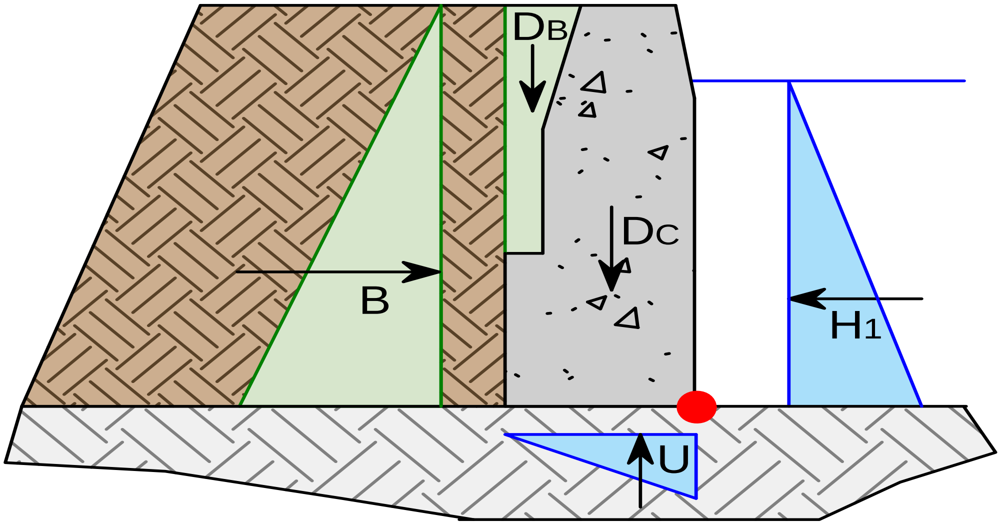
AECOM

PORT SEVERN BUNDLE

SPILLWAY

EXTREME LOADING CASE - E2

PROJECT : 6024-5720	DATE : March 2013	figure 8



N1 - MAXIMUM OPERATING LEVEL IN SUMMER

$D + H_1 + U$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

H_1 : HYDROSTATIC PRESSURE IN SUMMER

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada

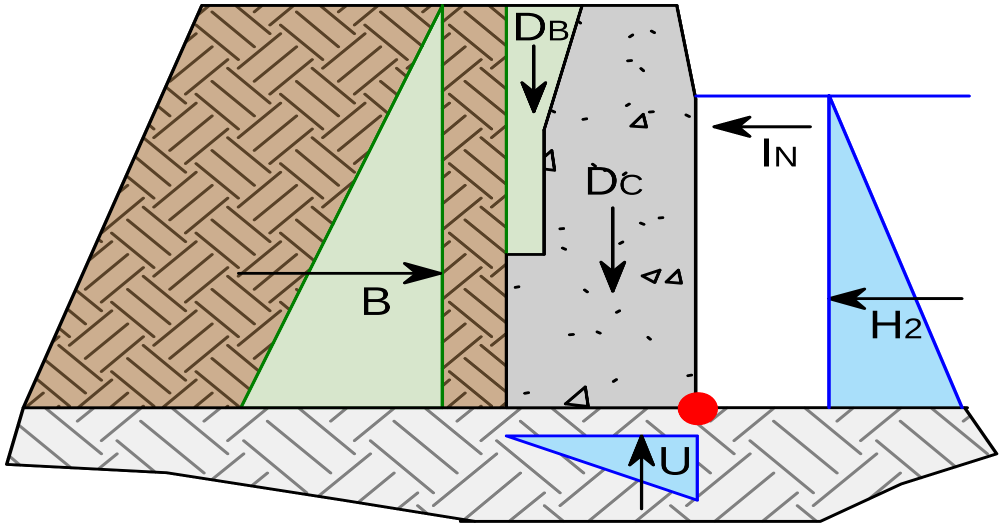
Public Works and Government Services Canada

PORT SEVERN BUNDLE

RETAINING WALL

NORMAL LOADING CASE - N1

PROJECT : 6024-5720	DATE : March 2013	figure 9



N2- MAXIMUM OPERATING LEVEL IN WINTER WITH NORMAL STATIC ICE LOAD

$D + H_2 + I_N + B + U$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_N : NORMAL STATIC ICE LOAD

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada | Public Works and Government Services Canada

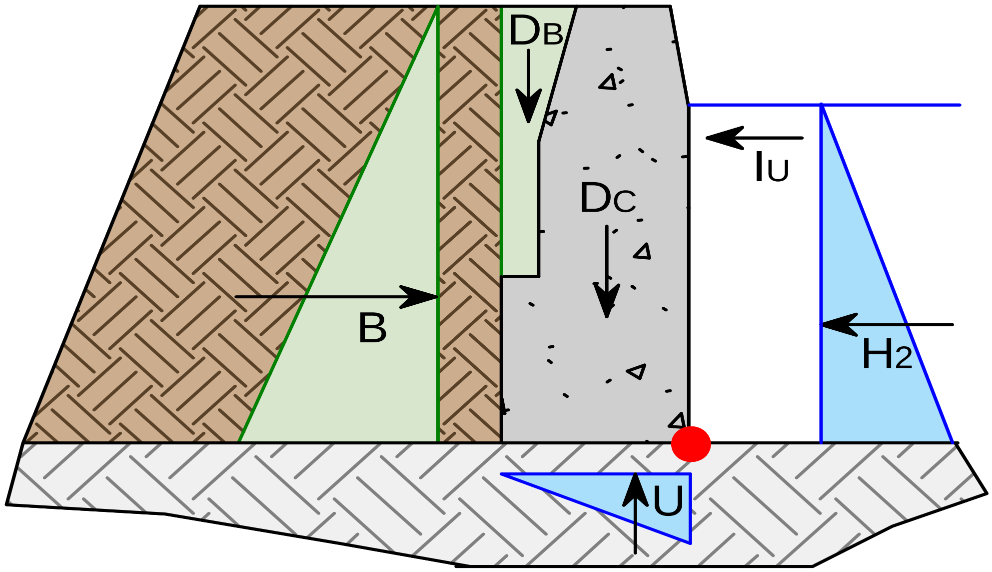
AECOM

PORT SEVERN BUNDLE

RETAINING WALL

NORMAL LOADING CASE - N2

PROJECT : 6024-5720	DATE : March 2013	figure 10



I1- MAXIMUM OPERATING LEVEL IN WINTER WITH UNUSUAL STATIC ICE LOAD

$D + H_2 + I_U + B + U$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

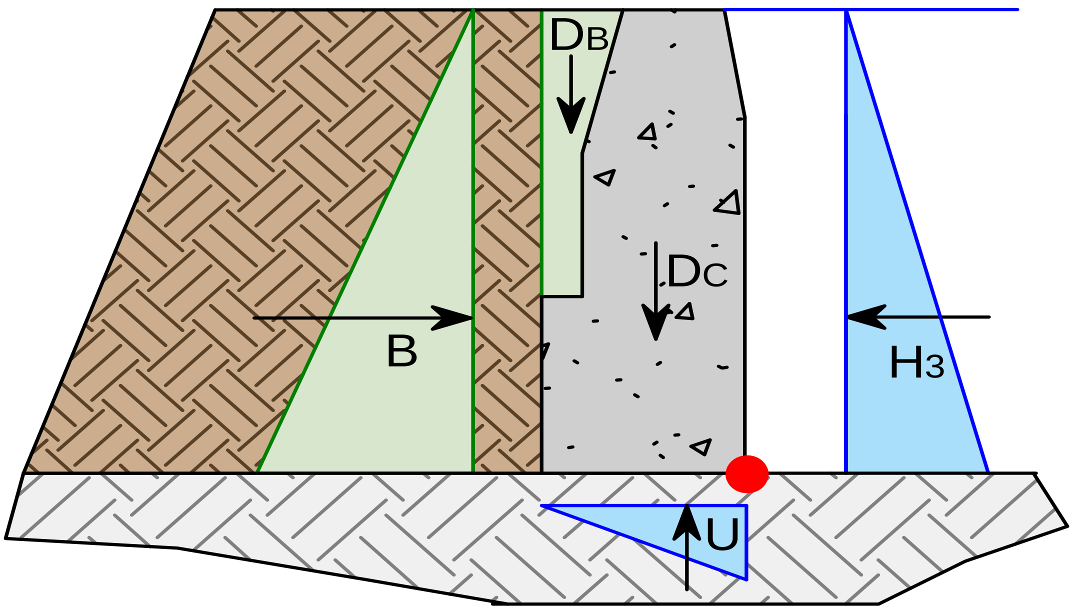
H_2 : HYDROSTATIC PRESSURE IN WINTER

I_U : UNUSUAL STATIC ICE LOAD

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
AECOM	RETAINING WALL UNUSUAL LOADING CASE - I1
	PROJECT : 6024-5720 | DATE : March 2013 | figure 11



I2 - INFLOW DESIGN FLOOD LEVEL (IDF)

$D + H_3 + B + U$

D_B : WEIGHT OF THE BACKFILL

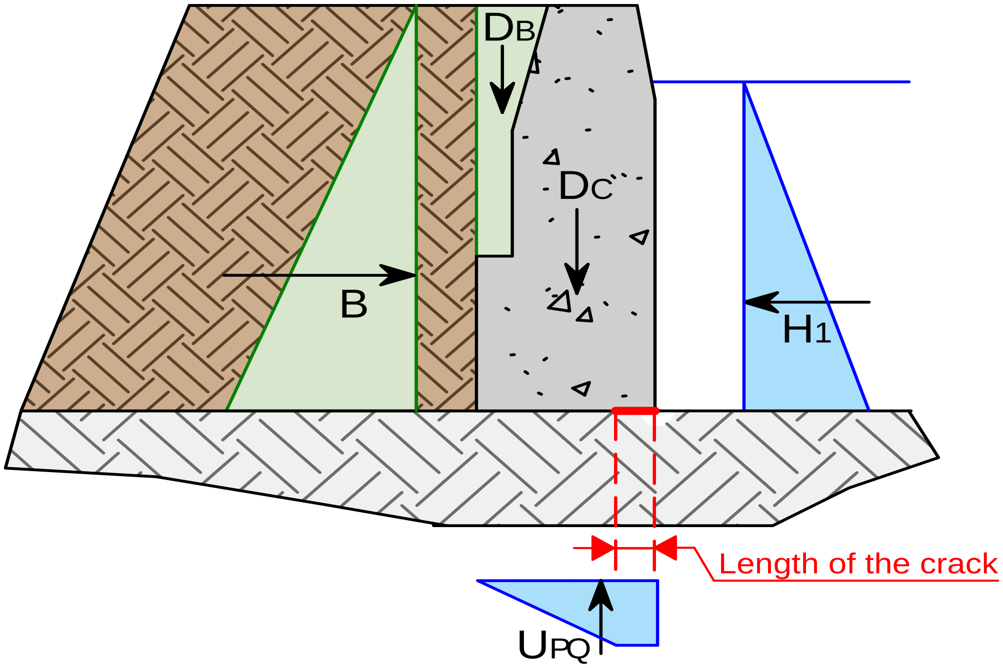
D_C : WEIGHT OF THE CONCRETE

H_3 : HYDROSTATIC PRESSURE WITH IDF

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	RETAINING WALL UNUSUAL LOADING CASE - I2
AECOM	PROJECT : 6024-5720 | DATE : March 2013 | figure 12



I3 - POST-SEISMIC ANALYSIS OF N1

$D + H_1 + B + U_{PQ}$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

H_1 : HYDROSTATIC PRESSURE IN SUMMER

B : BACKFILL PRESSURE

U_{PQ} : UPLIFT PRESSURE (POST-SEISMIC)

Travaux publics et Services gouvernementaux Canada

Public Works and Government Services Canada

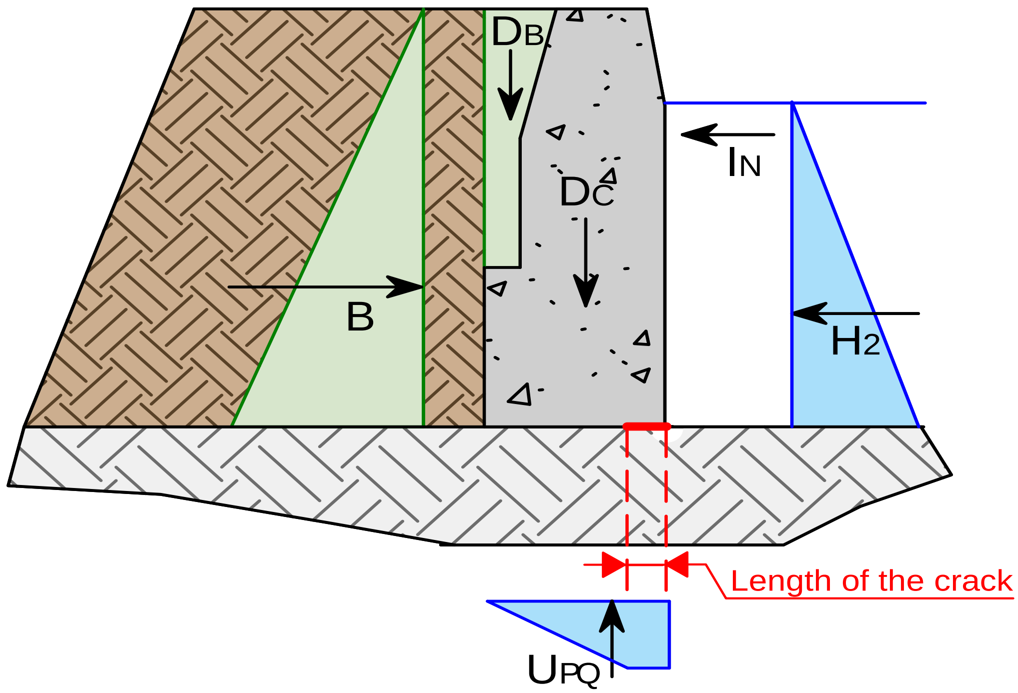
AECOM

PORT SEVERN BUNDLE

RETAINING WALL

UNUSUAL LOADING CASE - I3

PROJECT : 6024-5720	DATE : March 2013	figure 13



I4 - POST-SEISMIC ANALYSIS OF N2

$D + H_2 + I_N + B + U_{PQ}$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

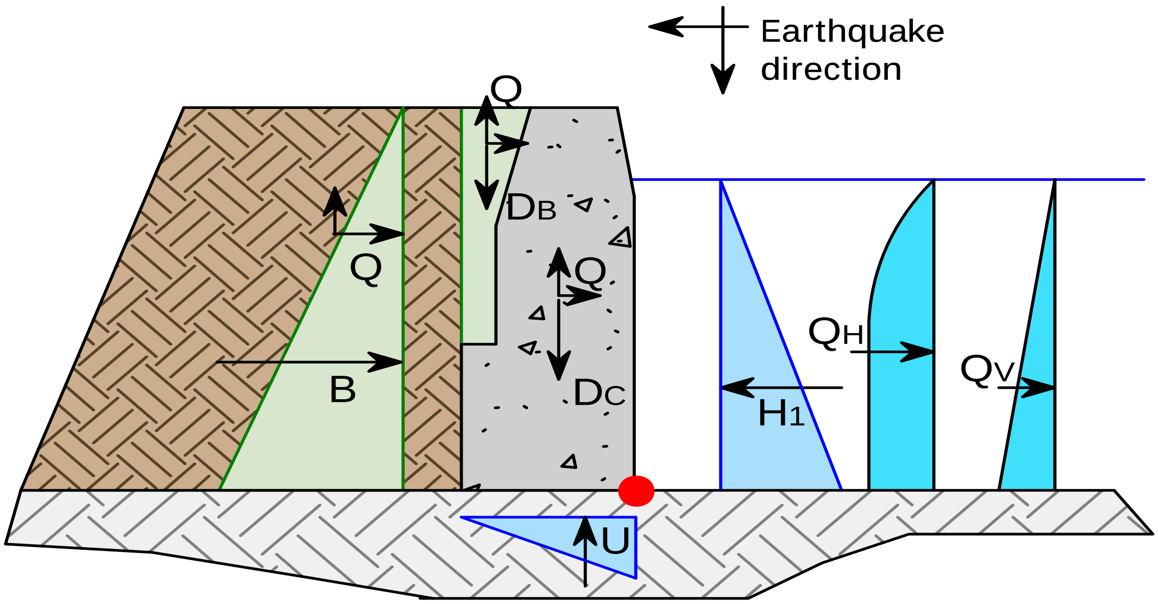
H_2 : HYDROSTATIC PRESSURE IN WINTER

I_N : NORMAL STATIC ICE LOAD

B : BACKFILL PRESSURE

U_{PQ} : UPLIFT PRESSURE (POST-SEISMIC)

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	RETAINING WALL UNUSUAL LOADING CASE - I4
AECOM	PROJECT : 6024-5720 / DATE : March 2013 / figure 14



E1 - EARTHQUAKE

$D + H_1 + B + U + Q$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

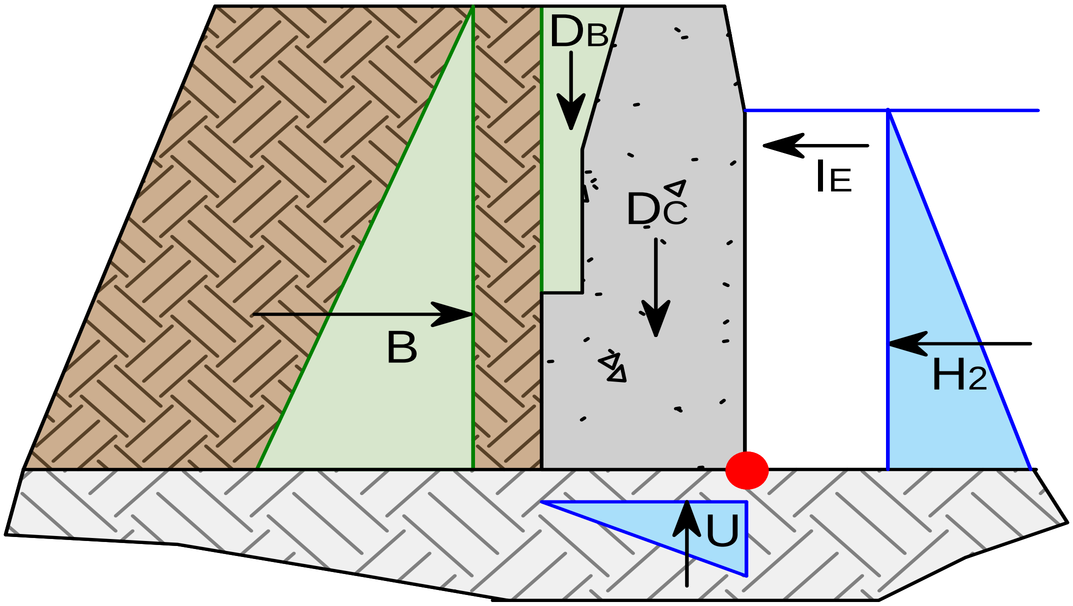
H_1 : HYDROSTATIC PRESSURE IN SUMMER

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Q : EARTHQUAKE LOAD

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
AECOM	RETAINING WALL EXTREME LOADING CASE - E1
	PROJECT : 6024-5720 | DATE : March 2013 | figure 15



E2 - MAXIMUM OPERATING LEVEL IN WINTER WITH EXTREME STATIC ICE LOAD

$D + H_2 + I_E + B + U$

D_B : WEIGHT OF THE BACKFILL

D_C : WEIGHT OF THE CONCRETE

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_E : EXTREME STATIC ICE LOAD

B : BACKFILL PRESSURE

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada

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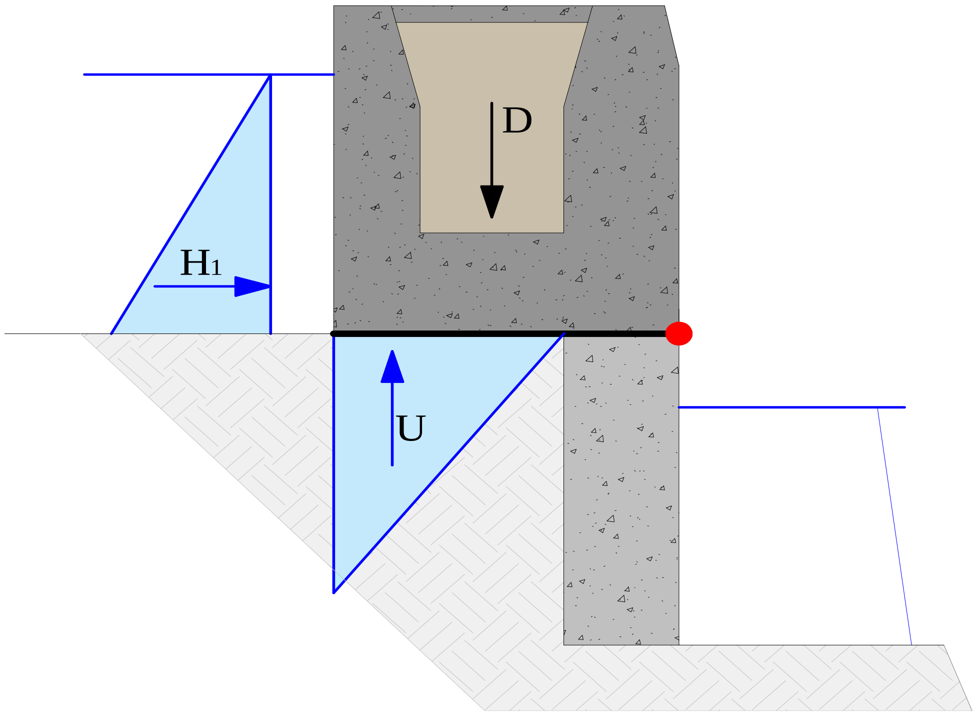
AECOM

PORT SEVERN BUNDLE

RETAINING WALL

EXTREME CASE LOAD - E2

PROJECT : 6024-5720	DATE : March 2013	figure 16



N1 - MAXIMUM OPERATING LEVEL IN SUMMER

$D + H_1 + U$

D : WEIGHT OF THE CONCRETE AND FILL

H_1 : HYDROSTATIC PRESSURE IN SUMMER

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada

Public Works and Government Services Canada

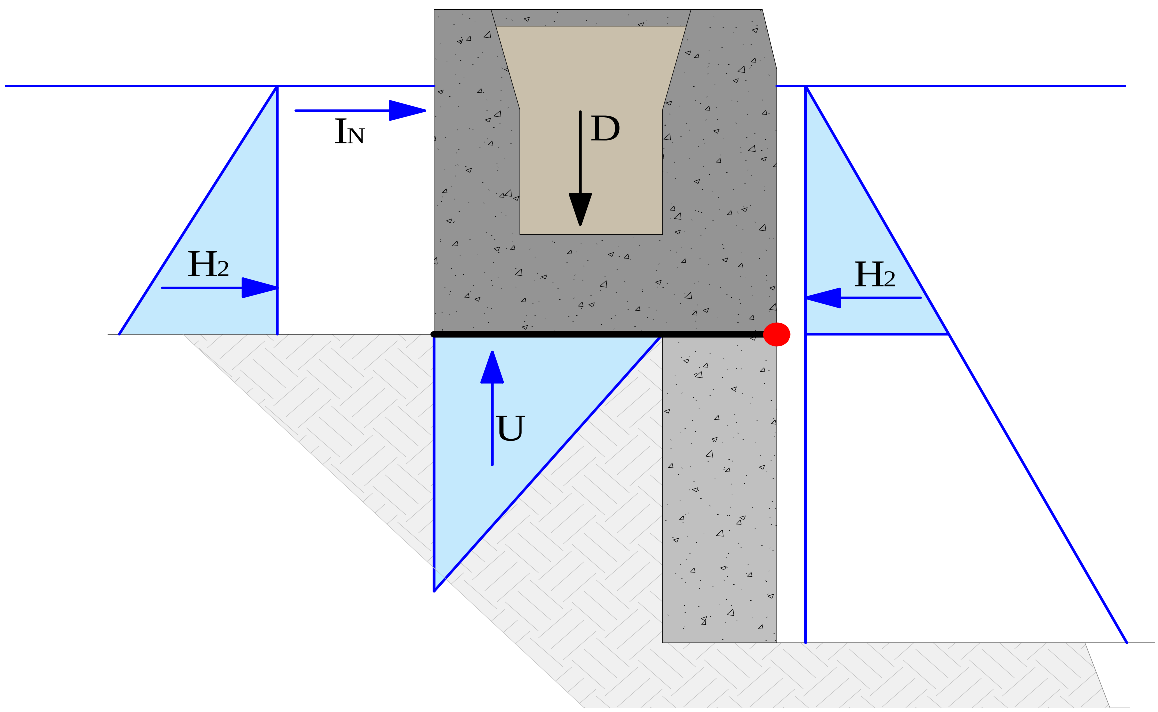
AECOM

PORT SEVERN BUNDLE

LOCK 45 - LEFT WALL

NORMAL LOADING CASE - N1

PROJECT : 6024-5720	DATE : July 2013	figure 17



N2- MAXIMUM OPERATING LEVEL IN WINTER WITH NORMAL STATIC ICE LOAD

$D + H_2 + I_N + U$

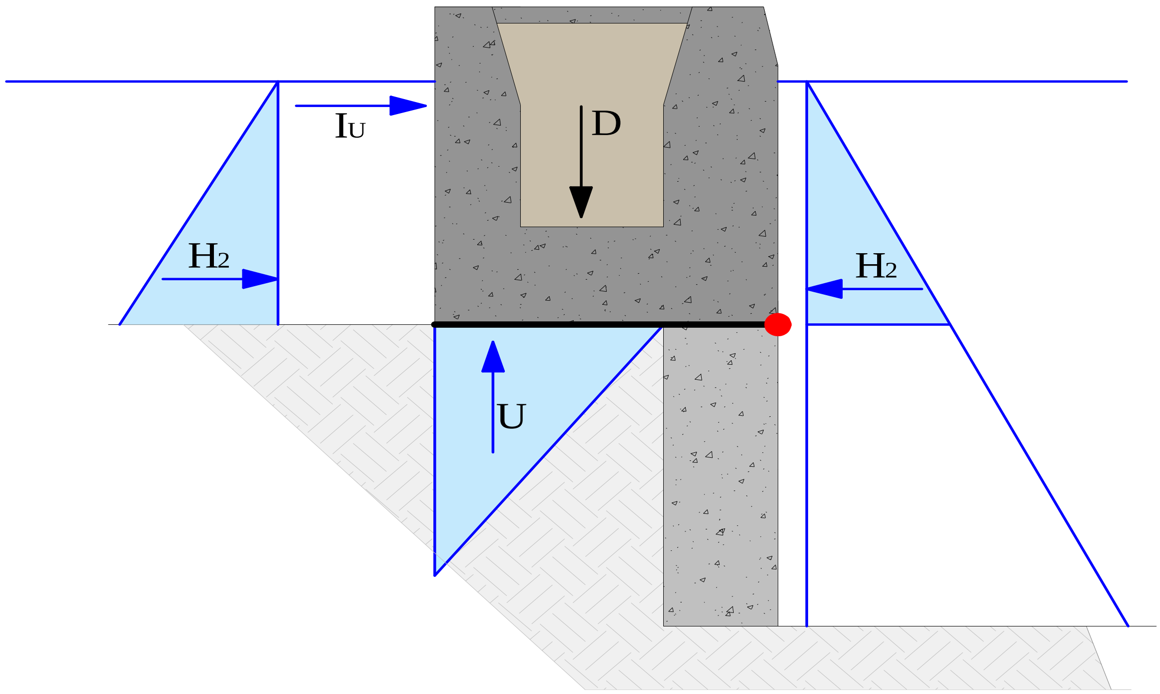
D : WEIGHT OF THE CONCRETE AND FILL

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_N : NORMAL STATIC ICE LOAD

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**	
AECOM	LOCK 45 - LEFT WALL NORMAL LOADING CASE - N2	
	PROJECT : 6024-5720	DATE : July 2013	figure 18



I1- MAXIMUM OPERATING LEVEL IN WINTER WITH UNUSUAL STATIC ICE LOAD

$D + H_2 + I_U + U$

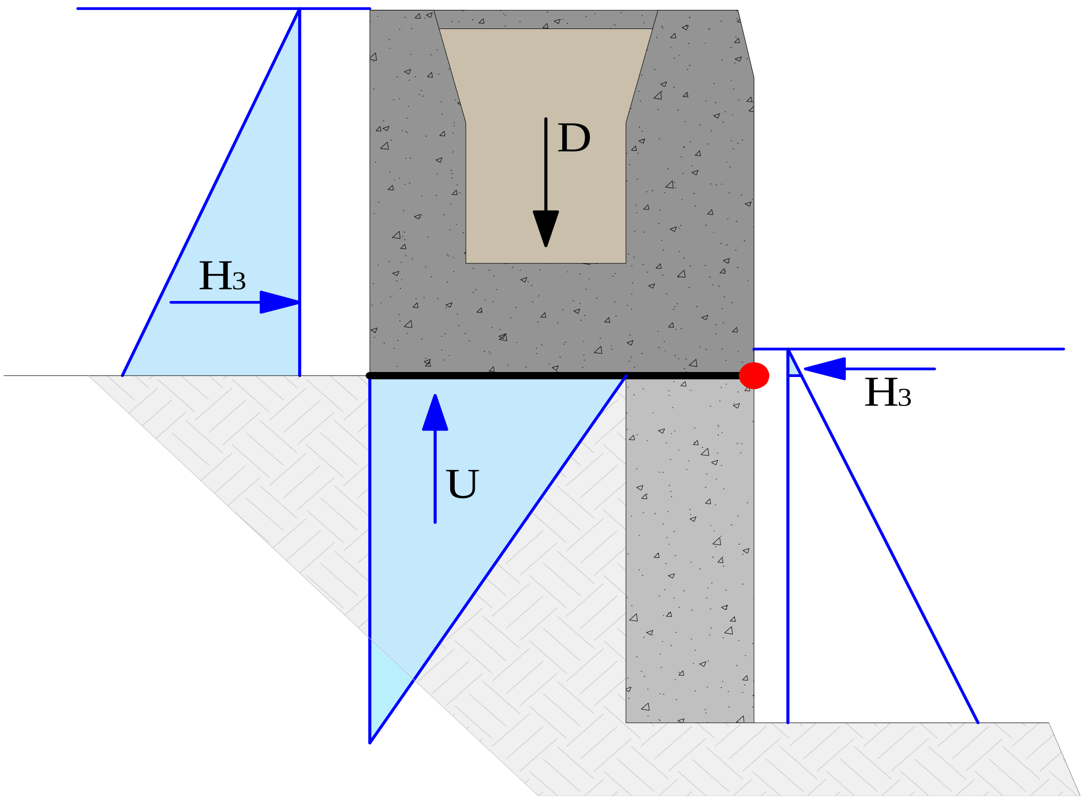
D : WEIGHT OF THE CONCRETE AND FILL

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_U : UNUSUAL STATIC ICE LOAD

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	LOCK 45 - LEFT WALL UNUSUAL LOADING CASE - I1
AECOM	PROJECT : 6024-5720 | DATE : July 2013 | figure 19



I2 - INFLOW DESIGN FLOOD LEVEL (IDF)

$D + H_3 + U$

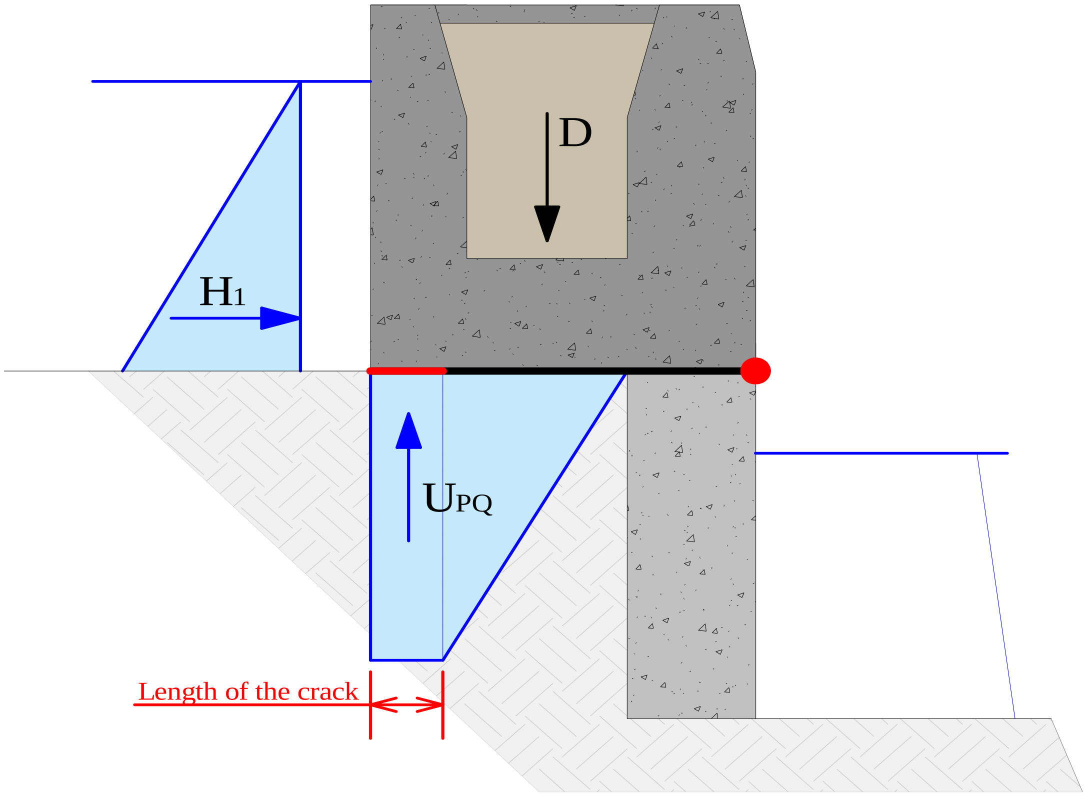
D : WEIGHT OF THE CONCRETE AND FILL

H_3 : HYDROSTATIC PRESSURE WITH IDF

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada — Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	LOCK 45 - LEFT WALL
AECOM	UNUSUAL LOADING CASE - I2

PROJECT : 6024-5720 | DATE : July 2013 | figure 20



I3 - POST-SEISMIC ANALYSIS OF N1

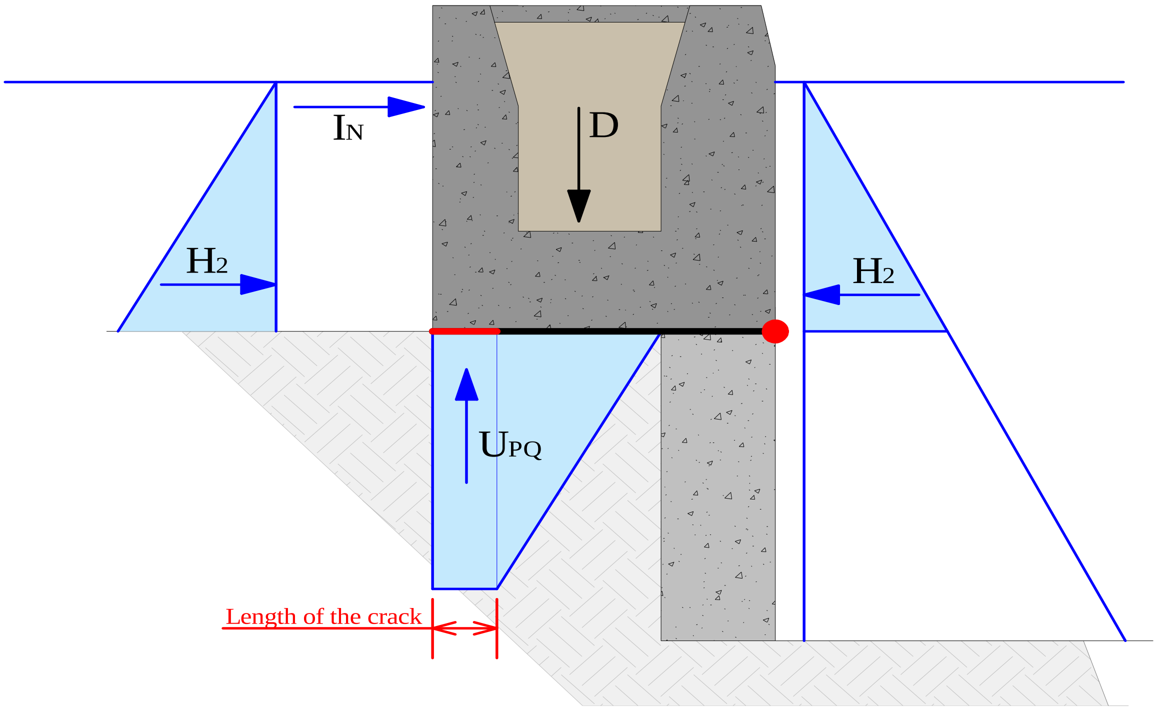
$D + H_1 + U_{PQ}$

D : WEIGHT OF THE CONCRETE AND FILL

H_1 : HYDROSTATIC PRESSURE IN SUMMER

U_{PQ} : UPLIFT PRESSURE (POST-SEISMIC)

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**	
	LOCK 45 - LEFT WALL UNUSUAL LOADING CASE - I3	
AECOM	PROJECT : 6024-5720	DATE : July 2013	figure 21



I4 - POST-SEISMIC ANALYSIS OF N2

$D + H_2 + I_N + U_{PQ}$

D :	WEIGHT OF THE CONCRETE AND FILL
H_2 :	HYDROSTATIC PRESSURE IN WINTER
I_N :	NORMAL STATIC ICE LOAD
U_{PQ} :	UPLIFT PRESSURE (POST-SEISMIC)

Travaux publics et Services gouvernementaux Canada

Public Works and Government Services Canada

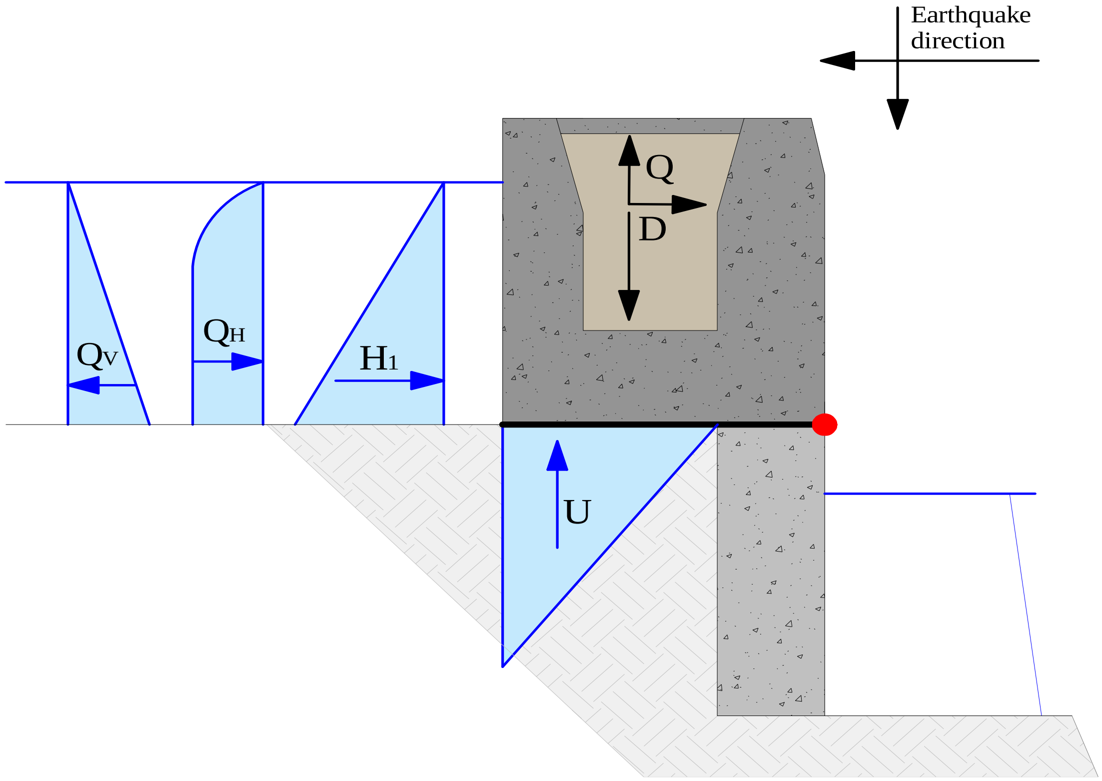
AECOM

PORT SEVERN BUNDLE

LOCK 45 - LEFT WALL

UNUSUAL LOADING CASE - I4

PROJECT : 6024-5720 | DATE : July 2013 | figure 22



E1 - EARTHQUAKE

$D + H_1 + U + Q$

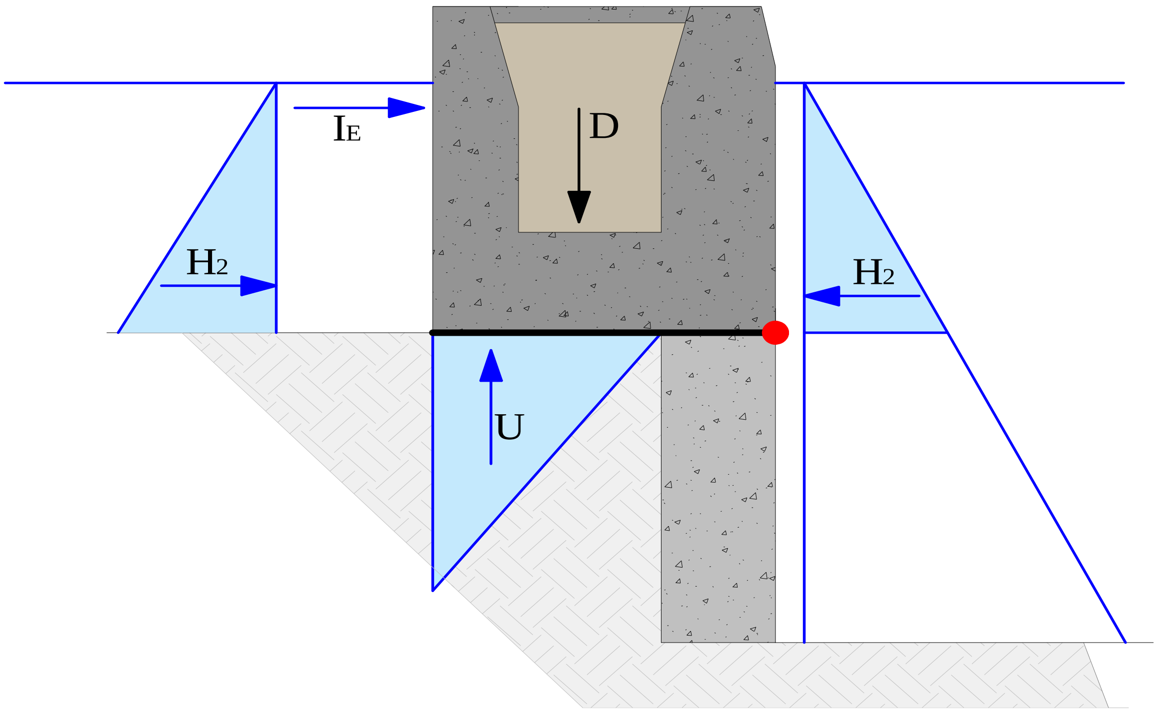
D : WEIGHT OF THE CONCRETE AND FILL

H_1 : HYDROSTATIC PRESSURE IN SUMMER

U : UPLIFT PRESSURE

Q : EARTHQUAKE LOAD

Travaux publics et Services gouvernementaux Canada / Public Works and Government Services Canada	**PORT SEVERN BUNDLE**
	LOCK 45 - LEFT WALL EXTREME LOADING CASE - E1
AECOM	PROJECT : 6024-5720	DATE : July 2013	figure 23



E2 - MAXIMUM OPERATING LEVEL IN WINTER WITH EXTREME STATIC ICE LOAD

$D + H_2 + I_E + U$

D : WEIGHT OF THE CONCRETE AND FILL

H_2 : HYDROSTATIC PRESSURE IN WINTER

I_E : EXTREME STATIC ICE LOAD

U : UPLIFT PRESSURE

Travaux publics et Services gouvernementaux Canada

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AECOM

PORT SEVERN BUNDLE

LOCK 45 - LEFT WALL

EXTREME LOADING CASE - E2

PROJECT : 6024-5720	DATE : July 2013	figure 24

Appendix F

Geotechnical/Concrete Investigation Report (Geo-Logic Inc., July 2012)

CONCRETE INVESTIGATION
DAMS AND LOCK 45 – TRENT SEVERN WATERWAY
PORT SEVERN, ONTARIO
PROJECT NO. G023781 B1

Prepared for
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JULY, 2012

<u>TABLE OF CONTENTS</u>

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CONCRETE INVESTIGATION
DAMS AND LOCK 45 – TRENT SEVERN WATERWAY
PORT SEVERN, ONTARIO
PROJECT NO. G023781 B1

1.0 INTRODUCTION

This report describes the results of a concrete coring investigation conducted along the Trent-Severn Waterway in Port Severn, Ontario, for the purpose of characterizing the concrete through core samples taken from various dams and a lock structure and providing the associated laboratory testing to provide assistance for AECOM to complete a safety review of Dam's C, D, E, G and Lock 45. Geo-Logic Inc. (Geo-Logic) was retained by AECOM (the Client) to conduct this work, in accordance with our proposals dated January 30 and February 5, 2012 and referenced as Geo-Logic proposal numbers PG-1139A/ 1139B.

2.0 PURPOSE AND SCOPE

The purpose of the investigation was to establish the characteristics and physical properties of the concrete structures to aid in the safety review. The required scope of work is based on the requirements provided by Mr. Martin Laflamme of AECOM.
The coring investigation consisted of the following coring sizes and depths at the locations listed:
1) Dam C: 3 cores to 0.6m depth, 100 mm diameter, 1 HQ core deep to 4.0m:
2) Dam D: 1 core to 0.6 m depth, 100 mm diameter, 1 HQ core deep 1.4 m;
3) Dam E: 1 core to 0.6 m depth, 100 mm diameter, 1 HQ core deep to 3.2m;
4) Dam G: 3 cores to 0.6 m depth, 100 mm diameter, 1 HQ core deep to 2m;
5) Upstream end of Lock 45-1 core to 0.6 m depth, 100 mm diameter, 1 HQ core deep to 3m

Once the Health and Safety Plan and Environmental Protection Plan were approved the coring work was carried out at the previously referenced locations.

1. Field notes during the core process were taken and core samples were visually inspected in the field and sealed in a core box and transported to our Peterborough laboratory for further inspection and laboratory testing.
2. The cored test holes were reinstated with Sika 212 non-shrink grout upon completion of the fieldwork in general compliance with our repair outline.
3. Laboratory analysis of materials encountered was performed, including uniaxial compression of concrete, mass density and Alkali Silica Reaction Gel and carbonization.
4. Preparation of a report to summarize findings and provide recommendations for geotechnical design of rehabilitation/ replacement strategy for the dam and lock structures.

3.0 SITE CONDITIONS

The Port Severn portion of the Trent Severn Waterway consists of eight dams, Dam A, B, C, D, E, G and the Main dam. Lock 45 provides passage between Little Lake and Georgian Bay and is located adjacent to the west side of the Main dam.

4.0 FIELD AND LABORATORY WORK PROGRAM

The concrete dams and lock structure were investigated by extending 100mm diameter cores for the shallow coreholes and 63.5mm diameter HQ cores for the deep coreholes. The upper (~0.6m) portions of the deep coreholes were first cored with a 100mm diameter core barrel and then switched to HQ diameter cores to the depth of termination. The core locations were selected by AECOM with final positioning conducted by Geo-Logic during the field investigation based on site conditions and coring apparatus positioning. The fieldwork was conducted during the period of April 2 through 5, 2012. Drilling was conducted by Geo-Logic using portable electric coring equipment. Some of the target depths of the cores could not be reached due to entry of the coring water with associated concrete shards and particles into the water course through fractures in the dam. The silica gel was placed in the hole but entry was still realized upon restarting the coring and the operation was ceased at the hole location.

The cores were returned to our laboratory for closer examination and testing as outlined in the terms of reference by the Client to include uniaxial compression of concrete, mass density and Alkali Silica Reaction Gel (ASR Gel) and carbonization.

After the date indicated on this report, the core samples will be stored in our laboratory for a period of six months and then disposed of, unless indicated otherwise.

5.0 RESULTS

5.1 Dam C

The concrete in the dam was investigated by advancing four (4) coreholes (CH-C01, CH-C02, CH-C03 and CH-C04) along the top of the dam. The corehole locations are indicated on the drawing, Plate 1.

Coreholes CH-C01, CH-C02 and CH-C04 were all shallow coreholes extending to a depth between 0.5 to 0.62m. Corehole CH-C03 was a deep corehole and was terminated at 4.0m depth (proposed to a depth of 5.6m) due to a debris blockage and loss of drilling water into the water course.

The concrete samples retrieved from the coreholes were found to have a light grey cement paste with rounded coarse aggregate of igneous and metamorphic origin. The cement paste was visually noted to be porous, be easily friable with a metal tool and generally surround and adhere to the larger aggregate. The larger aggregate sizes ranged from 30 to 100mm in diameter within the core samples. At most of the fracture zones a white chalky paste was observed being built up on either sides of the sample.

Within Corehole CH-C02 an intact horizontal fracture was also noted at 90mm depth from the surface, with the fracture continuing through some of the larger aggregate indicating the cracking occurred prior to the deterioration of the cement.

Within Corehole CH-C03 a large ~100mm diameter rounded coarse aggregate (gneiss) almost fully bisected the core at 300mm depth from the surface. Vertical fractures extended down from the base of this large aggregate some 50 to 80mm terminating within the 30 and 40mm diameter aggregate below. A secondary large diameter aggregate was contacted at 550mm depth from the surface. The metamorphic rock (schist) bisected the complete core diameter and contained small portions of pyrite crystals which can be expansive when exposed to weathering.

In Corehole CH-C03, similar large diameter rounded coarse aggregates were found up to a depth of about 1.5 m. Frequent “softer” zones where the coring rate increased were reported to be encountered at this location during the investigation, likely corresponding to the sections of the core that contained fewer large diameter aggregates. Some soil particles were observed in the spoil/ cutting water during coring beyond a 1.5m depth, the side of the core was stained along the fracture vertical at 1.5 to 1.6m depth. Small wood fragments were noted within the core from 2.5 to 3.7m depth and a section of concentrated smaller aggregates was noted from 3.2 to 3.4m depth.

At the termination depth of Corehole CH-C04 two large diameter aggregates measuring some 80 and 90mm were contacted. Upon extraction both aggregates separated from the cement paste on the interior portion of the core. In flow of the coring water into the water course was noted when reaching the 4 m depth.

Corehole photographs can be found in Appendix C.

<u>Number of Fractures within the Retrieved Core Samples</u>

Corehole Location	Total Length	Number of Fractures
CH-C01	620mm	1
CH-C02	500mm	1
CH-C03	4000mm	12
CH-C04	550mm	2

Laboratory Test Results for Dam C

Corehole Location	Depth from Surface	Type of Test	Lab No.	Results
CH-C01	0 to 42mm	Carbonation	CST-12-32	No observable evidence of carbonation
CH-C01	420mm	Compression	CST-12-33	Density 2275 kg/m^3, Strength 31.9 MPa
CH-C02	250mm	Compression	CST-12-35	Density 1752 kg/m^3, Strength 19.1 MPa
CH-C03	0-150mm	Compression	CST-12-34	Density 2141 kg/m^3, Strength 16.8 MPa
CH-C03	2900mm	Compression	CST-12-36	Density 2301 kg/m^3, Strength 24.0 MPa
CH-C03	2100mm	ASR Gel	CST-12-37	Absence of ASR in aggregates

5.2 Dam D

The concrete in the dam was investigated by advancing two (2) coreholes (CH-D01 and CH-D02) along the top of the dam. The corehole locations are indicated on the drawing, Plate 1.

Corehole CH-D01 was a shallow corehole extending to a depth of 0.65m. During the drilling of Corehole CH-D02 beyond the upper 0.7m drill spoil/ cuttings were washing out from cracks within the concrete. Bentonite pellets were utilized to plug and block the spoil/ cuttings from entering the water. Corehole CH-D02 was terminated at 1.4m depth when drill water circulation was lost likely due to a void. An attempt to block the void with further use of bentonite was made with a second attempt at drilling resuming the following day but drill water recovery was not obtainable.

The concrete samples retrieved from the coreholes were found to have a light grey cement paste with rounded coarse aggregate of igneous and metamorphic origin. The cement paste was visually noted to be porous, friable with metal tool and generally surround and adhere to the large rounded aggregate. The larger aggregate sizes ranged from 30 to 120mm in diameter within the core samples.

Within Corehole CH-D01 a large ~120mm diameter rounded aggregate was found at the end of the core. Horizontal intact partial fractures were observed at 120 and 300mm depth from the surface.

Within the upper portion of Corehole CH-D02 a wood fragment was noted at 120mm depth and a nominal coarse aggregate size of 30 to 40mm was found. Between 340 and 550mm depth from the surface, aggregate sizes of ~70 to 100mm diameter were found. The 100mm aggregate contacted between 450 and 550mm depth almost fully bisected the core.

It is it noted that fractures beyond the 0.7m depth at this location are present as coring water was noted flowing from behind the rear side of the wall and out the front of the wall. The fractures at 750mm depth and 900mm depth were associated with drill water losses.

Corehole photographs can be found in Appendix C.

Number of Fractures within the Retrieved Core Samples

Corehole Location	Total Length	Number of Fractures
CH-D01	650mm	2
CH-D02	1400mm	7

Laboratory Test Results for Dam D

Corehole Location	Depth from Surface	Type of Test	Lab No.	Results
CH-D01	0 to 41mm	Carbonation	CST-12-52	No observable evidence of carbonation
CH-D01	350mm	Compression	CST-12-53	Density 2182 kg/m^3, Strength 13.3 MPa
CH-D02	1200mm	Compression	CST-12-54	Density 2136 kg/m^3, Strength 12.1 MPa
CH-D02	900mm	ASR Gel	CST-12-55	Absence of ASR in aggregates

5.3 Lock 45

The concrete for the upstream end of the lock was investigated by advancing two (2) coreholes (CH-L45-01 and CH-L45-02) west of the channel and along the north side of the lock wall. The corehole locations are indicated on the drawing, Plate 1.

Corehole CH-L45-01 was a shallow corehole extending to a depth of 0.66m. Corehole CH-L45-02 was a deep corehole and was extended to 3.0m depth.

The upper 240 and 200mm of the concrete from the Coreholes CH-L45-01 and CH-L45-02 comprised a different concrete aggregate makeup than the concrete encountered below this depth. The concrete in these upper portions visually appeared to be less porous, contained a greater percentage of coarse aggregates, included a range in aggregate gradation with nominal size being around 19mm and the aggregates were noted to include more angular particles than that of the remaining cores. It is surmised that the surficial concrete encountered in the cores was newer concrete that was placed as a patch/ repair. Within Corehole CH-L45-02 a 6mm round piece of steel was contacted at 85mm depth from the surface. This steel could be part of a reinforcing wire mesh. The concrete samples retrieved from the coreholes were found to have a light grey cement paste. The underlying concrete was found to contain mainly large rounded aggregate of igneous and metamorphic origin and the cement paste was visually noted to be porous with good coverage and adherence to the aggregate surface. Both of the first fractures noted at around 250 and 220mm depth occurred in the underlying more porous concrete rather than the upper patched/ repaired concrete. The larger aggregate sizes ranged from 30 to 120mm in diameter within the lower portion of the core samples.

Within Corehole CH-L45-02 at 240mm depth an 80 to 90mm thick piece of gneiss bisected the core sample. Along the upper portion of the gneiss and within the highly fractured zone just below the upper concrete, some white chalky paste was observed along the aggregate and fractures. A secondary piece of large aggregate (gneiss) bisected the core sample at 1030 to 1150mm depth from the surface. At 2.8m depth a piece of metamorphic rock (schist) was contacted along the side of the core and contained small portions of pyrite.

Corehole photographs can be found in Appendix C.

<u>Number of Fractures within the Retrieved Core Samples</u>

Corehole Location	Total Length	Number of Fractures
CH-L45-01	660mm	2
CH-L45-02	3000mm	15

<u>Laboratory Test Results for Lock 45</u>

Corehole Location	Depth from Surface	Type of Test	Lab No.	Results
L45-01	0 to 60mm	Carbonation	CST-12-38	No observable evidence of carbonation
L45-01	400mm	Compression	CST-12-39	Density 2097 kg/m^3, Strength 14.3 MPa
L45-02	2600mm	Compression	CST-12-40	Density 2180 kg/m^3, Strength 14.6 MPa
L45-02	1200mm	ASR Gel	CST-12-41	Absence of ASR in aggregates

5.4 Dam E

The concrete in the dam was investigated by advancing two (2) coreholes (CH-E01 and CH-E02) along the top of the dam. The corehole locations are indicated on the drawing, Plate 1.

Corehole CH-E01 was a shallow corehole extending to a depth of 0.7m. Corehole CH-E02 was a deep corehole and was extended to 3.2m depth.

The concrete samples retrieved from the coreholes were found to have a light grey cement paste with rounded coarse aggregate of igneous and metamorphic origin. The cement paste was visually noted to be porous, friable and generally surround and ahere to the large rounded aggregate. The larger aggregate sizes ranged from 30 to 110mm in diameter within the core samples. Within the fracture zones of Corehole CH-E01, some white chalky paste was observed being built up on either sides of the fracture.

Within Corehole CH-E01 micro fractures were observed around several of the larger aggregates at around 150mm depth from the surface. A large ~100mm diameter rounded aggregate was found at the end of the core run and fully bisected the core.

Within the upper portion of Corehole CH-E02 a partial vertical intact fracture was found originating from the surface to 80mm depth. A small piece of metal, likely a nail or bolt was contacted at 790mm depth and again at the end of the core run within Corehole CH-E02. A piece of wood possibly from the original formwork was also contacted along with another nail or bolt was noted within the remaining 150mm of the 3.2m core run. Since the date of obtaining the core sample, separation of the wood from the surrounding concrete was noted due to loss of moisture and shrinkage of the piece of wood. At around 2m depth increased friction from loose debris during drilled was encountered. The fractures in this region are noted to be rounded off at the edges for this 75mm long area.

Corehole photographs can be found in Appendix C.

Number of Fractures within the Retrieved Core Samples

Corehole Location	Total Length	Number of Horizontal Fractures
CH-E01	700mm	2
CH-E02	3200mm	14

Note: One of the fractures in CH-E02 was not included as it was made during sampling in order for the retrieved sample to fit into the core box.

Laboratory Test Results for Dam E

Corehole Location	Depth from Surface	Type of Test	Lab No.	Results
CH-E01	0 to 50mm	Carbonation	CST-12-48	Observable evidence of carbonation to 4mm
CH-E01	400mm	Compression	CST-12-49	Density 2190 kg/m^3, Strength 16.2 MPa
CH-E02	2800mm	Compression	CST-12-50	Density 2162 kg/m^3, Strength 8.0 MPa
CH-E02	2470mm	ASR Gel	CST-12-51	Absence of ASR in aggregates

5.5 Dam G

The concrete in the dam was investigated by advancing four (4) coreholes (CH-G01, CH-G02, CH-G03 and CH-G04) along the top of the dam. The corehole locations are indicated on the drawing, Plate 2.

Coreholes CH-G01, CH-G02 and CH-G03 were all shallow coreholes extending to a depth between 0.60 to 0.63m. Corehole CH-C04 was a deep corehole and was terminated at 2.0m depth.

The upper 240 and 270mm of the concrete encountered in the Coreholes CH-G03 and CH-G04 was comprised of a different structure than the concrete encountered below this depth. The concrete in these upper portions was found to be less porous, contained a higher percentage of coarse aggregates, included a range in aggregate gradation with nominal size being around 19mm and the coarse aggregates were noted to include more angular particles than that of the remaining cores. It is possible that these upper portions were newer concrete that was placed as a patch/ repair. Within Corehole CH-G04, a 10M piece of reinforcing steel was contacted at 120mm depth from the surface. This steel could be part of dowel placed at the time of the patch/ repair. The concrete samples retrieved from the coreholes were found to have a light grey cement paste. The underlying concrete was found to contain mainly large rounded coarse aggregate of igneous and metamorphic origin and the cement paste was visually noted to be porous. It is noted that the first fracture in Corehole CH-G03 at around 290mm depth occurred in the underlying more porous concrete rather than the upper patched/ repaired concrete. The first fracture noted within Coreholes CH-G04 occurred on a diagonal from 200 to 270mm depth within the upper patched/ repaired concrete. In the underlying concrete and at the other core locations the larger aggregate sizes ranged from 30 to 140mm in diameter.

Within Corehole CH-G01, intact horizontal fractures were noted at around 150 and 250mm depth from the surface. A partial vertical fracture was noted extending to a depth of 40mm from the surface. At most of the fracture zones a white chalky paste was observed being built up on either sides of the sample. The similar white paste clearly outlined the aggregate within the first horizontal fracture. Over the 620mm length of core taken, five large diameter aggregates between 70 to 140mm were noted.

Within Corehole CH-G02 an intact horizontal fractures zones were noted at 100mm and 180 to 200mm depth from the surface. The fractures continued through most of the larger aggregates. Chalky white paste was observed at some of the edges of the fractures. At 260mm depth a cold joint was found. Some yellow staining noted on the faces of the cold joint. The larger aggregate within the core ranged from 30 to 50mm.

Within Corehole CH-G04 at 1.1m depth below the surface, soil was contacted within the fracture. An associated loss of the return in drilling water, soil particles within the cuttings and staining and soil deposits on the retrieved core sample confirms this fracture contained soil. At the termination depth of core at large 140mm diameter piece of metamorphic rock (gneiss) was contacted. To remove the core at 2.0m depth little to no effort was required to break the core. Based on the rock line profile shown on the provided as built drawing C-11-3380, dated 1918 it is possible that at 2m depth bedrock maybe what was contacted. However given the similar composition and diameter of aggregates found within the core samples this cannot be confirmed.

Corehole photographs can be found in Appendix C.

Number of Fractures within the Retrieved Core Sample

Corehole Location	Total Length	Number of Horizontal Fractures
CH-G01	620mm	4
CH-G02	600mm	2 (cold joint also present)
CH-G03	630mm	1
CH-G04	2000mm	9

Note: One of the fractures in CH-G04 was not included as it was made during sampling in order for the retrieved sample to fit into the core box.

Laboratory Test Results for Dam G

Corehole Location	Depth from Surface	Type of Test	Lab No.	Results
CH-G01	0 - 106mm	Carbonation	CST-12-42	No observable evidence of carbonation
CH-G02	300mm	Compression	CST-12-43	Density 1931 kg/m^3
CH-G03	75mm	Compression	CST-12-44	Density 2142 kg/m^3, Strength 24.1 MPa
CH-G04	300mm	Compression	CST-12-45	Density 2156 kg/m^3, Strength 13.3 MPa
CH-G04	1400mm	Compression	CST-12-46	Density 2123 kg/m^3, Strength 14.9 MPa
CH-G04	1720mm	ASR Gel	CST-12-47	Absence of ASR in aggregates

Note: A failure of the compression testing apparatus resulted in erratic reading without an obtainable compressive strength for CH-G02, CST-12-43.

6.0 SUMMARY

Based on the corehole investigation, Geo-Logic offers the following comments:

1. The compressive strength of the concrete tested from within the core samples ranged from 8.0 to 31.9 MPa with the average strength being around 16 MPa. The corresponding average density was found to be 2126 kg/m^3. As noted within the individual summaries, large diameter aggregates were prominent in the majority of the core samples. The increased gaps within the aggregate gradation and lack of angular aggregates in the majority of the concrete would correlate to the lower compressive strength found. The greatest range in densities at an individual location was found from the samples taken from Dam C and Lock 45. The average compressive strength found indicates possible higher water to cement ratio was used within the original concrete and that the strength versus time is likely on a declining trend.
2. With the exception of the testing conducted at Dam E, carbonation was not present in the surface samples tested. For Corehole CH-E01 after the addition of Phenolphthalein, the concrete section was observed to show a light pink colouration rather than a dark pink colouration up to a depth of 4 mm. The concrete was concluded to have some minimal carbonation depth.
3. The aggregate in the concrete was found to be free of dolomitic aggregate normally associated with Alkali Aggregate Reactivity (AAR) but some of the metamorphic rock did have some glassy silicates present. Within the samples tested, signs of the solution revealing reaction rims around the aggregates or the addition of the red reagent exposing a pink tinge confirming the presence of Alkali Silicate Reaction gel were not found indicating a low risk for reactivity from the aggregates present. The low strength of the concrete is more likely natural weathering and/or high water cement ratio in the concrete.
4. As detailed in the previous section, fracturing was present within the coreholes, a void was contacted at 1.4m depth in Corehole CH-D02, an infilled fracture with soil was contacted in Corehole CH-G04 and a cold joint was contacted Corehole CH-G02. These fractures, void and cold joint indicate the concrete as a whole is no longer acting as a homogeneous system. During the investigation, from the samples recovered and based on the review of the site photos is apparent that fractures, repairs and misalignment of the concrete are present at each of the structure locations.
5. Due to the deteriorating nature of the cement and aggregates in the concrete, its ability to resist weathering is decreasing rapidly with time. The permeability is increasing while the strength is decreasing with time. The option of reconstruction of the structures as opposed to patching is seen as the more effective maintenance strategy in the future.

We trust that the contents of this report meet with your immediate requirements. Should you have any questions or concerns regarding any aspect of this report, or should you require any further assistance, please do not hesitate to contact our office.

Sincerely yours,
Geo-Logic Inc.
GEOTECHNICAL ENGINEERS
AND HYDROGEOLOGISTS

Mark Narduzzi, P. Eng.
Project Engineer

Andy Fawcett, P. Eng.
Senior Engineer

MN/AF/mn

7.0 STATEMENT OF LIMITATIONS

This report is intended solely for AECOM and Parks Canada and other parties explicitly identified in the report and is prohibited for use by others without Geo-Logic's prior written consent. This report is considered Geo-Logic's professional work product and shall remain the sole property of Geo-Logic. Any unauthorized reuse, redistribution of or reliance on the report shall be at the Client and recipient's sole risk, without liability to Geo-Logic. Client shall defend, indemnify and hold Geo-Logic harmless from any liability arising from or related to Client's unauthorized distribution of the report. No portion of this report may be used as a separate entity; it is to be read in its entirety and shall include all supporting drawings and appendices.

The recommendations made in this report are in accordance with our present understanding of the project, the current site use, ground surface elevations and conditions, and are based on the work scope approved by the Client and described in the report. The services were performed in a manner consistent with that level of care and skill ordinarily exercised by members of geotechnical engineering professions currently practicing under similar conditions in the same locality. No other representations, and no warranties or representations of any kind, either expressed or implied, are made. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties.

All details of design and construction are rarely known at the time of completion of a geotechnical study. The recommendations and comments made in the study report are based on our subsurface investigation and resulting understanding of the project, as defined at the time of the study. We should be retained to review our recommendations when the drawings and specifications are complete. Without this review, Geo-Logic will not be liable for any misunderstanding of our recommendations or their application and adaptation into the final design.

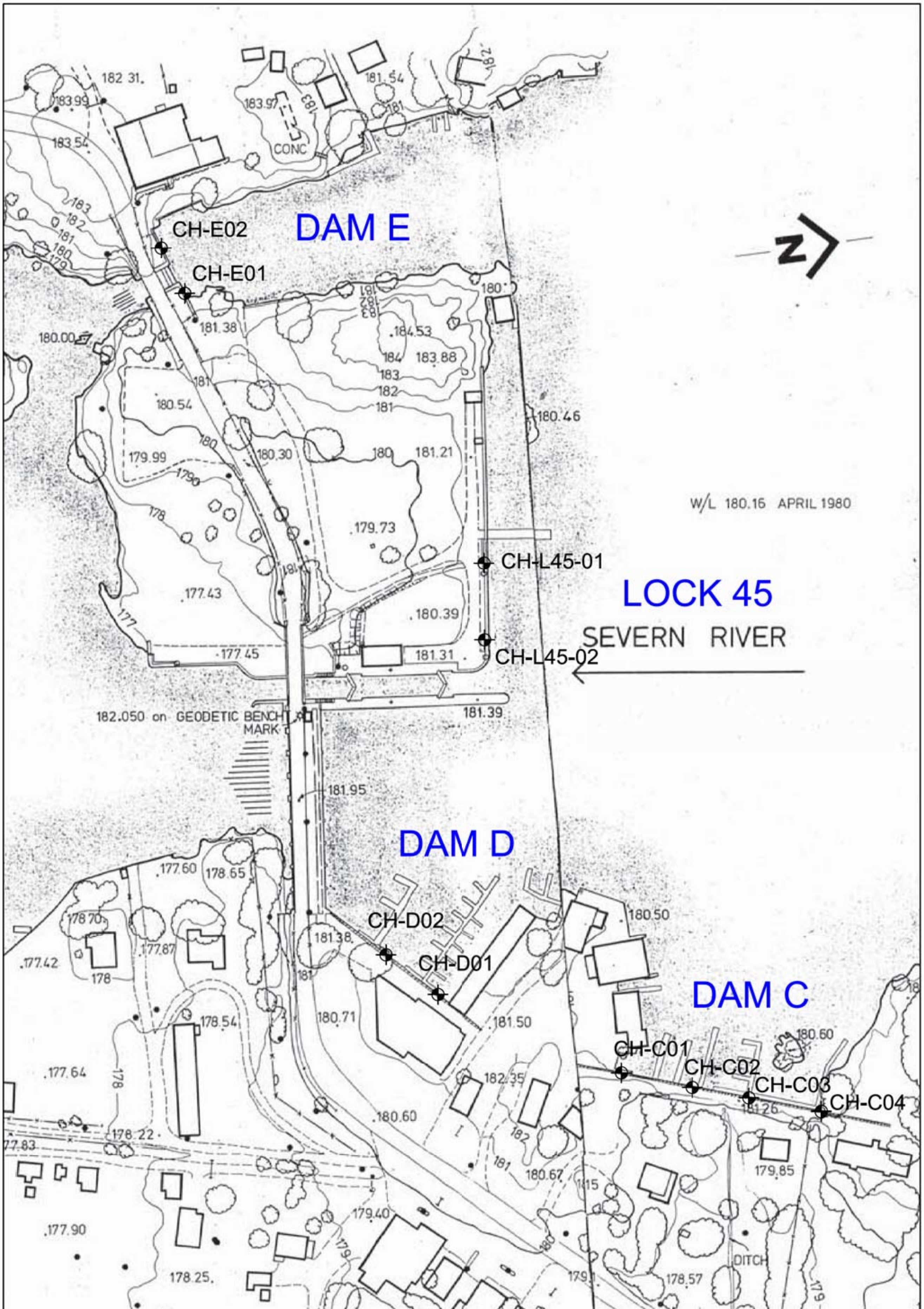
By issuing this report, Geo-Logic is the geotechnical engineer of record. It is recommended that Geo-Logic be retained during construction of all foundations and during earthwork operations to confirm the conditions of the subsoil are actually similar to those observed during our study. The intent of this requirement is to verify that conditions encountered during construction are consistent with the findings in the report and that inherent knowledge developed as part of our study is correctly carried forward to the construction phases.

It is important to emphasize that a corehole investigation is, in fact, a random sampling of a site and the comments included in this report are based on the results obtained at the fourteen (14) corehole locations at five (5) different sites only. The subsurface conditions confirmed at these testhole locations may vary at other locations. The subsurface conditions can also be significantly modified by the construction activities on site (ex. excavation, dewatering and drainage, blasting, pile driving, etc.). These conditions can also be modified by exposure of soils or bedrock to humidity, dry periods or frost. Concrete, soil and groundwater conditions between and beyond the test locations may differ both horizontally and vertically from those encountered at the test locations and conditions may become apparent during construction which could not be detected or anticipated at the time of our investigation. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations. If changed conditions are identified during construction, no matter how minor, the recommendations in this report shall be considered invalid until sufficient review and written assessment of said conditions by Geo-Logic is completed.

APPENDIX A

Plate 1 – Corehole Location Plan - Dams C, D, E and Lock 45

Plate 2 – Corehole Location Plan – Dam G



DAMS C, D, E AND LOCK 45
COREHOLE LOCATION PLAN

PORT SEVERN RD.
TRENT SEVERN WATERWAY
PORT SEVERN, ONTARIO

PROJECT NO. :	G023781-B1
SCALE :	N.T.S
DATE :	JULY 2012
PLATE NO. :	1

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DAM G
COREHOLE LOCATION PLAN

PORT SEVERN RD.
TRENT SEVERN WATERWAY
PORT SEVERN, ONTARIO

N
W
E
S

PROJECT NO. :	G023781-B1
SCALE :	N. T. S.
DATE :	JULY 2012
PLATE NO. :	2

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APPENDIX B

Concrete Compressive Test Report

Carbonation Testing Reports

Alkali Aggregate Reaction Test Reports

347 Pido Road Unit 29
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CONCRETE CORE TEST REPORT

Project: *Dams and Lock 45, Trent Severn Waterway - Port Severn, Ontario*

Project No: *G023781B1*

Client: *AECOM*

Date Rec:

Project Location: *Port Severn Road*

Date Tested: *April 20, 2012*

LABORATORY TEST RESULTS

Core #	Geo-Logic Core #	Length (mm) Ground	Diameter (mm)	Density Kg/m^3	L/D Ratio	L/D Factor	Load (Lbs.)	Strength Corrected MPa
C-01	CST-12-33	141	95	2275	1.48	0.96	235.7	**31.9**
C-03	CST-12-34	186	95	2141	1.96	0.99	120.3	**16.8**
C-02	CST-12-35	147	95	1752	1.55	0.96	140.8	**19.1**
C-03	CST-12-36	98	62	2301	1.58	0.96	75.5	**24.0**
L-01	CST-12-39	123	95	2097	1.29	0.93	109.0	**14.3**
L-02	CST-12-40	117	62	2180	1.89	0.99	44.5	**14.6**
G-02	CST-12-43	136	95	1931	1.43	0.95		*****
G-03	CST-12-44	165	95	2142	1.74	0.98	174.0	**24.1**
G-04	CST-12-45	186	95	2156	1.96	0.99	95.2	**13.3**
G-04	CST-12-46	72	62	2123	1.16	0.91	49.4	**14.9**
E-01	CST-12-49	119	95	2190	1.25	0.96	119.6	**16.2**
E-02	CST-12-50	99	62	2162	1.60	0.96	25.2	**8.0**
D-01	CST-12-53	178	95	2182	1.87	0.98	96.3	**13.3**
D-02	CST-12-54	113	62	2136	1.82	0.98	37.3	**12.1**

REMARKS
*An erratic reading from the testing apparatus did not permit an obtainable compressive strength for CH-G02, CST-12-43.

CORE DEPTHS

Issued By:

Date Issued: *Jul-12*

Geo- *Logic Inc.*

Lab Form CSA-A23.2-14C

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April 23, 2012

Re: Carbonation Testing
Client: AECOM
Project: Dams and Lock 45, Port Severn, ON
Project No.G023781 B1

Core #:	BH-C01
Laboratory #:	CST-12-32
Location:	Dam C, Port Severn
Date: Received: **Date Tested:**	April 17, 2012 April 23, 2012
Diameter of Sample (mm)	92
Depth of Sample (mm)	0-42
Results of Carbonation Test.	After the addition of Phenolphthalein, the entire concrete section changed from a grey to a dark pink colouration.
Conclusion	There was no observable evidence of carbonation present in the core.
Remarks	

Note: Results were determined according to ASTM C856.

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Project: Dams and Lock 45, BH-C03
Project Number: G023781 B1
Structure Element: Dam C
Sample Depth: Vertical Core 67mm diameter, 2100mm depth

General Concrete Comments:

- The observed nominal aggregate size was determined to be 30mm. The core make-up was primarily comprised of larger uncrushed aggregate with trace amounts of uncrushed particles.
- The concrete paste was light grey in colour and relatively easy to break apart with a metal tool.
- The sample did not contain any observable cracks or fractures. The paste was noted visually to be very porous.

ASR Screening Test (SHRP-C-315) Comments:

- After the addition of Uranyl acetate solution, observation under UV light did not reveal any evidence of silica gel product.
- The addition of the red reagent to the sample produced the same result. There was not any observable silica gel product revealed.

Summary

The sample tested had primarily uncrushed aggregates with trace amounts of uncrushed particles, with a nominal size of 30mm. The addition of Uranyl Acid and pink solution did not reveal evidence of the presence of alkali-silica reactivity. In accordance with SHRP-C-315 the above results are indicative of the absence of Alkali-Silica Reactivity in the aggregates present in the concrete.

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April 23, 2012

Re: Carbonation Testing
Client: AECOM
Project: Dams and Lock 45, Port Severn, ON
Project No.G023781 B1

Core #:	BH-D01
Laboratory #:	CST-12-52
Location:	Dam D, Port Severn
Date: Received: **Date Tested:**	April 17, 2012 April 23, 2012
Diameter of Sample (mm)	92
Depth of Sample (mm)	0-41
Results of Carbonation Test.	After the addition of Phenolphthalein, the entire concrete section changed from a grey to a dark pink colouration.
Conclusion	There was no observable evidence of carbonation present in the core.
Remarks	

Note: Results were determined according to ASTM C856.

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Project: Dams and Lock 45, BH-D02
Project Number: G023781 B1
Structure Element: Dam D
Sample Depth: Vertical Core 67mm diameter, 900mm depth

General Concrete Comments:

- The observed nominal aggregate size was determined to be 35mm. The core make-up was primarily comprised of larger uncrushed aggregate with trace amounts of uncrushed particles.
- The concrete paste was light grey in colour and relatively easy to break apart with a metal tool.
- The sample did not contain any observable cracks or fractures. The paste was noted visually to be very porous.

ASR Screening Test (SHRP-C-315) Comments:

- After the addition of Uranyl acetate solution, observation under UV light did not reveal any evidence of silica gel product.
- The addition of the red reagent to the sample produced the same result. There was not any observable silica gel product revealed.

Summary

The sample tested had primarily uncrushed aggregates with trace amounts of uncrushed particles, with a nominal size of 35mm. The addition of Uranyl Acid and pink solution did not reveal evidence of the presence of alkali-silica reactivity. In accordance with SHRP-C-315 the above results are indicative of the absence of Alkali-Silica Reactivity in the aggregates present in the concrete.

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April 23, 2012

Re: Carbonation Testing
Client: AECOM
Project: Dams and Lock 45, Port Severn
Project No.G023781 B1

Core #:	BH-L45-O1
Laboratory #:	CST-12-38
Location:	Lock 45, Port Severn
Date: Received: **Date Tested:**	April 17, 2012 April 23, 2012
Diameter of Sample (mm)	92
Depth of Sample (mm)	0-60
Results of Carbonation Test.	After the addition of Phenolphthalein, the entire concrete section changed from a grey to a dark pink colouration.
Conclusion	There was no observable evidence of carbonation present in the core.
Remarks	

Note: Results were determined according to ASTM C856.

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Project: Dams and Lock 45, BH-L45-02
Project Number: G023781 B1
Structure Element: Lock 45
Sample Depth: Vertical Core 67mm diameter, 1200mm depth.

General Concrete Comments:

- The observed nominal aggregate size was determined to be 31mm. The core make-up was primarily comprised of larger uncrushed aggregate with trace amounts of uncrushed particles.
- The concrete paste was light grey in colour and relatively easy to break apart with a metal tool.
- The sample did not contain any observable cracks or fractures. The paste was noted visually to be very porous.

ASR Screening Test (SHRP-C-315) Comments:

- After the addition of Uranyl acetate solution, observation under UV light did not reveal any evidence of silica gel product.
- The addition of the red reagent to the sample produced the same result. There was not any observable silica gel product revealed.

Summary

The sample tested had primarily uncrushed aggregates with trace amounts of uncrushed particles, with a nominal size of 31mm. The addition of Uranyl Acid and pink solution did not reveal evidence of the presence of alkali-silica reactivity. In accordance with SHRP-C-315 the above results are indicative of the absence of Alkali-Silica Reactivity in the aggregates present in the concrete.

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April 23, 2012

Re: Carbonation Testing
Client: AECOM
Project: Dams and Lock 45, Port Severn, ON
Project No.G023781 B1

Core #:	BH-E01
Laboratory #:	CST-12-48
Location:	Dam E, Port Severn
Date: Received: **Date Tested:**	April 17, 2012 April 23, 2012
Diameter of Sample (mm)	92
Depth of Sample (mm)	0-50
Results of Carbonation Test.	After the addition of Phenolphthalein, the concrete section was observed to have a light pink colouration beginning at the surface and measuring down 4mm. Beyond 4mm the remainder of the core exhibited a dark pink colouration.
Conclusion	There was observable evidence of carbonation present in the core at a depth of 0 to 4mm.
Remarks	

Note: Results were determined according to ASTM C856.

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Project: Dams and Lock 45, BH-E02
Project Number: G023781 B1
Structure Element: Dam E
Sample Depth: Vertical Core 67mm diameter, 2470mm depth

General Concrete Comments:

- The observed nominal aggregate size was determined to be 35mm. The core make-up was primarily comprised of larger uncrushed aggregate with trace amounts of uncrushed particles.
- The concrete paste was light grey in colour and relatively easy to break apart with a metal tool.
- The sample did not contain any observable cracks or fractures. The paste was noted visually to be very porous.

ASR Screening Test (SHRP-C-315) Comments:

- After the addition of Uranyl acetate solution, observation under UV light did not reveal any evidence of silica gel product.
- The addition of the red reagent to the sample produced the same result. There was not any observable silica gel product revealed.

Summary

The sample tested had primarily uncrushed aggregates with trace amounts of uncrushed particles, with a nominal size of 35mm. The addition of Uranyl Acid and pink solution did not reveal evidence of the presence of alkali-silica reactivity. In accordance with SHRP-C-315 the above results are indicative of the absence of Alkali-Silica Reactivity in the aggregates present in the concrete.

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April 23, 2012

Re: Carbonation Testing
Client: AECOM
Project: Dams and Lock 45, Port Severn, ON
Project No.G023781 B1

Core #:	BH-G01
Laboratory #:	CST-12-42
Location:	Dam G, Port Severn
Date: Received: **Date Tested:**	April 17, 2012 April 23, 2012
Diameter of Sample (mm)	92
Depth of Sample (mm)	0-106
Results of Carbonation Test.	After the addition of Phenolphthalein, the entire concrete section changed from a grey to a dark pink colouration.
Conclusion	There was no observable evidence of carbonation present in the core.
Remarks	

Note: Results were determined according to ASTM C856.

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Project: Dams and Lock 45, BH-G04
Project Number: G023781 B1
Structure Element: Dam G
Sample Depth: Vertical Core 67mm diameter, 1720mm depth

General Concrete Comments:

- The observed nominal aggregate size was determined to be 19mm. The core make-up was primarily comprised of larger uncrushed aggregate with trace amounts of uncrushed particles.
- The concrete paste was light grey in colour and relatively easy to break apart with a metal tool.
- The sample did not contain any observable cracks or fractures. The paste was noted visually to be very porous.

ASR Screening Test (SHRP-C-315) Comments:

- After the addition of Uranyl acetate solution, observation under UV light did not reveal any evidence of silica gel product.
- The addition of the red reagent to the sample produced the same result. There was not any observable silica gel product revealed.

Summary

The sample tested had primarily uncrushed aggregates with trace amounts of uncrushed particles, with a nominal size of 19mm. The addition of Uranyl Acid and pink solution did not reveal evidence of the presence of alkali-silica reactivity. In accordance with SHRP-C-315 the above results are indicative of the absence of Alkali-Silica Reactivity in the aggregates present in the concrete.

APPENDIX C

Concrete Core Photographs

Picture 1: Dam C: Concrete core from Corehole CH-C01.

Picture 2: Dam C: Concrete core from Corehole CH-C02.

Picture 3: Dam C: Upper portion of concrete core from Corehole CH-C03.

Picture 4: Dam C: Last section of 100mm diameter concrete core at 550mm depth Corehole CH-C03. A 50mm portion of Schist cobble shown, pyrite present along fracture.

Picture 5: Dam C: Lower portion of the concrete core from Corehole CH-C03.

Picture 6: Dam C: Concrete core from Corehole CH-C04.

Picture 7: Concrete cores, Dam C, chalky white paste on faces of fractures.

Picture 8: Dam D: Concrete core from Corehole CH-D01.

Picture 9: Dam D: Large aggregate at the bottom of the core sample, Corehole CH-D01.

Picture 10: Dam D: Concrete cores from Corehole CH-D02.

Picture 11: Dam E: Concrete core from Corehole CH-E01.

Picture 12: Dam E: Concrete core from Corehole CH-E02.

Picture 13: Lock 45: Concrete core from Corehole CH-L45-01. Upper patch/ repair.

Picture 14: Lock 45: Concrete core from Corehole CH-L45-02. Upper patch/ repair.

Picture 15: Lock 45: Corehole CH-L45-02, photo on the left showing 6mm diameter steel, photo on the right showing aggregate containing pyrite.

Picture 16: Dam G: Concrete core from Corehole CH-G01.

Picture 17: Dam G: Concrete core from Corehole CH-G02. Cold joint shown.

Picture 18: Dam G: Concrete core from Corehole CH-G03. Upper patch/ repair.

Picture 19: Dam G: Concrete core from Corehole CH-G04. Upper portion patch/ repair.

Picture 20: Dam G: Corehole CH-G04, *Left photo,* Upper patch/ repair Corehole CH-G04. 10M reinforcing steel shown. *Right photo,* soil deposits on fracture at 1100mm depth.

Picture 21: Dam G: Corehole CH-G01. Large diameter aggregates shown along with white chalky paste around aggregates. 100mm diameter core.

Appendix G
Ice Loading Criteria Memorandum

Memorandum

Date:	Wednesday, September 21th 2012
Project No.:	0522157-2000
Project:	Dam Safety Review: Port Severn Bundle
To:	Shawn Filion, PWGSC
From:	Annie Dumas, Jeremy Kent-Johnston
Subject:	CEATI Ice Loading Criteria

Distribution:	Jacques Béland, PWGSC	Brett McLellan, PCA
	Bob Nash, PCA	Dave Ness, PCA
	André Roy, PCA	

1 Introduction and Purpose

The purpose of this memo is to discuss the methodology used to develop an ice load for the Port Severn Dam Safety Review. The ice load on the dam is assessed using the CEATI[1] method, which consists of an Ice Loads Calculation Program and outputs an ice load for various Canadian locations.

2 Ice Load Criteria

The CEATI Ice Loads Calculation Program is an Excel based program which outputs ice loads for various locations in Manitoba, Ontario and Quebec. The software gives the line load for the ice at both the dam and stoplogs for a user specified return period. The inputs to the program are the change of water level parameters. These parameters are made up of the water level cycle amplitude, the time to go from maximum to minimum water level and the change in average forebay water level during the (ice loading) event.

The user must specify the geographic location of the site. In the case of the Port Severn Dam Safety Review, the location is:

- Latitude 44.804°N
- Longitude 79.720°W

The model requires meteorological data to simulate the ice cover and determine the loading pattern on a hypothetical dam. The software will search for the nearest meteorological stations and provide an output for the modelled ice load using the specified water level parameters at that location. The user must interpolate from the nearest results to get a site specific value.

The nearest sites to Port Severn are shown in Table 2.1.

[1] CEATI Report T002700-0206. Static Loads on Hydro Electric Structures: Part 2: Design Guide

Table 2.1 - Nearest Meteorological Stations to Port Severn

Station Name	Province	Latitude	Longitude	Distance from Site (km)	Years of Record
Toronto Met Res Stn	Ontario	43.800	79.550	112.38	8
Mount Forest	Ontario	43.983	80.750	122.49	21
Peterborough A	Ontario	44.233	78.367	124.57	30
North Bay A	Ontario	46.367	79.417	175.24	45

The assumed Port Severn water level parameters are shown in Table 2.2.

Table 2.2 – Water Level Parameters

Water Level Parameters	
Water level cycle amplitude:	10 cm
Time to go from max. to min. water level or vice-versa:	5 days
Change in average forebay water level during the event.	0 cm

Table 2.3 shows the results of the ice load modelling for the closest 4 meteorological sites. Note that the ice loading at these locations is due to thermal expansion of the ice cover only (Ice Temp Change Only). The ice loads range from 72.1 to 83.5 kN/m for the normal case and 201.3 to 280.4 kN/m for the worst case. The worst case will be used in the stability analysis as the unusual load case.

Table 2.3 – Ice Loading Results

Station Name	Loads Generated by	1:100 Year Load (kN/m)	
		Purely Thermal	Worst Case Condition
Toronto Met Res Stn	Ice Temp. Change Only	72.1	201.3
Mount Forest	Ice Temp. Change Only	74.0	204.2
Peterborough A	Ice Temp. Change Only	83.5	224.9
North Bay A	Ice Temp. Change Only	82.2	280.4

The Peterborough and North Bay sites are believed to be most representative of the Port Severn Site, despite not being the geographically closest. This is based on comparing the Port Severn climate to the sites shown in Table 2.3.The climate station chosen to represent the Port Severn Location was the HONEY HBR BEAUSOLEIL station, located 12.7 km from the dam site. The climate normal data for the Mount Forest site was unavailable.

The winter temperature and snowfall are most similar to the Peterborough site and the winter rain is most similar to the North Bay site. In January and December, the variation of the values is too large to be deemed similar, so these are labeled N/A (not applicable). The results of this analysis are shown in Table 2.4.

Table 2.4 – Winter Month Climate Normals[2]

Month	Jan	Feb	Mar	Apr	May	Nov	Dec
Daily Average Temperature (C)							
Port Severn	-8.5	-7.2	-2.1	5.4	12.4	2.8	-5.1
North Bay	-13	-10.9	-4.8	3.3	11.2	-1.4	-9.1
Peterborough	-8.9	-7.7	-2	5.7	12.4	1.7	-5.3
Toronto	-7.1	-6	-0.7	6.2	13	3.1	-3.1
Rainfall (mm)							
Port Severn	9	9.4	32.9	58.3	70.6	74.6	16.2
North Bay	16.9	9.6	31.9	51.4	85.5	58.6	19.9
Peterborough	24.2	21.9	37.1	59.3	72.8	62.7	31.9
Toronto	18.3	24.6	45.3	57.8	69.7	62.8	40.8
Snowfall (cm)							
Port Severn	100	57.4	26.4	9.4	0.4	29.3	92
North Bay	63	52.2	38	16.2	2.1	35	61.3
Peterborough	40.7	30.2	25.4	7.5	0.1	16.3	40.3
Toronto	30.8	25.4	20.7	6.9	0	9.6	38.9
Representative Climate Station (PB - Peterborough, NB - North Bay, T – Toronto, N/A – Not Applicable)							
Daily Avg Temp (C)	PB	PB	PB	PB	PB	T	PB
Rainfall (mm)	NA	NB	NB	T	NB	T	NB
SnowFall (cm)	NA	NB	PB	PB	PB	NB	NA

3 Return Period

Neither the CEATI nor the CDA[3] provide a return period for Ice Load calculations. In Quebec, the normal loading case is defined as the 1:100 ice event[4].

According to the CEATI Ice Loads report (2003), the thermal load, when combined with certain water level fluctuations, will produce a much higher total ice load. This value is output by the model and described as the Worst Case scenario. This is the value which will be used for the extreme loading case.

4 Ice Thickness

The CEATI software does not output the modelled ice thickness. The ice thickness calculation frequently used, especially in Quebec, is related to the freezing index, a cumulative index based upon the freezing degree days metric. The ice load is assumed to act at half of this depth

For many sites in Quebec, the ice thickness is given as equal to 600 mm[5]. In this case, the load acts on the dam approximately 300 mm below the water surface.

[2] http://climate.weatheroffice.gc.ca/climate_normals/index_e.html
[3] Canadian Dam Association, Dam Safety Guidelines, 2007
[4] Société d'énergie de la baie james. *Ice Impact Forces and Thermal Expansion Loads on the Structures of the La Grande Complex*
[5] Hydro-Quebec, *Design Guide for Hydraulic Structures Guidelines and Criteria for Calculating Ice Loads* - APPENDIX A

In the CDA, Structural Considerations for Dam Safety (2007), Section 4.6, it discusses the ice thickness for use in Dam Safety Reviews. According to this text, the ice load is normally considered to act at 300 mm (1 foot) below the water level. This is consistent with the Hydro-Quebec approach.

In addition, according to Thermal Ice Forces on Concrete Dams (1994)[6], the majority of ice pressure occurs within the upper 300 mm (12 in) of the ice sheet. It appears that ice thickness over 300 mm has little impact on the overall ice load magnitude.

Therefore, the ice thickness for the Port Severn stability analysis is assumed to be 600 mm and acting on the dam at 300 mm below the water level.

5 Consideration

The normal 100 year CEATI ice loads for Peterborough and North Bay are 83.5 kN/m and 82.2 kN/m respectively. For the extreme load case, the loads are 224.9 kN/m to 280.4 kN/m for Peterborough and North Bay, respectively.

The Peterborough site is deemed to be the most similar, climatically, to the Port Severn site, as discussed in Section 2. Therefore, the values of the Peterborough ice load estimates will be used. For the normal loading case, an ice load value of 83.5 kN/m will be used in the Port Severn Dam Safety Review.

An additional unusual load case will be considered, usually 50% higher than the normal ice load. For the unusual ice load, a value of 125.3 kN/m will be used in the Port Severn Dam Safety Review.

For the extreme load case, a load of 224.9 kN/m will be used.

The results are summarized below in Table 5.1.

Table 5.1 – Summary of Ice Loads

Load Case	Line Load Value
Normal Ice Load	83.5 kN/m
Unusual Ice Load	125.3 kN/m
Extreme Ice Load	224.9 kN/m

All loads will be acting 300 mm below the water level. All loads considered have a return period of 100 years.

[6] P.K. Ko, M.S. Ho, and G.F. Smith - *THERMAL ICE FORCES ON CONCRETE DAMS; RECENT DEVELOPMENTS* Ontario Hydro, 1994.

Appendix H
Questionnaire to Operators

Questionnaire for operators

The following is a list of questions for the operators asked during the start-up meeting and during the site inspection:

1. Please describe operating staff capability (sector crew size, number of dams under its responsibility, distance to travel, experience, which shop/headquarter the operators come from, etc.).

   Answer:

   The crew is based at the Washago shop and has 6 dam sites, totalling 13 dams, under its responsibility, from Lake Simcoe to Georgian Bay. The shop is a 1-hour drive away from the Main Dam.
   During navigation season: Two lock operators who are trained to operate the dam are on site throughout the day at each of the 6 dams, from May to October. In addition, 5-6 maintenance employees are on duty and based at the Washago shop.
   During freshet: The lock is shut down and the 2 operators are assigned to the dam.
   During winter (October to May): The 5-6 maintenance employees based in Washago can operate the dam.
   Experience: among the maintenance crew, 2 have more than 25 years experience, 1 has between 10 and 15 years, and 1 has 5 years. Among the lock operators, 3 have more than 25 years experience and 9 have between 5 and 10 years.

   Revision:

   Personnel changes have occurred between the interview date and the delivery of the DSR. By July 2013, the crew based at the Washago shop was reduced to four full-time experience operators and one seasonal hire, available four months a year during summer time.

2. When is fall drawdown and when is the dam operated to reach normal operating level in the spring? Please provide dates.

   Answer:

   Fall drawdown to start after Thanksgiving with a target for total drawdown on December 1st.
   Normal operating level to be reached 1 week before May 21.

3. Please describe operating procedures:
   - in the summer
   - in wintertime
   - during floods.

   Answer:

   The operating procedures are described in the Standing Orders, a series of documents that were submitted to AECOM by S. Filion (see Appendix 12).

4. Can all stop logs be lifted from each bay under normal conditions?

   Answer:

   Sluices 7-8-9 have not been opened in 27 years. The first 7 or 8 stop logs are easy to remove, last 4 or 5 are more difficult (logs are bouncing or "chattering").

   - Under high water levels?

   Answer:

   Water levels are almost the same (fluctuation within 30 cm throughout the year)

   - With ice, frazil?

Answer:

Ice or frazil ice needs to be chipped away from the stop logs before they can be removed. Once this is done, the same number of stop logs as in normal conditions can be removed.

5. How long does it take per stop log and total time per bay for the 10 and 12 log bays using the mechanical log lifter? Using the backup winches?

   Answer:

   Log lifter: The first 7 or 8 logs within 30 minutes. More difficult to remove the last 4 or 5 (logs are bouncing or "chattering"). When a stop log breaks, it takes an extra 30-45 minutes to remove it.
   Back-up winches (manual): 5 hours.

6. Please describe the procedure followed when stop log gains are frozen in winter.

   Answer:

   Breaking the ice with a pike pole.

7. Have ice or debris ever jammed or blocked the spillway?

   Answer:

   Yes, definitely. Ice jams have happened, especially in springtime. They occur when a quick melt generates ice blocks, which hit the pier noses.
   Trees sometimes get jammed in the bays.
   Frazil ice is a problem. When frazil ice is present it is usually necessary to use the manual winches instead of the log lifter, since it is not possible to feel the recessed steel handling rings *at both ends of the logs.*

8. Please provide a description of the procedures and equipment used to jack the stop logs into position.

   Answer:

   Log jacking is performed by placing a hydraulic jack on the top stop log and a steel cross beam underneath the deck at each end of the sluiceway. Operators on the deck manually pump the jacks.

9. When stop logs get jammed in the gains, what is the procedure followed to dislodge them? Are logging tongs used?

   Answer:

   Yes, tongs are used.

10. Please provide a brief inventory of the stop logs; i.e., categorize by age and condition; provide the approximate number of each category.

    Answer:

    40 new stop logs were received last year for the whole sector, which is exceptional: normally 24 stop logs per year are received. This is not enough considering that the life expectancy is shorter for new stop logs than it was 40 years ago. It is believed that there are still original logs in bays 7, 8, and 9.
    It is estimated that approximate percentages for each age and condition category are as follows:
    - Very old (in service prior to 1950-60) and in poor condition: 30%
    - Old (between 1950 and 1980) and in fair condition: 25%
    - Relatively recent (in service since 1980) and in good condition: 45%

11. How many stop logs break during a typical operating season?

    Answer:

    Typically about 6 stop logs per year break for the entire sector. The breaks result from wear induced by the occurrence of "chattering" (vibration of the stop logs), as well as damage caused by the use of logging tongs. When a log splits the removal operation causes a delay of about 30 to 45 minutes.
    It is often necessary to shut the upstream set of gains to remove damaged stop logs.

12. Where are the backup winches stored in the case of the log lifter being out of service?

    Answer:

    Main dam: on rails; Dams E and G, back-up brought by truck from another dam. No established procedure.

    - How long does it take to mobilize the backup winches?

      Answer:

      Main dam: not long; Dams E and G, can take more than 1 hour.

    - How many workers are required to pull logs using the backup winches?

      Answer:

      2 workers.

13. Please describe maintenance activities done during the year (including: Do the operators have to climb over the log lifter or work on top of it? What specific maintenance is done to ensure that the log lifter is operational at all times? What are the refueling procedures? How do they ensure that the fuel level is adequate?).

    Answer:

    On a daily basis: checking the oil and other fluids, start-up. Monthly: visual inspection of hydraulic lines, telescopic rams, and hydraulic controls, oil changes (not necessarily every month), wear and damage inspection.
    It is sometimes required to climb on the log lifter for maintenance activities, such as accessing the hydraulic lines running to the top of the telescopic rams. A ladder is used to do so.
    The fuel level is checked every time the log lifter is used. A dip stick is used as there is no fuel gauge. It is important that the unit never runs out of fuel because it may induce damage to the carburetor. The fuel reservoir contains about 3 to 3.5 gallons. A 3-gallon fuel tank is stored at the opposite end of the unit and a 5-gallon tank is stored in the building adjacent to the dam. The unit is refuelled from its current location on the deck simply by pouring fuel from the tank into the reservoir. Accidental spills are very rare and limited in volume to a few drops that are wiped off with a cloth.

14. Are there any spare parts and backup equipment available?

    Answer:

    No.

15. Please describe any unusual operating conditions that may have been observed?

    Answer:

    The use of logging tongs.
    The removal of ice.
    Logging operations occurring at night. Lighting is inadequate.

16. Has water seepage or standing water been observed on the downstream side of the dams?

    Answer:

    Yes, water seepage is observed in general at most dams, and in particular at Dam G.

    Revision:

    What was thought to be water seepage at Dam G is actually water draining in the catch basins on the road and out through outlet pipes. Sinkholes at Dam C, Dam D and the Upstream Shoreline Wall suggest erosion, possibly from seepage through the damaged concrete. The origin of the water ponding behind Dam C could also be poor drainage.

17. Have traces of erosion, cracks, settlement, sinkholes been observed on the dams or in the dams surroundings?

Answer:

Yes, PWGSC and PCA accompanied AECOM after the start-up meeting to point out specific issues at different locations.

18. What are the minimum and maximum water levels during a normal year?
    - Minimum and maximum water levels in summer?

      Answer:

      Navigation season: Min 180.42 m, max 180.50 m,
    - Minimum and maximum water levels in winter?

      Answer:

      Non Navigation season: Min 180.20 m, max 180.30 m
      (during site inspection on 2011-12-06: 180.25 m)

19. What are the safety protocols/procedures in place for the operation of the dam?

    Answer:

    Safety protocols are described in the Standing Orders (see Appendix 12, section 7.1).

20. What is the average and maximum thickness of ice measured during past years?

    Answer:

    24 to 30 inches.

21. What are the approximate dates of ice cover formation, on an average year?

    Answer:

    Normally the ice cover forms at the beginning of December, and stays in place up to late March, sometimes to mid-April.
    - Early dates? Ice cover may form as early as November.
    - Late dates? Ice cover may form as late as January.

22. Are there known hazardous flow conditions for boats using the lock, such as return currents?

    Answer:

    With flows higher than 90 cms, the waterway is closed. To some degree, currents occur as soon as dam is open.

23. What protocol is used to verify that no one is upstream or downstream of the dam before operations are performed?

    Answer:

    Visual only. But increase in flow is never rapid since stop logs may only be removed one at a time, at intervals of about 3 minutes.

24. Can the log lifter reach and be used on all sluiceways?

    Answer:

    No. Sluices 1 and 9 require the manual winches. But sluices 7-8-9 have not been opened in 27 years.

25. How is the manual winch moved? How any workers are required to move the winches?

    Answer:

    Main Dam: manually, on rails; Dams E and G: by truck and manually by 2 workers.

26. Have the winches been used within the last year?

    Answer:

    Yes, during the summer season when maintenance is carried out on the log lifter.

Appendix I

Inundation Mapping – All Dam Break Scenarios

Figure I.1 Main Dam – Inundation mapping – Sunny Day Scenario (With and Without Dam Failure)

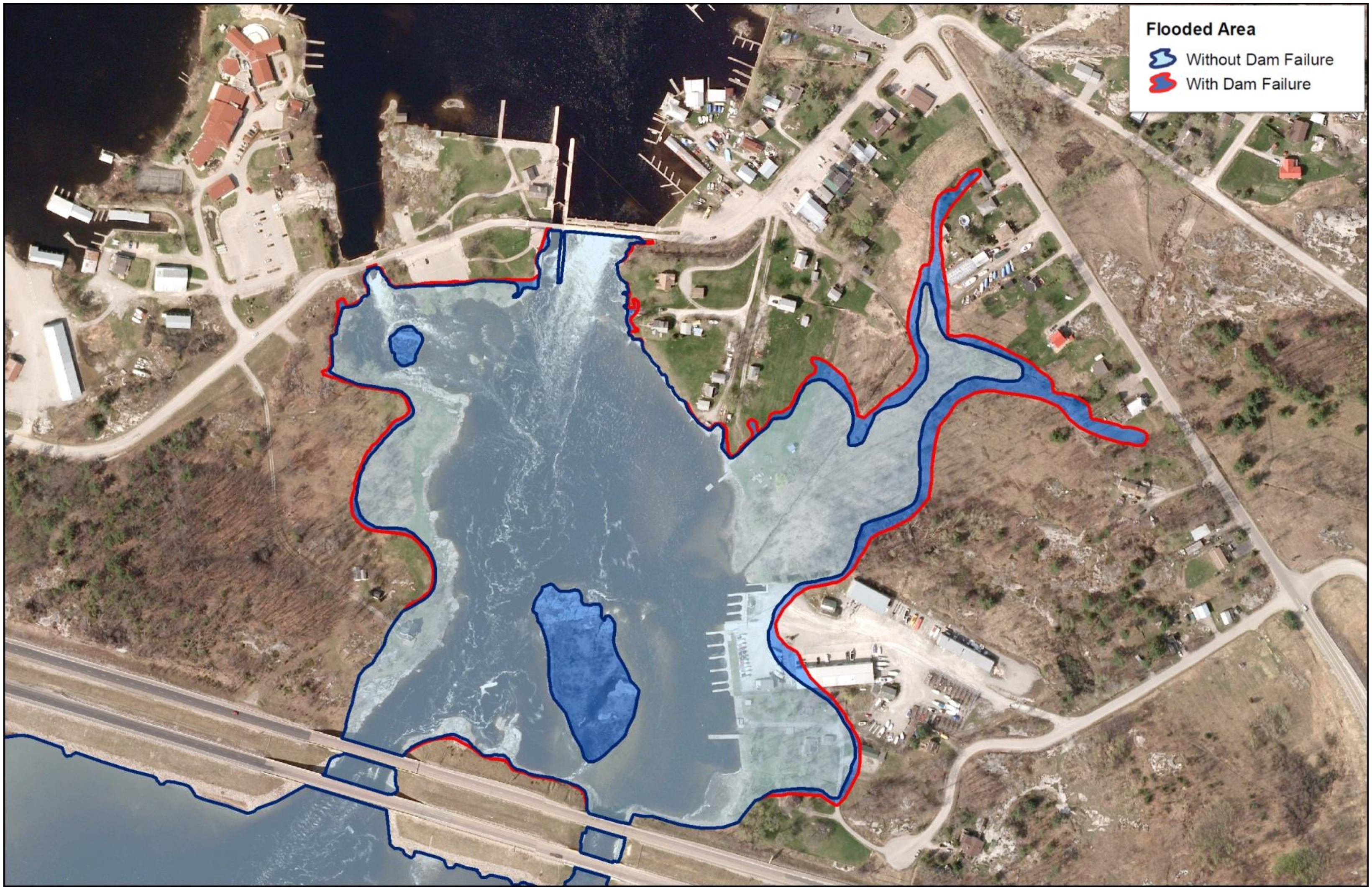


Figure I.2 Main Dam – Inundation mapping – 100-year Flood Scenario (With and Without Dam Failure)

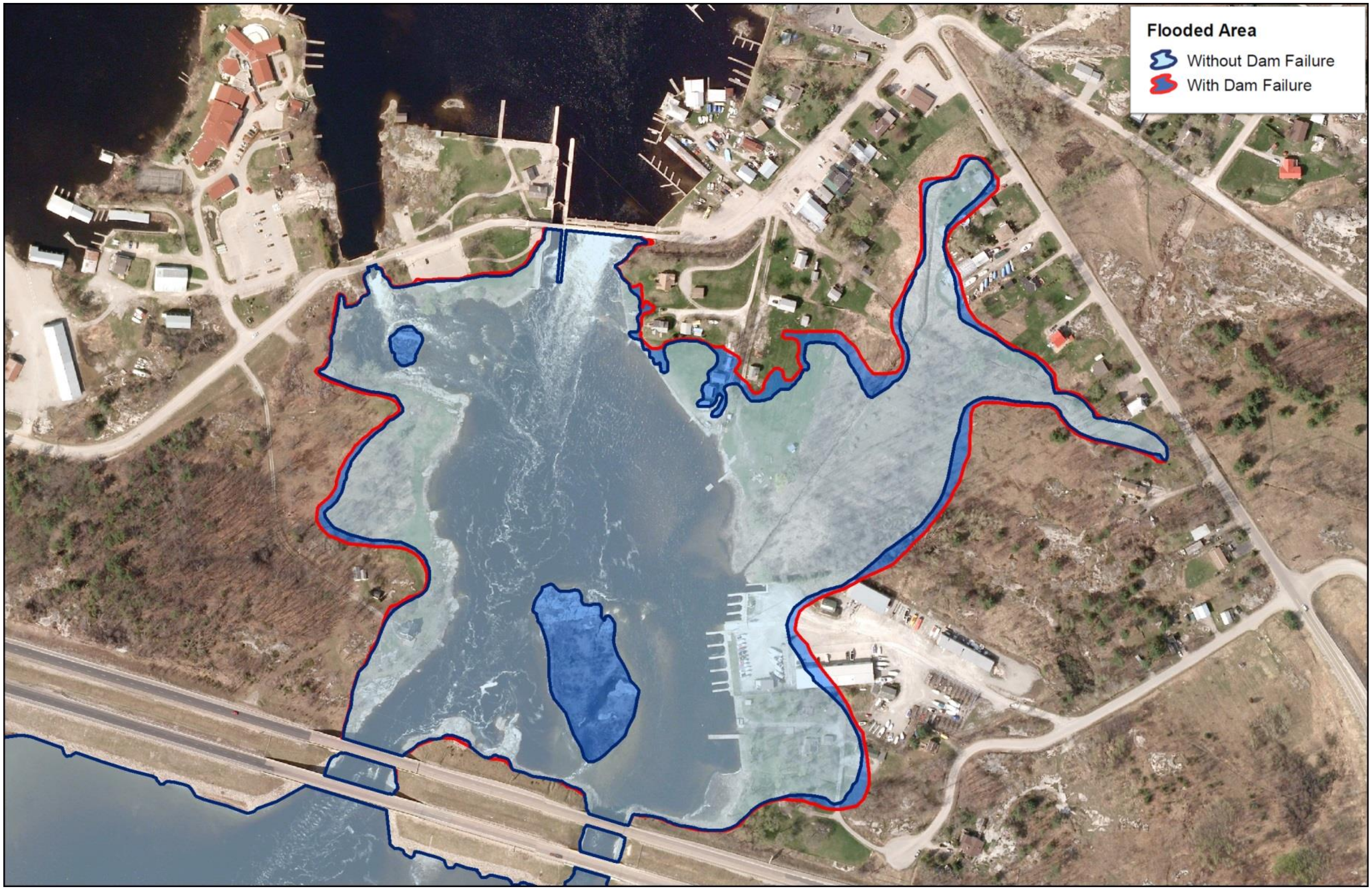


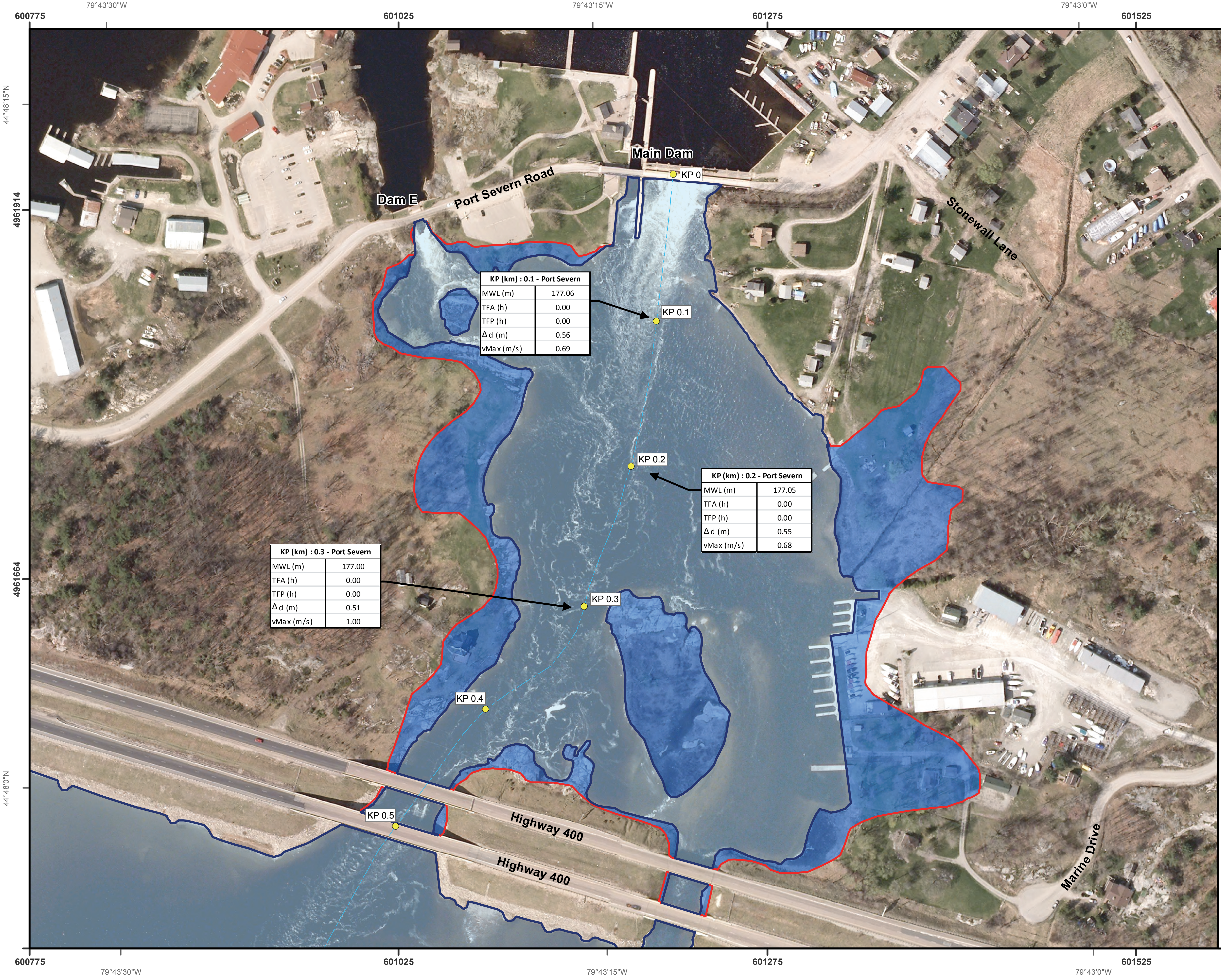
Figure I.3 Main Dam – Inundation mapping – 1,000-year Flood Scenario (With and Without Dam Failure)

Figure I.4 Main Dam – Inundation mapping – 10,000-year Flood Scenario (With and Without Dam Failure)

Figure I.5 Main Dam – Inundation mapping – PMF Scenario (With and Without Dam Failure)

Appendix J
Detailed Inundation Mapping

- **Failure under Sunny Day Condition**
- **Failure under IDF Condition**



KP (km) : 0.1 - Port Severn	
MWL (m)	177.06
TFA (h)	0.00
TFP (h)	0.00
Δ d (m)	0.56
vMax (m/s)	0.69

KP (km) : 0.2 - Port Severn	
MWL (m)	177.05
TFA (h)	0.00
TFP (h)	0.00
Δ d (m)	0.55
vMax (m/s)	0.68

KP (km) : 0.3 - Port Severn	
MWL (m)	177.00
TFA (h)	0.00
TFP (h)	0.00
Δ d (m)	0.51
vMax (m/s)	1.00

Travaux publics et Services gouvernementaux Canada — Public Works and Government Services Canada

Parcs Canada — Parks Canada

Dam Safety Review
Port Severn, Trent-Severn Waterway

INUNDATION MAP
DAM BREAK - SUNNY DAY SCENARIO
MAIN DAM

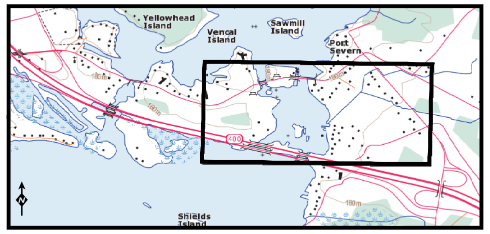
Flooded Area

- Without Dam Failure
- With Dam Failure

Symbols

KP : Kilometric Point (km)
MWL : Maximum Water Level (m)
TFA : Time of Flood Arrival (h)
TFP : Time of Flood Peak (h)
Δ d : Water Depth (m)
vMax : Maximum Velocity (m/s)

Site location



Sources :
Orthophoto: PWGSC, Parcs Canada (2012)
National Road Network: Edition 2.0.1, 2007, Geobase (2013)

GIS analysis : AECOM
Project : 60273660

0 25 50 Meters
1 : 2,500

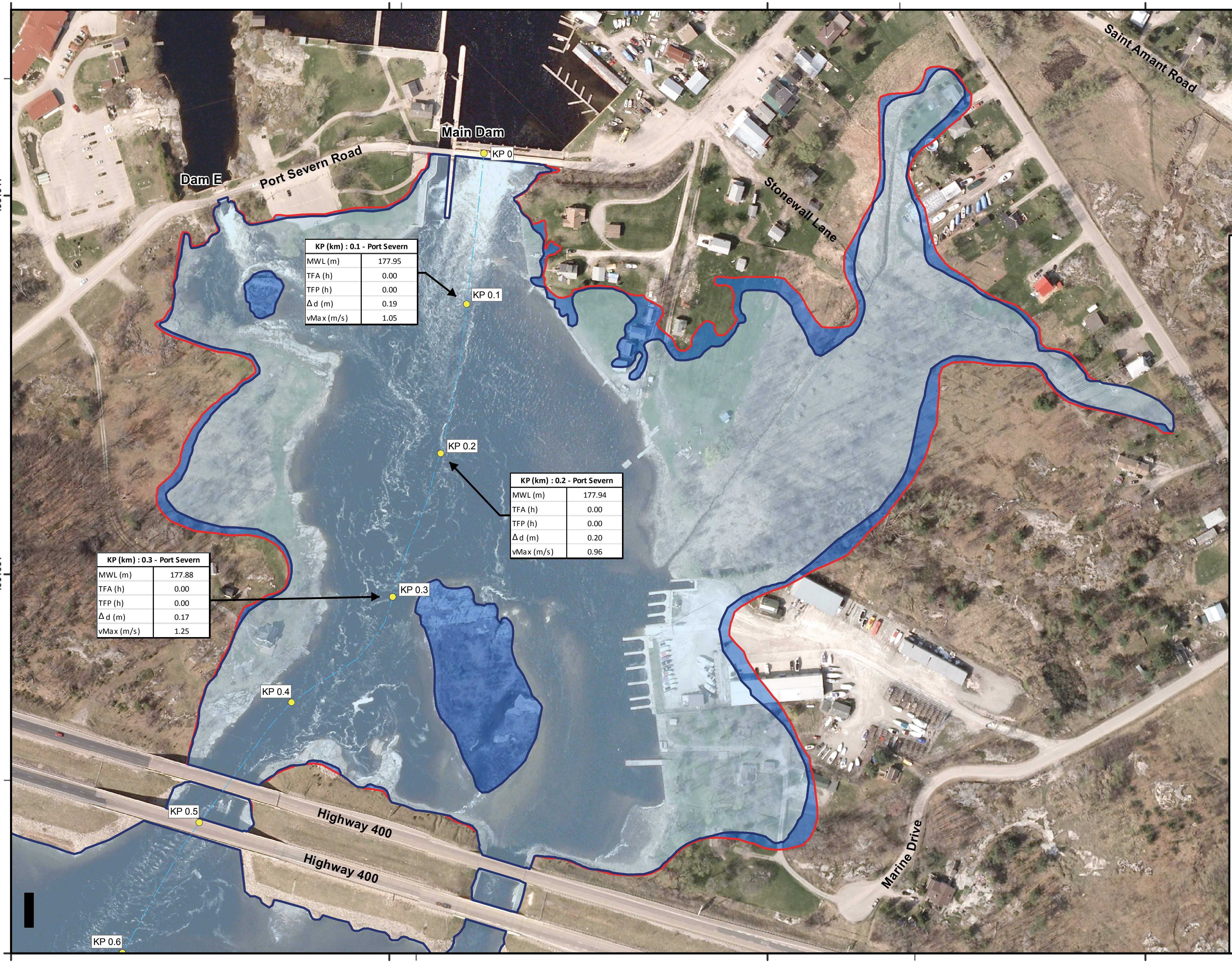
Projection : UTM Zone 17N
Datum : NAD 83

Map 1

March 2013

AECOM

600898 | 79°43'15"W 601148 | 601398 | 79°43'0"W | 601648 | 79°42'45"W 601898



KP (km) : 0.1 - Port Severn	
MWL (m)	177.95
TFA (h)	0.00
TFP (h)	0.00
Δd (m)	0.19
vMax (m/s)	1.05

KP (km) : 0.2 - Port Severn	
MWL (m)	177.94
TFA (h)	0.00
TFP (h)	0.00
Δd (m)	0.20
vMax (m/s)	0.96

KP (km) : 0.3 - Port Severn	
MWL (m)	177.88
TFA (h)	0.00
TFP (h)	0.00
Δd (m)	0.17
vMax (m/s)	1.25

600898 | 601148 79°43'15"W | 601398 | 79°43'0"W | 601648 | 79°42'45"W 601898

Travaux publics et Services gouvernementaux Canada — Public Works and Government Services Canada

Parcs Canada — Parks Canada

Dam Safety Review
Port Severn, Trent-Severn Waterway

INUNDATION MAP
DAM BREAK - IDF SCENARIO
MAIN DAM

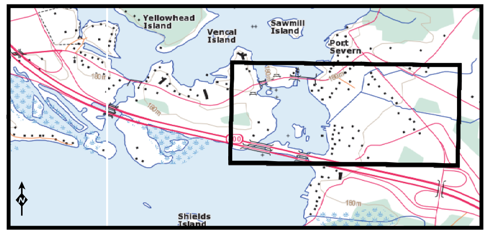
Flooded Area

- Without Dam Failure
- With Dam Failure

Symbols

KP : Kilometric Point (km)
MWL : Maximum Water Level (m)
TFA : Time of Flood Arrival (h)
TFP : Time of Flood Peak (h)
Δd : Water Depth (m)
vMax : Maximum Velocity (m/s)

Site location



Sources :
Orthophoto: PWGSC, Parcs Canada (2012)
National Road Network: Edition 2.0.1, 2007, Geobase (2013)

GIS analysis : AECOM
Project : 60273660

0 25 50 Meters
1 : 2,500

Projection : UTM Zone 17N
Datum : NAD 83

Map 2

March 2013

AECOM

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AECOM is a global provider of professional technical and management support services to a broad range of markets, including transportation, facilities, environmental, energy, water and government. With approximately 45,000 employees around the world, AECOM is a leader in all of the key markets that it serves. AECOM provides a blend of global reach, local knowledge, innovation and technical excellence in delivering solutions that create, enhance and sustain the world's built, natural and social environments. A Fortune 500 company, AECOM serves clients in more than 140 countries and had revenue of $8.2 billion during the 12 months ended March 31, 2013.

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