QUINTE CONSERVATION

Final Report

Bellrock Dam

Napanee Watershed

April 2004

prepared by

ACRES International

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Preface

This report presents the results of a civil, geotechnical, hydrologic and hydraulic Dam Safety Review for Bellrock Dam based on the draft Ontario Dam Safety Guidelines (ODSG), (Ministry of Natural Resources, 1999).

This report is divided into the following main parts:

- Executive Summary
- Part I – Data Collection and Site Inspection
- Part II – Dam Classification
- Part III – Hydrotechnical Assessment
- Part IV – Dam Assessment
- Part V – Recommendations and Costs.

Executive Summary

Executive Summary

The Quinte Conservation (QC) is the custodian of more than 30 dams in the Moira River, Salmon River and Napanee River watersheds and, as such, is responsible for their operation and maintenance. Many of the dams were built in the 1950s and 1960s, and the aging infrastructure requires that a program of inspection and dam safety technical analysis be completed to form the basis for an assessment program to decide if site facilities should be decommissioned or rehabilitated and upgraded. Four dams located within the Napanee River watershed were identified by QC for dam safety reviews. These are the Colebrook, Bellrock, Third Depot Lake and Second Depot Lake dams.

Acres International Limited (Acres) has been retained by QC on October 31, 2003 to perform dam safety review for the selected dams based on guidelines and procedures given in the draft Ontario Dam Safety Guidelines (ODSG) prepared by the Ministry of Natural Resources (MNR, 1999).

The study for Bellrock Dam was divided into tasks that could be summarized as follows:

Task 1 – Data Collection and Site Inspection
An initial review of the available information on the Bellrock Dam was performed during the kickoff meeting (November 5, 2003) and site inspection (November 5, 2003), by Acres inspection team. Inspections were undertaken from crest level and from upstream and downstream sides of the structures (where applicable). Digital photographs were taken to document structural, hydrological and geotechnical aspects of the dam and appurtenant structures. The Pre-Inspection Background Information (Form B1) and the Dam Inspection Report (Form B2) were completed. Assessment drawings (two) were prepared to fully define the features, survey and deficiencies of the dam.

The dam is a concrete gravity structure with integral concrete spillway and overflow sections. The dam is in fair to good condition with some map cracking and erosion at the waterlines. A major concrete deterioration was observed at the downstream face of the wingwall with its junction with the right pier. The overflow weir was not inspected due to spillage at the time of inspection.

Task 2 – Dam Classification

The consequences of a dam failure are assessed in terms of the incremental hazard posed by the structure, based on guidelines and procedures given in the ODSG (MNR, 1999). The incremental hazard potential (IHP) assigned to a dam is defined as the potential for increase in loss of life, property damage, disruption in social and economic activities and/or environmental impacts caused by failure of the dam structure incrementally above that which would have occurred without failure of the dam.

There are permanent residences downstream from the dam. Land use surrounding the impoundment is largely forest with some agriculture and residential usage near the town of Bellrock. There is no potential for loss of life. The socioeconomic, financial and environmental losses are very limited if the dam fails given its small size. The flood flow due to the failure would be quickly reduced in the downstream channel and the wetland area.

Based on the above conditions, the Bellrock Dam is classified as a VERY LOW IHP dam for the sunny day and flood failures.

Task 3 – Hydrotechnical Assessment

The purpose of the flood analysis was to estimate peak flood flows and hydrographs for the 2-yr, 5-yr, 10-yr, 20-yr, 50-yr and 100-yr return period floods and the probable maximum flood (PMF) for the Bellrock Dam study area. The deterministic peak flow estimates for the damsite were derived using the Guelph All-Weather Storm Event Runoff Model (GAWSER) (Schroeter and Whiteley, 1987) provided by QC.

The study results indicate that the reservoir storage of Bellrock Dam is small, so there is little peak flow reduction for a flood event higher than 1:2-yr event.

Task 4 – Dam Assessment

Three sections were analyzed which included the sluiceway, overflow and gravity structures. Due to the VERY LOW IHP, the sections were not analyzed for earthquake loading. The results indicate that the gravity section satisfied the summer normal loading conditions. The sluiceway and overflow sections do not meet the ODSG requirements under summer normal conditions. All sections developed unstable cracks under winter ice load conditions and failed to meet the ODSG requirements.

The gravity section satisfied criteria under the 1:50-yr flood condition. The overflow and sluiceway both developed unstable cracks along the base and, therefore, did not satisfy the ODSG requirements.

Task 5 – Recommendations and Costs

As a result of the dam safety review for the Bellrock Dam, a number of recommended actions and maintenance activities were identified that are intended to address concerns related to satisfying current dam safety criteria. These ranged from maintenance activities to more significant issues that may require rehabilitation work. The following action plan is recommended:

- Evaluate/monitor/inspect the right and left abutments of the dam for seepage discharge points.
- Remove deteriorated concrete, apply repair material and undertake stabilizing measures.

An overall cost summary of the remedial repairs, including allowances for contractor mobilization/demobilization, control of water, contingency, engineering and supervision, could be summarized as follows:

Description	Estimated Cost
Construction costs (required immediately)	$8,250
Construction costs (required within 2 years)	$57,750
Total	**$66,000**

Figure ES-1

Bellrock Dam

Description

Original construction:	1958
Type:	Gravity
Height:	2.5 m
Length:	21 m
Reservoir area:	0.25 km^2

Classification

Sunny day:	VERY LOW
Flood:	VERY LOW
IDF:	50-yr flood
Spillway capacity (PMF):	N/A

Condition

General condition:	Generally in fair to good condition
Previous investigations:	Crysler & Lathem Ltd. (CCL) study in 1977 to 1978

Stability

- The gravity section satisfied the summer normal loading conditions.
- The sluiceway and overflow sections do not meet the ODSG requirements under summer normal conditions.
- All sections developed unstable cracks under winter ice load conditions and failed to meet the ODSG requirements.
- The gravity section satisfied criteria under the 1:50-yr flood condition.
- The overflow and sluiceway both developed unstable cracks along the base and, therefore, did not satisfy the ODSG requirements.

Recommendations and Costs

- Evaluate/monitor/inspect the right and left abutments of the dam for seepage.
- Remove deteriorated concrete, apply repair material and undertake stabilizing measures.
- Estimated costs associated with the above: $66,000.

Part I
Data Collection and Site Inspection

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Part I – Data Collection and Site Inspection

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1 Introduction

1 Introduction

1.1 Background and Purpose

The province of Ontario has not yet implemented dam safety regulations, although Ontario Power Generation (OPG) has been actively engaged for some time in the safety evaluation of their own dams. However, as part of their mandate under the Lakes and Rivers Improvement Act, the Ontario Ministry of Natural Resources (MNR) has introduced dam safety and flood emergency contingency planning requirements that are based, in part, on the Canadian Dam Association Guidelines. These have been formalized in the form of a draft document entitled "Ontario Dam Safety Guidelines" (ODSG).

The Quinte Conservation (QC) is the custodian of more than 30 dams in the Moira River, Salmon River and Napanee River watersheds and, as such, is responsible for their operation and maintenance. Many of the dams were built in the 1950s and 1960s, and the aging infrastructure requires that a program of inspection and dam safety technical analysis be completed to form the basis for an assessment program to decide if site facilities should be decommissioned or rehabilitated and upgraded. Four dams located within the Napanee River watershed were identified for dam safety reviews. These dams are

- Third Depot Lake Dam
- Second Depot Lake Dam
- Colebrook Dam
- Bellrock Dam.

The scope of this dam safety review for the Bellrock dam facilities includes a technical analysis and review of issues with respect to structural integrity and the ability to safely manage flood inflows of a severity befitting the size and location of the structure.

The Bellrock Dam is located in the town of Bellrock at Depot Creek, approximately 61 m east of Bellrock Mill, which drains into the Camden Swamp from the north and then discharges into Napanee River.

1.2 Site Inspection

An inspection of the Bellrock Dam for the Quinte Conservation (QC) was performed on November 5, 2003, with the following objectives.

- Inspect and document the condition of log lifting equipment, stop logs, gains, gain covers, power hoists, gates, railings, catwalks, fencing, log booms, safety cables, signs, etc, and recommend appropriate measures to correct unsafe conditions.

- Review the operation and surveillance plans and determine their adequacy.

- Measure all major dam dimensions and create scaled drawings. Significant differences between 'original' drawings and current measurements to be noted.

- Undertake a detailed structural inspection to assess existing conditions, confirm concrete strength and integrity, and confirm concrete contact with other materials.

- Prepare detailed plans and drawings to document the location, type and extent of surface deterioration. Identify the probable cause or causes of such deterioration.

- Complete Pre-Inspection Background Information Form B1 and Dam Inspection Report Form B2.

- Document important features using a digital camera.

- Carry out geological mapping at the dam to identify geological features that could affect the stability of the structure and estimate the concrete/bedrock contact strength.

- Map seepage areas downstream from the dam and estimate flow rates.

- Assess and document any evidence of seepage or settlement.

- Undertake a hydrotechnical assessment of the damsite, noting downstream and upstream reservoir conditions, including surrounding land use, floodplain and bank vegetation type. Document discharge facilities settings and water releases, and measure reservoir and tailwater levels at the time of site inspection.

- Inspect downstream conditions and document any infrastructure at risk in the event of individual or consequential dam failure.

- Perform site survey in order to establish critical elevations (geodetic if already available at the dam, otherwise local datum).

A summary of the major characteristics of the dam inspected is presented in Table 1.1. The dam is a concrete gravity structure with integral spillway and overflow sections. The location of the dam is shown in Figure 1.1.

Inspections were undertaken from deck level and from upstream and downstream sides of the structures. Appendix A contains a selected set of photographs taken during the site visit. Direct reference to these photographs is indicated in the text. Appendix B includes the Pre-Inspection Background Information Form B1 and the Dam Inspection Report Form B2, while Appendix C contains the survey, structural and deficiency drawings.

This Part I of the report presents the results of the qualitative inspection of the above site, identifies problem areas, and indicates where future investigations and/or studies might be warranted.

Table 1.1

Description of Bellrock Dam

Name of Dam	Access	Description				Available Data			Inspections and Previous Repairs	
		Reservoir Area (km^2)	H (m)	L (m)	No. of Sluices	Hydrotechnical	Dwgs	Reports	Date	Description
Bellrock Dam	13 km upstream of Colebrook Dam	0.012	2.5 max.	21 from bank to bank	One stop log bay 3 m wide with 7 installed stop logs	- Head-discharge curve - Reservoir capacity curve - Monthly and annual mean discharges at Camden east and Napanee stations	1977 concept drawing	- 1970 Pre-engineering study by Totten Sims Hubicki Associates Limited - 1978 Napanee River Basin Study by CCL Consultants	mid-1970	Mid-1970s repairs - refurbishing of the concrete piers

Note: In this report, “left” and “right” reference dam structures or riverbanks when looking downstream.

Bellrock Dam

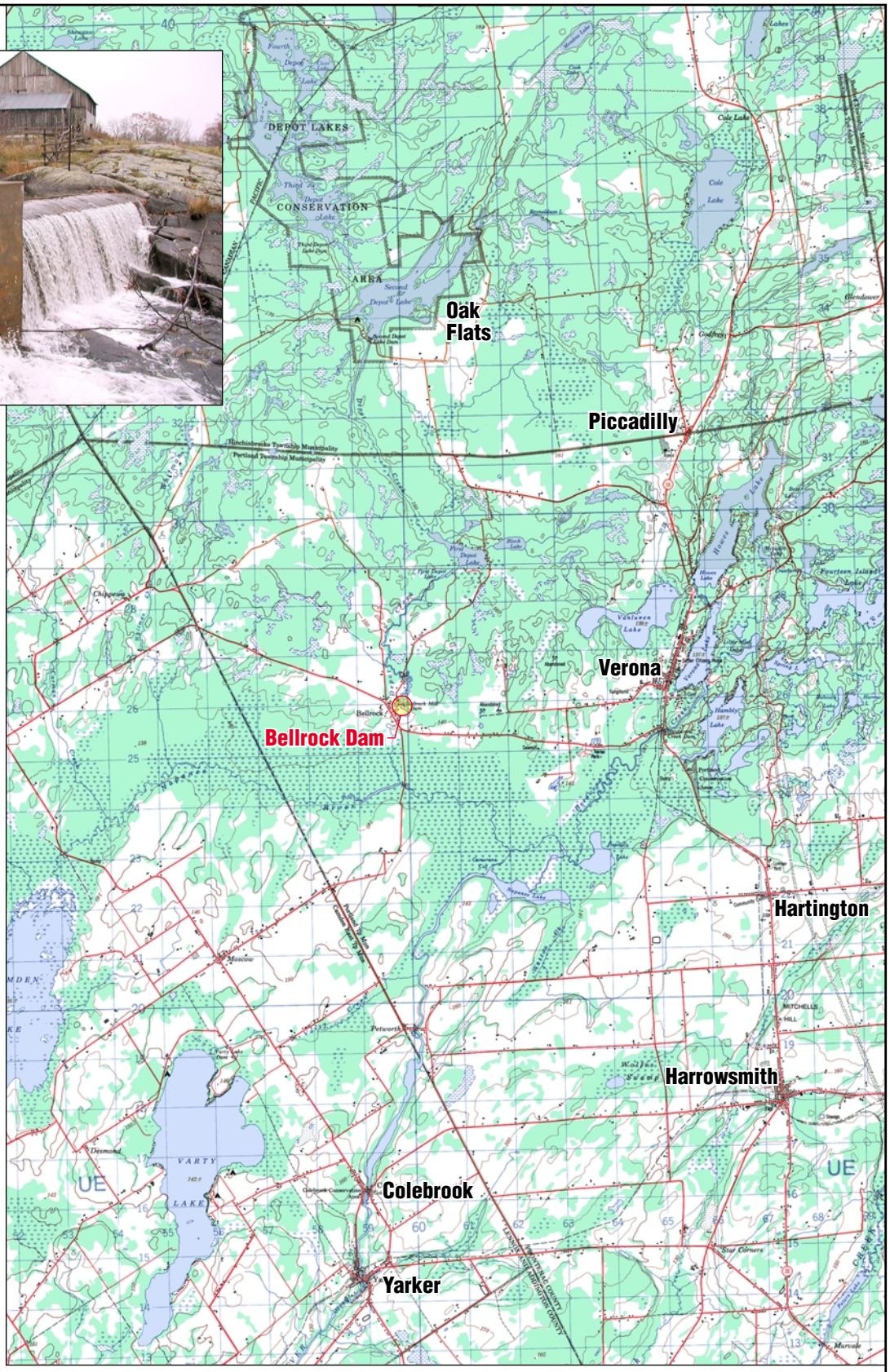

Figure 1.1
Quinte Conservation
Bellrock Dam Location

ACRES International

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2 Kickoff Meeting, Data Collection and Data Room

2 Kickoff Meeting, Data Collection and Data Room

2.1 Kickoff Meeting

A kickoff meeting took place at 9:00 a.m. on November 5, 2003, at the QC office, 2061 Old Highway 2, R.R. #2, Belleville, Ontario. Present at the meeting were:

- QC - Mr. Terry Murphy and Mr. Paul McCoy
- Acres - Mr. Richard Donnelly, Mr. Fadi Chidiac, Mr. Amin Touhidi and Mr. Ross Zhou.

The meeting followed the agenda below:

- introduction of attendees
- overview of Acres assignment objectives
- discussion of disposition of reference materials (data and information)
- site visit with Mr. Harry Stinson of QC to the Bellrock Dam was arranged
- a list of documents that were provided at the meeting was prepared.

2.2 Data Collection

The following project documents were provided to Acres during the kickoff meeting on November 5, 2003, for Acres initial/up-front review prior to the site visit to Bellrock Dam:

- Napanee River Basin Study by Crysler & Lathem Ltd. (CCL) Consultants, 1978
- Napanee Salmon Upstream Lakes Watershed Study by CCL Consultants, 1981
- GAWSER, Guelph All-Weather Storm Event Runoff computer hydrological model, 1999
- Moira River Integrated Flood Forecast System (MRIFFS), 2000.

2.3 Data Room

The following project documents were provided to Acres on November 7, 2003, during Acres data room at QC's office in Belleville:

- Napanee River, Design Flow with Dams Computer Printout;
- Napanee River, HEC-2 Data Computer printout.

3 Initial Data Review

3 Initial Data Review

As a first step in the assessment process, an initial review of the available information was performed during the kickoff meeting at QC's office in Belleville. As part of this process, a drawing showing plan view and cross sections of the Bellrock Dam was copied from CCL Consultants' 1978 report entitled "Napanee River Basin Study" to be used as a 'mark-up drawing' during Acres site inspection.

The results of this review provided a general understanding of the characteristics of the Bellrock site and the operational issues and the types of structural/ geotechnical problems that might be expected on the basis of the prevailing topographic, climatic and geological conditions. The following are some problems that may be expected to occur at a dam of this type:

- leakage at defects in the concrete, at the concrete/foundation contact or through open discontinuities
- typical concrete deterioration problems
- sliding stability problems associated with winter ice loadings
- inadequate spill capacity
- public and operational safety issues (signage, fall arrest systems, handrail condition, etc).

During the site inspection, the potential for these types of problems was specifically addressed, in addition to other issues that became apparent during the course of the site inspection.

4 Data Review and Site Inspection Observations

4 Data Review and Site Inspection Observations

4.1 Introduction

Acres civil, geotechnical and hydrotechnical engineers, accompanied by Mr. Harry Stinson of QC, made a site inspection and evaluation of the Bellrock Dam on November 5, 2003. The results of these inspections are presented in the following sections; digital photographs are in Appendix A; and comprehensive Forms B1 and B2 are in Appendix B—all in accordance with the requirements of the request for proposal (RFP).

4.2 Background

The Bellrock Dam is located in the town of Bellrock on Depot Creek, which empties into the Cameron Swamp from the north. This dam sits astride private land. The construction date of this dam is unknown. From a review of available documents, recent repairs of this dam seem to be done in the mid-1970s. This dam was built to maintain water supply for an adjacent mill, which has been restored with the intent of establishing a historic park.

The Bellrock Dam is an overflow concrete structure with a central stop log sluice. The maximum dam height is 2.5 m. A secondary structure, a retaining wall with a sluiceway, exists about 50 m to the west along the reservoir edge. The sluice formerly provided water to the adjacent mill that is no longer in operation.

4.3 Physiographical and Geological Setting

The Bellrock Dam is built on Depot Creek located in the town of Bellrock. The creek is among the headwaters of the Napanee River, which flows in a well-marked valley of preglacial origin.

The damsite is situated on the Precambrian Shield (Reference 1). Extending to the east, west and south is a large peat bog many kilometres long, and to the north are shallow glacial till and rock ridges.

The country rock is Precambrian granite. The bedrock is exposed or has less than 1 m of overburden cover. At the damsite, the overburden cover is practically nonexistent.

The damsite is located in a forested area of gentle to moderate relief. Rock outcroppings are large and prominent.

The downstream riverbanks show numerous rock outcroppings. The riverbed adjacent to the dam also shows rock exposures; however, at the time of inspection the majority of the riverbed area was not visible due to high water levels.

4.4 Record of Observations

4.4.1 Dam General Description

The Bellrock Dam, looking downstream from the right bank (west) to the left bank (east), consists of the following components (see drawings in Appendix C):

- right wingwall, consisting of a concrete retaining wall 7.3 m long, 2.5 m high and 2.1 m thick (see Drawing 15396-BD-C-001, Section A-A)

- stop log opening 3 m wide, 2.5 m high, equipped with seven logs (see Drawing 15396-BD-C-001, Section B-B)

- concrete overflow weir 8 m long, 1 m high (average), 1 m thick (see Drawing 15396-BD-C-001, Section C-C).

The Mill dam is a sluiceway that exists about 50 m to the west along the reservoir edge. The sluice formerly provided water to the adjacent mill that is no longer in operation.

4.4.2 Geotechnical Aspects

4.4.2.1 Previous Geotechnical investigations

Crysler & Lathem Ltd. (CCL) conducted a study in 1977-78 (Reference 2) that examined hydraulic aspects of this site and the stability of the dam. Trow Associates examined the site for CCL and concluded that

- flow exists along the concrete/rock contact on the west (left) abutment; the east was obscured by flow
- grouting of the contact was recommended.

Remedial works have not been done at the dam.

4.4.2.2 Bellrock Dam

Foundation Preparation and Characteristics
There are no construction records of the foundation preparation. The dam is clearly founded on bedrock, for there is very little overburden, and abundant smooth bedrock outcroppings exist on the shoulder of the dam.

The bedrock consists of a gray, crystalline, coarse-grained granite or granodiorite. The granite is faintly weathered (oxidization along joints) and very strong. Two dominant joint sets occur at the dam. They are described below. The orientations are related to true north.

Discontinuity	Dip/Dip Direction (deg)	Remarks
Joint Set 1	40/140	Continuous, parallel to flow, favorable, open at surface.
Joint Set 2	20/110	Discontinuous, favorable, dipping to left abutment, open at surface.

Rock Foundation Properties
The dam is founded on bedrock consisting of good quality, very strong, crystalline granite. The jointing orientation is favorable and ensures a

high sliding resistance. The critical sliding surface is, therefore, the rock/concrete interface.

Shear Strength Parameters
The frictional shear resistance parameter for the concrete-to-rock interface has been calculated assuming the dam is founded on a natural smooth surface, the Barton formula for a 2.5-m structure and a conservative joint roughness coefficient (JRC) of 5. The values are listed below.

- Basic shear resistance 30°
- JRC 5
- Equivalent roughness 13°
- Basic + roughness 43°

Seepage and Uplift
Seepage was not observed during the inspection because the dam was overflowing. Trow's observation in 1978 of seepage along the concrete/rock contact on the left side is still valid. It is recommended that 100% uplift be applied for stability assessment.

Instrumentation
No instrumentation exists.

Geotechnical Assessment
The concrete structure is founded on good quality, very strong, crystalline granite. The critical sliding surface is the rock/concrete interface, and in the absence of foundation preparation data, a smooth rock surface is assumed and a shear resistance of 43° is recommended. No further investigation and instrumentation is recommended.

4.4.2.3 Previous Geotechnical Investigations

The overflow weir is a low structure about 1 m high with a battered or inclined downstream face. An earth roadway exists to the immediate south of the wall that supports the road infill material.

The wall appears to be founded on bedrock given the paucity of overburden in the area. Granite bedrock is exposed in the water at the east end of the wall and immediately downstream on the west end.

References

1 Map 2556, Quaternary Geology of Ontario, Ministry of Northern Development and Mines, 1:1 000 000, 1991.

2 Crysler & Lathem Ltd., June 1978, "Napanee River Basin Study". Report submitted to Napanee Region Conservation Authority.

4.4.3 Civil/Structural Engineering Aspects

The rock at the left abutment of the Bellrock Dam was exposed (Photos 1 and 2), and water was spilling above the overflow weir portion of the dam during the site inspection (Photo 3).

Right Wingwall

Some map cracking was noticed on the crest of the wall, and light spalling was observed at the junction wall/pier with no sign of erosion at the waterline (Photo 4). No sign of wall movement was observed. A major concrete deterioration was observed at the downstream face of the wall with its junction with the right pier (Photo 5). Also, seepage through the concrete joints was noticed. Stability of this structure could be an area of concern. If problems are identified, repair works may include crack treatment.

Stop Log Opening

Rough concrete surfaces were noticed at the waterline inside the piers. Deck, allowing crossing from the right wingwall to the stop log guides, is of steel grating (Photo 6). Guards are fabricated of steel pipe. Slot (gain) steel was not severely rusted above the waterline. The steel deck was in good condition and well fixed to the concrete piers. The steel deck components need some painting. The topside of the concrete piers was showing some map cracking with spalling observed near the stop log gains (Photos 7 and 8). Hoist equipment was not available during the site inspection. There were no stop logs on the deck. The number of stop logs

in place could not be determined due to water spilling. As reported by the dam operator, seven stop logs remain in the stop log sluice year round with operation limited to occasional removal to facilitate clean-out of the headpond or other such infrequent requirements (low flow augmentation).

Concrete Overflow Weir
The weir was not inspected due to spillage at the time of inspection (Photo 9). Stability against overturning and sliding of this structure could be an area of concern.

Safety
A "Dam - Danger" sign was clearly visible on the upstream side of the sluice opening (looking downstream). The sign was generally in good condition. It was noted that "No Trespassing" signs were seen on the left and right access sides to the dam. There is a barrier at the right abutment to prevent access to the structure. The barrier was locked and unlocked by the dam operator at the time of the site inspection.

The Mill dam is a retaining wall 32 m long (Photo 10) with a sluiceway that formerly provided water to an adjacent mill that is no longer in operation (Photo 11). The concrete was in good condition. A "Dam - Danger" sign was clearly visible on the sluiceway guard. The sign was generally in good condition (Photo 12).

4.4.4 Hydrotechnical Aspects

The Bellrock Dam is located in the town of Bellrock at Depot Creek, approximately 61m east of the Bellrock Mill, which drains into the Cameron Swamp from the north and then discharges into Napanee River. The Napanee River empties into Lake Ontario via the Bay of Quinte. This damsite is approximately 13 km upstream of the Colebrook Dam. There is a sluiceway dam located near the Bellrock Dam upstream of the mill to provide water for the mill's operation.

The total drainage area controlled by the dam is 186.3 km^2. The height of the Bellrock Dam is approximately 2.4 m. The headpond at Bellrock Dam is small, encompassing an area of approximately 11 600 m^2. Wind wave is not a problem at this damsite. The estimated storage at the full supply level is

approximately 28 000 m^3. The dam has an overflow weir on the left side (looking downstream), which is approximately 8.1 m wide, and one stop log bay, which is 3 m wide and contains a maximum of seven stop logs.

There are no permanent residences downstream of the dam. Land use surrounding the impoundment is largely forest with some agriculture and residential usage near the town of Bellrock.

At the time of the visit, water was spilling over the top of the stop logs and the overflow weir. The depth of water over the crest of the overflow weir was estimated to be 0.05 m. approximately. The flow was estimated to be less than 5 m^3/s, based on the stage-discharge curve of the weir. The downstream tailrace of the dam is formed by exposed bedrocks with very steep slopes, and hence the flow regime downstream of the dam is supercritical. High tailrace water level during flooding seems not to be a problem.

A staff gauge is installed near the sluice gate on the right pier. The staff gauge reading appears to be based on local datum.

The dam can be accessed from both sides by local roads year round.

5 Phase 2 Site Investigations

5 Phase 2 Site Investigations

No field investigations were performed at the Bellrock Dam during the course of the current dam safety review. A number of recommended actions were identified that are intended to address concerns related to satisfying current dam safety criteria. These are detailed in Part V of this report.

Appendix A

Site Digital Photographs

Photo 1 - November 6, 2003
Bellrock Dam
Bellrock Dam - Upstream View

Photo 2 - November 6, 2003
Bellrock Dam
Bellrock Dam - Downstream View

Photo 3 - November 6, 2003
Bellrock Dam
Bellrock Dam - Left Abutment

Photo 4 - November 6, 2003
Bellrock Dam
Bellrock Dam - Upstream Right Wingwall

Photo 5 - November 6, 2003
Bellrock Dam
Bellrock Dam - Downstream Right Wingwall

Photo 6 - November 6, 2003
Bellrock Dam
Bellrock Dam - Steel Deck

Photo 7 - November 6, 2003
Bellrock Dam
Sluice Structure - Slot (Gain) Steel

Photo 8 - November 6, 2003
Bellrock Dam
Bellrock Dam - Slot (Gain) Steel

Photo 9 - November 6, 2003
Bellrock Dam
Bellrock Dam - Overflow Weir

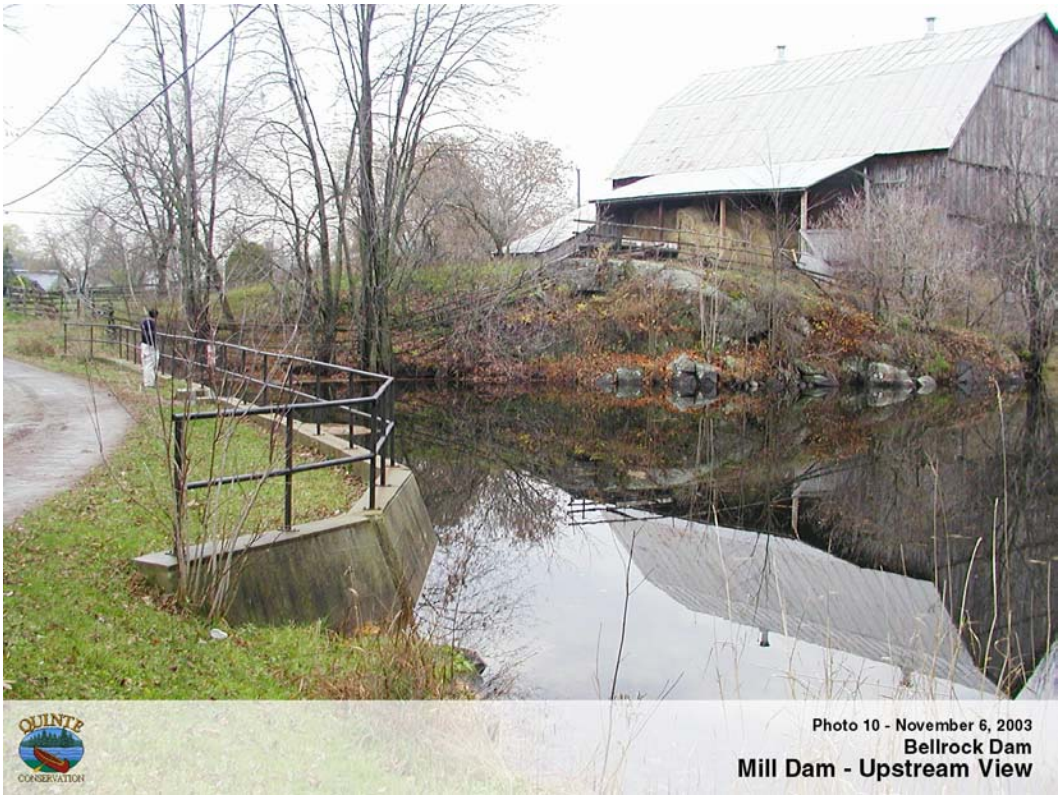

Photo 10 - November 6, 2003
Bellrock Dam
Mill Dam - Upstream View

Photo 11 - November 6, 2003
Bellrock Dam
Mill Dam - Adjacent Mill

Photo 12 - November 6, 2003
Bellrock Dam
Mill Dam - "Danger - Dam" Sign

Appendix B

Forms B1 and B2

Form B1

Quinte Conservation

Pre-Inspection Background Information

Prepared By:	Acres International Limited
Name of Dam:	Bellrock Dam
Latest Construction:	Construction date is unknown. The dam is an overflow concrete with a central stop log sluice. A secondary structure, a retaining wall with sluiceway, exists about 50 m to the west along the reservoir edge.
Inspection Dates:	Mid-1970s
Access:	13 km upstream of Colebrook Dam
Reservoir Storage:	28 000 m^3
Reservoir Area:	0.012 km^2
Watershed:	Napanee River Watershed
Drainage Area:	186 km^2
Gauge Info:	A staff gauge is installed near the sluice gate on the right pier. The reading appears to be based on local datum.
Rule Curves:	Head-discharge and reservoir capacity curves
List of Drawings:	1977 concept drawing
Topographic Maps:	1:50 000 scale
Soil and Land-Use Maps:	Not applicable
Dam Height:	2.4 m
Dam Length:	21 m

No. of Sluiceways:	One controlled by stop logs
No. of Stop Logs per Bay:	7 stop logs
Hydrologic Flows:	See Section 4.4.4, “Hydrotechnical Aspects” of Data Collection and Site Inspection - Bellrock Dam.
Hydraulic Analysis:	See Section 4.4.4, “Hydrotechnical Aspects” of Data Collection and Site Inspection - Bellrock Dam.
Dam Operation:	Stop logs remain year round with operation limited to occasional removal to facilitate clean-out of the headpond or other such infrequent requirements (i.e., low flow augmentation).
Soils Reports:	None available
Underwater Inspections:	Not applicable
Divestment Opportunities:	None identified
Known Problems:	None identified
Summary of File:	Quinte Conservation Bellrock Dam

Form B2

Quinte Conservation

Dam Inspection Report

Date: November 5, 2003

Structure: Bellrock Dam

Location: In the Town of Bellrock at Depot Creek

Inspected By: F. Chidiac, A. Touhidi and R. Zhou of Acres International

Weather: Cloudy, 4°C

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1. Earth Embankment (including emergency spillway)

Not applicable.

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2. Concrete Structures (wingwall, concrete overflow weir, stop log opening)

For details, see the photographs in Appendix A and the assessment drawings in Appendix C.

Results from inspection of the Bellrock Dam are summarized as follows:

- **Right Wingwall**
  - Some map cracking was noticed on the crest of the wall.
  - Light spalling was observed at the junction wall/pier with no sign of erosion at the waterline.
  - A major concrete deterioration was observed at the downstream face of the wall.

- **Stop Log Opening**
  - Rough concrete surfaces were noticed at the waterline inside the piers.
  - The topside of the concrete piers was showing some map cracking with spalling observed near the stop log gains.

- **Concrete Overflow Weir**
  - The weir was not inspected due to spillage.

The mill dam was in good condition.

---

3. Wooden and Metal Structures (decks, gains, railings, conduits, etc)

For details, see the photographs in Appendix A and the assessment drawings in Appendix C.

Deck allowing crossing from the right wingwall to the stop log guides is of steel grating. Guards are fabricated of steel pipe. Slot (gain) steel was not severely rusted above the waterline. The steel deck was in good condition and well-fixed to the concrete piers. The steel deck components need some painting.

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4. Gates and/or Stop Logs (type and number)

For details, see the photographs in Appendix A and the assessment drawings in Appendix C.

The number of logs per bay could not be determined due to water spilling above the logs. No logs on deck. As reported by the dam operator, seven stop logs remain in the stop log sluice year round with operation limited to occasional removal to facilitate clean-out of the headpond or other such infrequent requirements (low flow augmentation).

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5. Water Level Gauge (reading and condition)

A staff gauge is installed near the sluice gate on the right pier. The reading appears to be based on local datum.

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6. Winches (type and number)

For details, see the photographs in Appendix A and the assessment drawings in Appendix C.

None available.

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7. Valves (type and number)

Not applicable.

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8. Boom (driftwood, chains, anchors)

There is no boom at the dam.

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9. Erosion (upstream and downstream)

For details, see the photographs in Appendix A and the assessment drawings in Appendix C.

The rock at the left abutment was exposed.

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10. Seepage or Leaks

For details, see the photographs in Appendix A.

Leakage was detected at the right wingwall.

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11. Access Route (location of gate keys, winch handles and keys)

13 km upstream of Colebrook.

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12. Safety Issues (public and operator)

For details, see the photographs in Appendix A.

See Item 13, Signage, and Item 3, Wooden and Metal Structures.

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13. Signage

For details, see the photographs in Appendix A.

- “Dam - Danger” sign on upstream sluice in good condition.
- “No Trespassing” sign on access (left and right sides).

---

14. Divestment and/or Decommissioning Opportunities

None identified.

---

15. General Remarks

Stability against overturning and sliding of this structure was seen as an area of concern.

---

16. Recommendations

- Install a “No Trespassing” sign at the right and left sides of the structure.
- Monitor leakage at the right wingwall and left abutment.
- Perform stability assessment and recommend stabilizing measures.
- The requirement for a log boom should be reviewed.

Appendix C

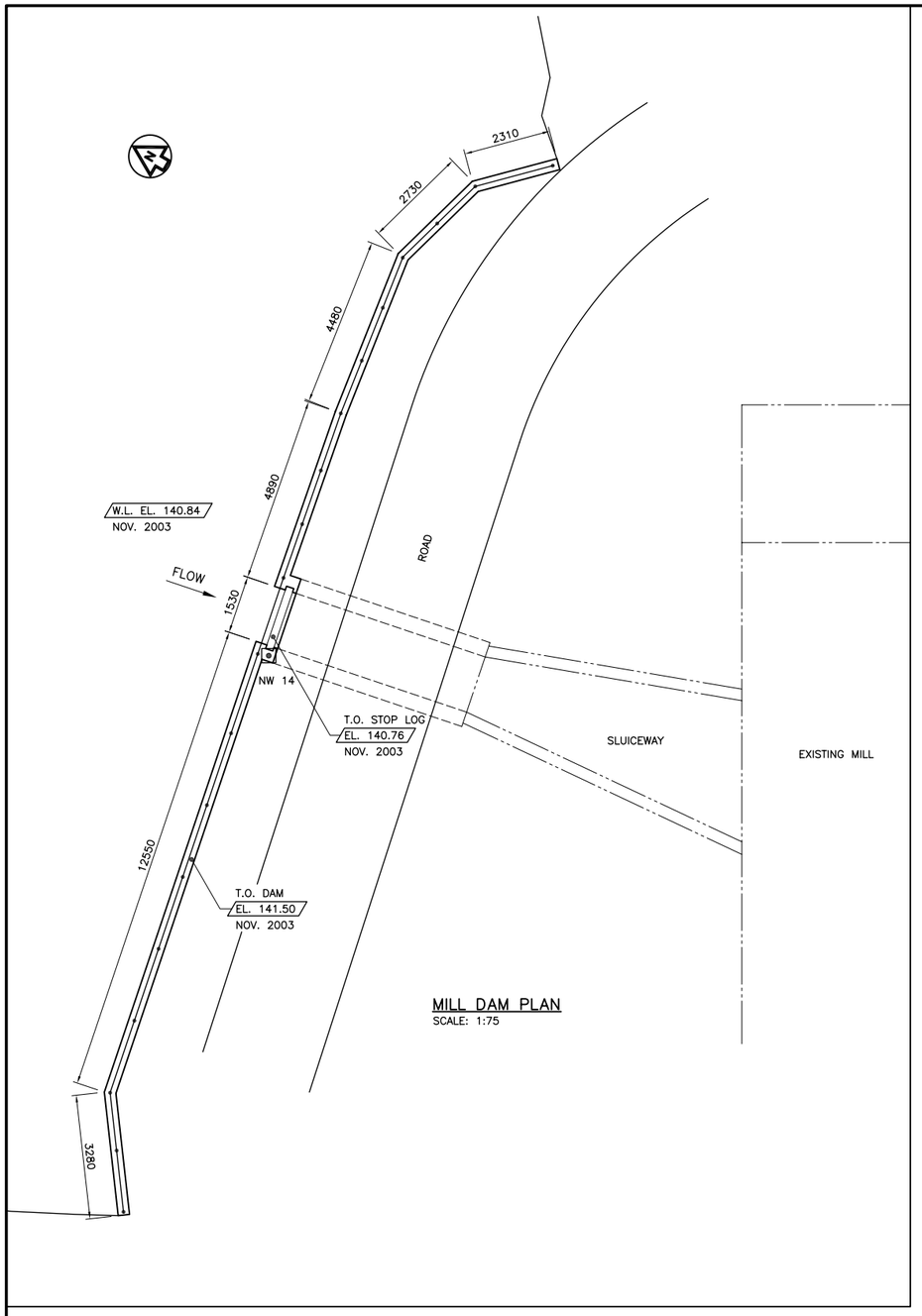
Assessment Drawings

Appendix C

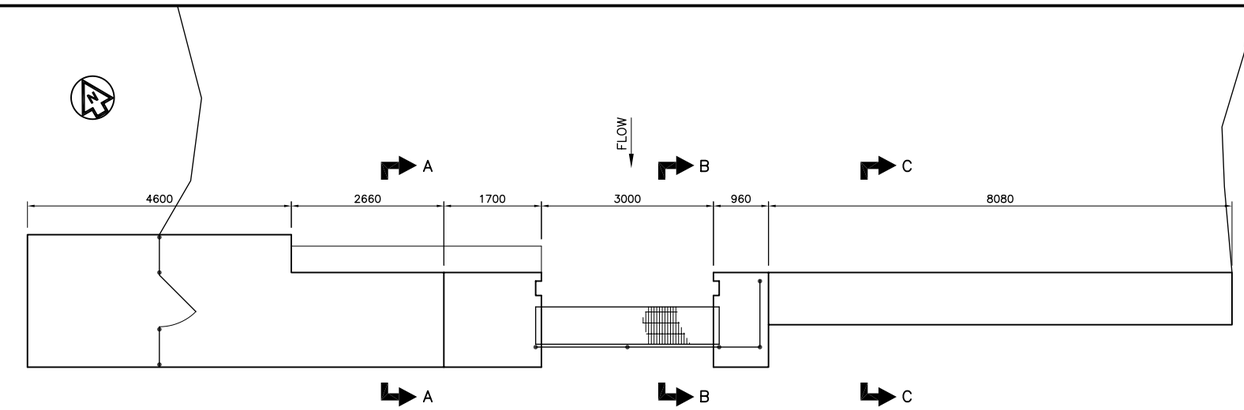
Assessment Drawings

List of Drawings

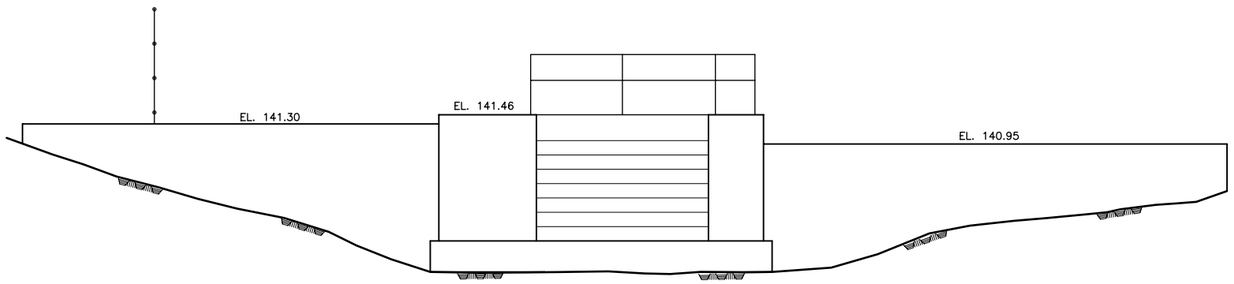
Drawing Number	Revision Number	Title
15396-BD-C-001	A	Bellrock Dam Plans, Elevation and Sections
15396-BD-C-002	A	Bellrock Dam Concrete Condition Plan and Elevations



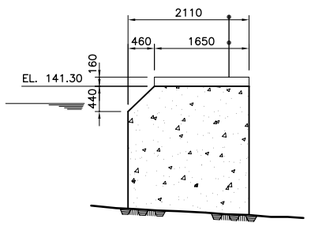
MILL DAM PLAN
SCALE: 1:75



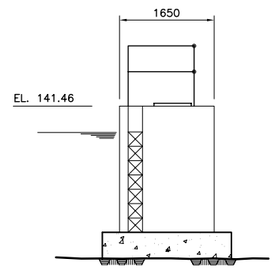
BELLROCK DAM PLAN
SCALE: 1:50



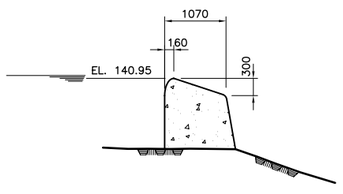
BELLROCK DAM ELEVATION (LOOKING UPSTREAM)
SCALE: 1:50



SECTION A-A
SCALE: 1:50



SECTION B-B
SCALE: 1:50



SECTION C-C
SCALE: 1:50

LEGEND

STRUCTURE SHOWN IN SECTION CONCRETE

IB 13 IRON BAR

NW 14 NAIL AND WASHER

NOTES

1. REFERENCE DRAWING:
   INSPECTION SUMMER 1977, FIGURE 4–7.
2. ELEVATIONS OF MILL DAM ARE FROM SURVEY INFORMATION BY GLOBAL SURVEYING SERVICES, NOV. 2003.
   ELEVATIONS OF BELLROCK DAM ARE FROM FIGURE 4–7.
3. VERTICAL DATUM CGD 1928, BASED ON BM 883005.
   HORIZONTAL NETWORK NAD83.

CONTROL	NORTHING	EASTING	ELEVATION
IB 13	4 926 275.862	359 805.117	142.577
NW 14	4 926 267.046	359 853.993	141.515

4. ALL DIMENSIONS ARE IN MILLIMETERS.
   ALL ELEVATIONS ARE IN METERS.

Scale 1:50 0 1 2 3 4 5 Metres
Scale 1:75 0 1.5 3.0 4.5 6.0 7.5 Metres

DATE	NO.	ISSUE / REVISION	CH.	APP.	APP.
DEC. 12, 2003	A	ISSUED WITH INSPECTION REPORT			

ACRES

QUINTE CONSERVATION
BELLEVILLE, ONTARIO
DAM SAFETY REVIEWS - SECOND DEPOT, THIRD DEPOT AND COLEBROOK DAMS

ACRES INTERNATIONAL
DESIGN
PREPARED
CHECKED
DRAWING
PREPARED X. ZHAO
CHECKED
PROJECT DISCIPLINE LEAD
PROJECT ENGINEER
PROJECT MANAGER FADI CHIDIAC

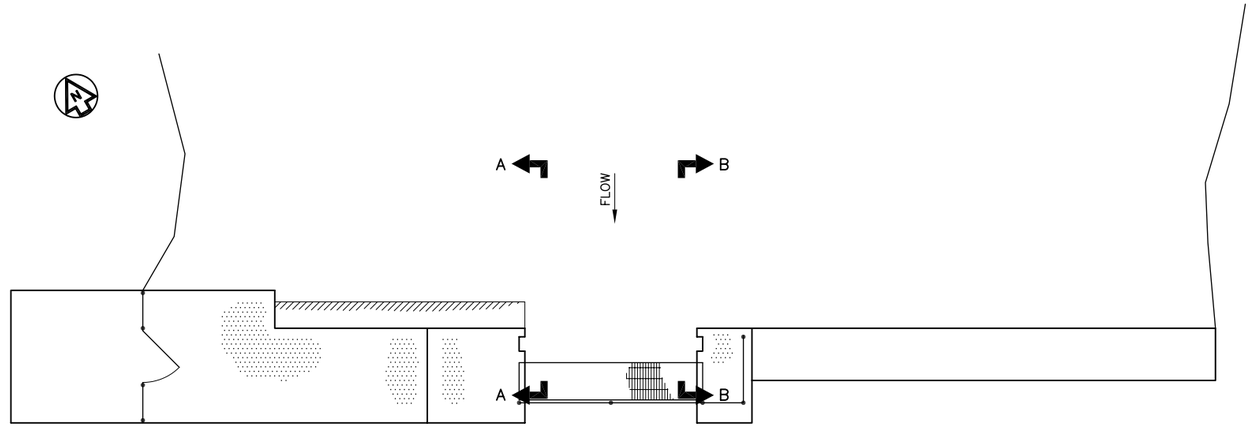
BELLROCK DAM

PLANS, ELEVATION AND SECTIONS

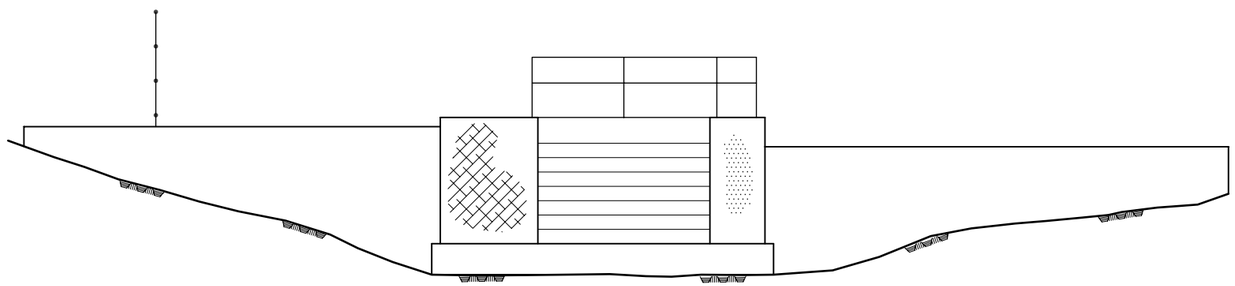
SCALE AS NOTED
ACRES PROJECT NO. P15396.00
DRAWING NO. 15396–BD–C–001
REVISION A

DRAWING No. 15396–BD–C–001 A

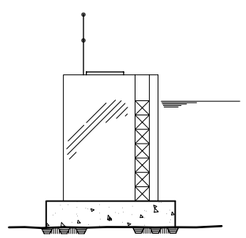
Plot Scale 1:1
Apr 19, 2004 8:41am Login name: XZhao
Drawing Name: R:\P15396\Civil\Bellrock\15396-BD-C-001.dwg



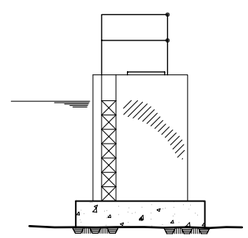
BELLROCK DAM PLAN
SCALE: 1:50



BELLROCK DAM ELEVATION (LOOKING UPSTREAM)
SCALE: 1:50



ELEVATION A-A
SCALE: 1:50



ELEVATION B-B
SCALE: 1:50

LEGEND

- MAJOR DETERIORATION
- EROSION AND ROUGH
- SPALLED
- C1 FINE CRACKING
- C2 MAJOR CRACKING
- STRUCTURE SHOWN IN SECTION

Scale 1:50 0 1 2 3 4 5 Metres

DATE	NO.	ISSUE / REVISION	CH.	APP.	APP.
DEC. 12, 2003	A	ISSUED WITH INSPECTION REPORT			

ACRES

QUINTE CONSERVATION
BELLEVILLE, ONTARIO
DAM SAFETY REVIEWS - SECOND DEPOT, THIRD DEPOT AND COLEBROOK DAMS

ACRES INTERNATIONAL
DESIGN
PREPARED
CHECKED
DRAWING
PREPARED X. ZHAO
CHECKED
PROJECT DISCIPLINE LEAD
PROJECT ENGINEER
PROJECT MANAGER FADI CHIDIAC

BELLROCK DAM
CONCRETE CONDITION
PLAN AND ELEVATIONS

SCALE AS NOTED
ACRES PROJECT NO. P15396.00
DRAWING NO. 15396-BD-C-002
REVISION A

DRAWING No. 15396-BD-C-002 A

Plot Scale 1:1
Apr 19 - 2004 , 8:43am Login name: XZhao
Drawing Name: R:\P15396\Civil\Bellrock\15396-BD-C-002.dwg

Part II
Dam Classification

Table of Contents

Part II – Dam Classification

LIST OF FIGURES

List of Figures

1 Introduction

1 Introduction

1.1 General

This Part II of the report presents the results of a preliminary classification for Bellrock Dam in terms of the incremental hazard potential (IHP) posed by the dam structure and the corresponding inflow design flood (IDF), based on procedures and guidelines given in the draft Ontario Dam Safety Guidelines (ODSG), [Ministry of Natural Resources (MNR), 1999] and on available information such as characteristics of the dam, reservoir, watershed, discharge facilities, downstream development, recreational activities, historical flooding, etc.

1.2 Dam Safety Assessment

The safety assessment of a dam comprises a procedural evaluation of the ability of a water-retaining structure to safely withstand all forces that could be expected to act on such a structure during its lifetime. For this purpose, a number of criteria have been developed to allow a systematic evaluation and classification of structures with respect to the potential failure risk it imposes. These criteria have been derived from most frequently observed types of failures of dams in the world and incorporate a classification system that addresses the following aspects:

- hazard classification
- flood handling capability evaluation
- dam break flooding evaluation
- structural integrity and stability assessment.

Figure 1.1 displays the dam safety assessment process, which is a graphical representation of full dam safety program. The hazard classification of a dam is determined by the degree of exposure or developments located on the river downstream from the dam, in terms of potential flood damage and loss of life (LOL) in case of a dam failure. Based on this assessment, a hazard classification index (HCI) is determined and a corresponding IDF is assigned. Figure 1.2, taken from the ODSG (MNR, 1999), lists the hazard classifications for dams. The HCI for a dam is also referred to as the IHP or incremental consequence category (ICC), which reiterates the hazard potential classification of a dam as that which will occur as a result of the failure of the dam, not the magnitude of the flood

event itself. These three terms are used interchangeably in this report and the corresponding IDF is the basis for all of the dam safety assessments. Figure 1.3, taken from the ODSG (MNR, 1999), stipulates a range on IDF magnitudes, depending upon hazard potential the dam poses.

A spillway is rated on the basis of its capacity as well as its capability to respond to emergency flood conditions on the river. These ratings are compared to the IHP and IDF requirement.

Except for calculations of spillway capacity and safety factors for stability of a structure, the dam safety assessment process allows for some qualitative interpretation where a strictly scientific approach is not possible. Generally, however, the guidelines are sufficiently well-defined to permit determination of dam safety conditions within practical and accurate limits.

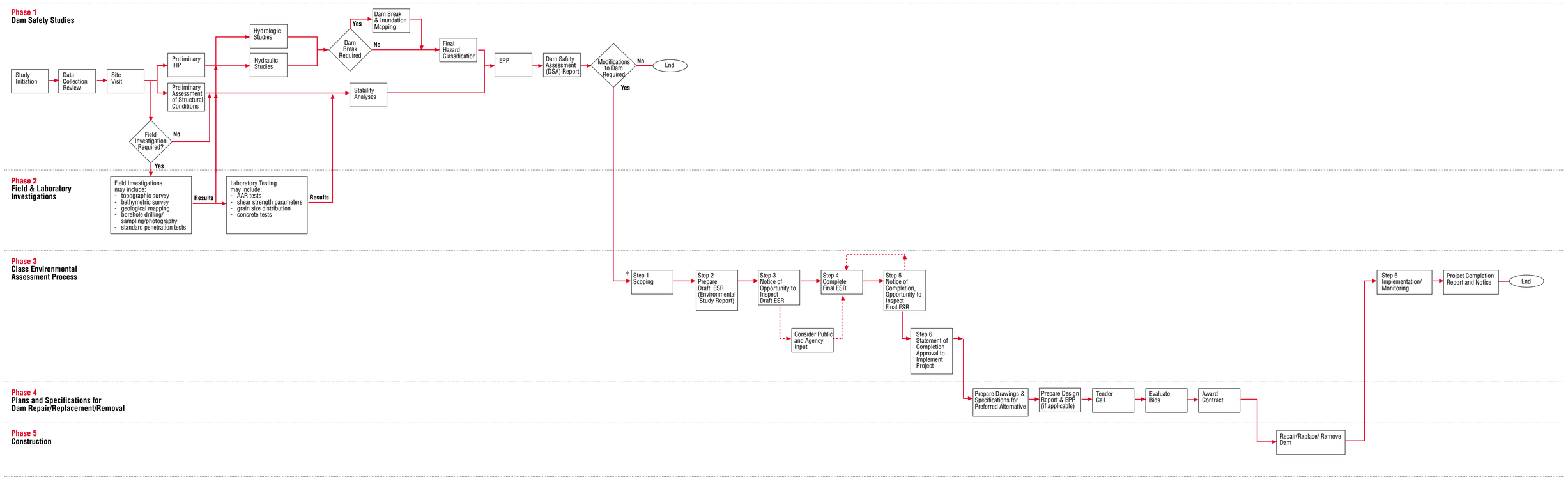


Figure 1.1
Quinte Conservation
Dam Safety Assessment Evaluation Activities

ACRES International

Figure 1.2: Hazard Potential Classification for Dams

(Source: MNR)

SELECTION CRITERIA

Hazard Potential	Loss of Life	Economic and Social Losses	Environmental Losses
Very Low	Potential for loss of life: None.	Damage to dam only. Little damage to other property. Estimated losses do not exceed $100,000.	Environmental Consequences: Short-term: Minimal Long-term: None
Low	Potential for loss of life: None. The inundation area (the area that could be flooded if the dam fails) is typically undeveloped.	Minimal damage to agriculture, other dams or structures not for human habitation. No damage to residential, commercial, industrial or land to be developed within 20 years. Estimated losses do not exceed $1 million.	No significant loss or deterioration of fish and/or wildlife habitat. Loss of marginal habitat only. Feasibility and/or practicality of restoration or compensating in kind is high, and/or good capability of channel to maintain or restore itself.
Significant	Potential for loss of life: None expected. Development within inundation area is predominantly rural or agricultural, or is managed so that the land usage is for transient activities such as with day-use facilities. There must be a reliable element of warning if larger development exists.	Appreciable damage to agricultural operations, other dams or residential, commercial, industrial development, or land to be developed within 20 years. Estimated losses do not exceed $10 million.	Loss or significant deterioration of important fish and/or wildlife habitat. Feasibility and/or practicality of restoration and/or compensating in kind is high, and/or good capability of channel to maintain or restore itself.
High	Potential for loss of life: One or more. Development within inundation area typically includes communities, extensive commercial and industrial areas, main highways, public utilities and other infrastructure.	Extensive damage to communities, agricultural operations, other dams and infrastructure. Typically includes destruction of or extensive damage to large residential areas, concentrated commercial and industrial land uses, highways, railways, power lines, pipelines and other utilities. Estimated losses exceed $10 million.	Loss or significant deterioration of critical fish and/or wildlife habitat. Feasibility and/or practicality of restoration and/or compensating in kind is low, and/or poor capability of channel to maintain or restore itself.

* Supporting References: MNR Guidelines for Approval Under the Lakes and River Improvement Act, 1977
MNR Fisheries Section, 1999
US Army Corps of Engineers, Dam Safety Assurance Program, 1995
Dam Structure Assessment Program, Ontario Hydro, 1990

Notes:

1. Consideration should be given to the cascade effect of dam failures in situations where several dams are situated along the same watercourse. If failure of an upstream dam could contribute to failure of a downstream dam(s), the minimum hazard potential classification of the upstream dam should be the same as or greater than the highest downstream hazard potential classification of the downstream dam(s).

2. Economic losses refer to all direct and indirect losses to third parties; they do not include losses to owner, such as loss of the dam, associated facilities and appurtenances, loss of revenue, etc.

3. Estimated losses refer to incremental losses resulting from failure of the dam or misoperation of the dam and appurtenant facilities.

For Hazard Potential Classification and Safety Criteria for tailings dams, refer to “Guidelines for Proponents, Rehabilitation of Mines”, issued by Ontario Ministry of Northern Development and Mines, 1995.

Figure 1.3: Minimum Inflow Design Floods for Dams

(Source: MNR)

Hazard Potential	Size of Dam and Inflow Design Floods*					
	Small		Medium		Large	
	Height < 7.5 m	Storage < 100 x 10^3 m^3	Height 7.5 to 15 m	Storage 100 x 10^3 to 1000 x 10^3 m^3	Height > 15 m	Storage > 1000 x 10^3 m^3
Damage to Dam only (very low)	25-year flood to 50-year flood		50-year flood to 100-year flood		100-year flood to Regulatory flood	
Low	25-year flood to 100-year flood		100-year flood to Regulatory flood		Regulatory flood to PMF	
Significant	100-year flood to Regulatory flood		Regulatory flood to PMF**		PMF**	
High	Regulatory flood to PMF**		PMF**		PMF**	

* For Minimum Inflow Design Floods for Mine Tailings dams, refer to "Guidelines for Proponents, Rehabilitation of Mines", issued by Ontario Ministry of Northern Development and Mines, 1995.

** Where Inflow Design Flood (IDF) less than PMF is to be adopted through risk-based approach for existing Significant and High Hazard dams, as indicated above, the proposal and the evaluations shall be subject to independent review and verification, as described in Section 3. IDF for these dams shall not be less than 1/2 PMF.

2 Dam Classification

2 Dam Classification

The consequences of a dam failure are assessed in terms of the incremental hazard posed by the structure, based on guidelines and procedures given in the draft ODSG (MNR, 1999). The IHP is defined as the potential for increase in LOL, property damage, disruption in social and economic activities and/or environmental impacts caused by failure of the dam structure incrementally above that which would have occurred without failure of the dam.

The initial, or preliminary, classification of a dam is commonly done on the basis of a conservative visual assessment of the potential for incremental losses. In cases where this preliminary classification results in a structure being deemed as a 'HIGH' IHP structure, a formal final classification is often determined by simulating dam break floods and assessing the effects of the resultant downstream flood inundation. This additional assessment is usually done to allow a somewhat less conservative selection of the IDF and the design base earthquake (DBE), and to allow accurate inundation maps to be prepared for emergency preparedness planning purposes. It is not necessarily, however, a strict requirement to assess the safety of a dam.

In all cases, the classification of the dam flows out of the estimate of incremental losses. These losses are estimated for two assumed dam failure scenarios. A normal (or sunny day) event in which the failure of the dam is assumed to have been initiated as the result of a chance event, such as an earthquake, that could occur at any time and cannot be predicted in advance. The dam is commonly assumed to be operating under normal water level conditions with no opportunity to operate flow control equipment. The second assumed dam failure event is associated with the IDF. In this case, it is generally assumed that recreational users downstream and upstream of the dam will not be at risk (since there is some warning time available with a major flood event) and that the flow control equipment is operational.

For this study, the IHP was selected on the basis of available information (e.g., characteristics of the dam, reservoir, watershed, discharge facilities, downstream development, recreational activities, historical flooding, etc). This preliminary IHP was then used to determine the IDF and the DBE for the site on the basis of the recommendations of the draft ODSG (MNR, 1999).

2.1 Sunny Day Dam Failure

The Bellrock Dam is approximately 2.5 m high and impounds 28 000 m^3 of water in the reservoir. This places the dam as a SMALL dam with SMALL storage.

There are permanent residences downstream from the dam. Land use surrounding the impoundment is largely forest with some agriculture and residential usage near the town of Bellrock. There is no potential for LOL. The socioeconomic, financial and environmental losses are very limited if the dam fails given its small size. The flood flow due to the failure would be quickly reduced in the downstream channel and the wetland area.

It should be noted that the Bellrock Dam is located downstream from the Second and Third Depot Lake dams. There is a possibility that this dam could fail due to the upstream dam's failure. However, given the small size of this dam and its impoundment, it is not very likely that the cascade failure of this dam will contribute significant incremental flood damages to the downstream area.

Based on the above conditions, the Bellrock Dam is classified as a VERY LOW IHP for sunny day failure conditions.

2.2 Flood Dam Failure

The Bellrock Dam is acting as an overflow weir, and hence, it is not expected to fail when the IDF occurs. Under flood conditions, if the Bellrock Dam fails, the incremental flood damage to the downstream area is insignificant since the storage behind the dam is very small and water released would result in a small incremental inundation area. For this reason, the incremental flood damage is limited and there would be no LOL as a consequence of the dam failure. The IHP assigned to this dam is, therefore, VERY LOW.

2.3 Inflow Design Flood

Based on the draft ODSG for the above IHP, the IDF for the Bellrock Dam is determined to be the 50-yr flood.

Part III
Hydrotechnical Assessment

Table of Contents

Part III – Hydrotechnical Assessment

List of Tables

List of Figures

1 Introduction

1 Introduction

1.1 General

This Part III of the report presents the results of hydrologic and hydraulic assessments for Bellrock Dam as follows:

- Section 2 - Hydrological/Hydraulic Assessment - Approach and Methodology

- Section 3 - Hydrological/Hydraulic Assessment

  The section includes the following main topics:

  - descriptions of river basin characteristics
  - development of flood and storm events
  - development of rainfall/runoff and flood routing models
  - flood flow estimates.

2 Hydrological/Hydraulic Assessment – Approach and Methodology

2 Hydrological/Hydraulic Assessment – Approach and Methodology

2.1 Hydrologic Analysis

The purpose of the hydrologic analyses was to estimate peak flood flows and hydrographs for the 2-yr, 5-yr, 10-yr, 20-yr, 50-yr and 100-yr return period floods, and the probable maximum flood (PMF).

The deterministic modeling of watershed runoff on an event basis method was applied for estimating peak flows.

The Bellrock Dam is not located at or near appropriate Water Survey of Canada (WSC) streamflow gauging stations. Application of transposed or regional runoff flood characteristics for dam safety use requires verification, which can be accomplished by deterministic modeling or regional flood frequency estimation techniques. The regulatory flood for the study basin is the flood resulting from the 100-yr rainstorm, which is well-documented. The duration and temporal distribution of the rainfall amounts in this regional storm event are provided in this reference, and estimates of the resulting flood runoff from an applicable watershed could only be synthesized by using a rainfall-runoff model.

The deterministic peak flow estimates were derived using the Guelph All-Weather Storm Event Runoff Model (GAWSER) (Schroeter and Whiteley, 1987) for the damsite and are summarized in Section 2.2.1.

2.2 Rainfall-Runoff Modeling

2.2.1 GAWSER Package

(a) Selection of Model

The GAWSER Package is a computer model for rainfall-runoff analysis, developed by the Guelph University. This model was selected for application to the basin of the dam because of its ability to develop discharge hydrographs for hypothetical rainfall events at one or more locations in a basin and its general versatility as an event model. More importantly, this model has been used and calibrated in the study watershed for flood forecast. The model is capable of

representing a single runoff event occurring over a period of time, typically several days, utilizing an appropriate calculation time step, usually 1 hour, to accurately compute runoff from storm event rainfall. It allows a wide variety of options for specifying precipitation, losses, base flow, runoff transformation and routing. In comparison to other event-simulation models, GAWSER is relatively compact and is able to execute a variety of computational procedures in a single computer run.

(b) General Description of Model

The GAWSER model (Schroeter and Whiteley, 1987) is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process over the entire watershed, or within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters that specify the particular characteristics of the component and mathematical relations, which describe the physical process. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.

(c) Setup of GAWSER Model

The flood forecast model for Moria River, Salmon River and Napanee River was developed in 2000 by Schroeter & Associates. The model consists of a complicated watershed configuration, including three major river systems in the QC area. The Bellrock Dam was modeled as subwatershed 417. Upstream of the dam, there are several subwatersheds, including Second, Third, Fourth and Fifth Depot Lakes. For the purpose of this analysis, the only changes made to the existing model were the flow-stage-storage rating curves, reflecting the operation policies for different seasons, and using newly surveyed elevation in Canadian Geodetic Datum.

(d) Input Data

Physical parameters for each river basin, including drainage area, stream course length and average slope, from the existing GAWSER model were adopted for this analysis.

- **Precipitation:** Precipitation over the drainage area was determined from the Kingston Station (No. 6104146). A representative temporal distribution of the average rainfall for the study area was applied to calculate the input hyetograph.

- **Losses:** The runoff volume for each subbasin was computed by the US Soil Conservation Service (SCS) CN method with an optional initial loss representing antecedent moisture condition (AMC II). Details of the loss computations can be found in the Moira River integrated flood forecast system final technical report (Schroeter & Associates, 2000).

- **Stream Channel Routing:** Routing of hydrographs through channels in the GAWSER model is accomplished by rating curve method based on typical channel cross section for each river reach. Some of the watersheds in the study area have limited reaches with large river channels and are dominated by large lakes or reservoirs, located immediately upstream of the dams. The channel routing option was, therefore, not required.

- **Reservoir Routing:** Storage routing techniques were used by the model to route flows through lakes or reservoirs located at the damsite. Applicable elevation/outflow relationships were derived from available site facility information. A discharge rating curve for the lake or reservoir was developed based on structure discharge characteristics and input to GAWSER to compute the reservoir outflows.

- **Rainfall Excess To Runoff Transformation:** Precipitation excess was transformed to direct runoff using the unit hydrograph technique. The unit hydrograph adopted was expressed in terms of the unit hydrograph parameters. Details can be found in the Moira River Study Report (Schroeter & Associates, 2000).

(e) Input Rainfall Data

Precipitation data, which is required for input to the event model, is described in Section 2.2.2.

2.2.2 Storm Event Data Analysis

(a) Precipitation Data

Precipitation data are required as the driving input to the GAWSER model. These data are required on an event basis and to provide an appropriate calculation resolution between runoff volume, peak discharge and response time of the various drainage basins.

Floods vary greatly in intensity and duration, depending on storm patterns, drainage basin characteristics and other factors. A summer storm on a small drainage basin may generate a flood with a very high peak flow but of short duration. On a large basin, the peak flow from a similar storm may be significantly attenuated by storage and resistance in the catchment before it reaches the basin outlet. Spring rain on snowmelt events, on the other hand, are likely to be of lesser intensity but of much greater area extent and longer duration. The runoff volume is the dominant factor resulting in flood flows for this type of flood events. Unused storage capacity in a catchment that may be sufficient to attenuate peak runoff and present significant flooding from a summer storm of short duration may be ineffective for a severe event of this type. Both types of flood events need to be analyzed to determine the design flood.

Therefore, two types of design events were analyzed and used in the study. The first type of design precipitation event is the summer design storm event. The other type of design event is the rain on snowmelt conditions. Data from Kingston meteorological station were analyzed and applied for the analysis.

Station ID	Name	Period of Record	Years
6104146	Kingston	1968 to 1994	27

The station has relatively long period of records and is located close to the study area. It is realized that there are several new meteorological stations installed within or near the study watershed for flood forecasting purposes. Use of these data would be desirable if they contain long-term records. However, none of these station's data could be used to obtain reliable statistics for developing design storms due to their short recording periods. On the other hand, it is more critical to use these station data for real time flood forecasting, since the rainfall amount varies significantly both in time distribution and in geological locations. However, the extreme storms do not change significantly within a geological region, since they do not necessarily come from the same storm. The data from the Kingston station were applied in the simulation model.

(b) Design Storms and Temporal Distributions

A design storm consists of three important factors: storm volume, duration and distribution.

Rainfall Depth-Duration-Frequency Relationship

Rainfall depth-duration-frequency (DDF) or intensity-duration-frequency data are available in the form of tables and graphs in Atmospheric Environment Service (AES), Environment Canada. AES provides both short duration DDF (from 5 minutes to 24 hours) and long duration DDF (from 1 day to 30 days) design storms. The DDF data are based on statistical analysis of long-term rain gauge records in a region. Maximum cumulative rainfall amounts for 1 day to 30 days were fitted to a modified Gumbel extreme value distribution. Total precipitation for any return period could then be obtained from the fitted distribution.

Before a design storm can be developed from AES data, two storm parameters must be determined: the duration of the storm and the time intervals for each rainfall increment. The duration needed is directly related to the time of concentration of the basin, as determined from an analysis of recorded data or by computation. The duration should be at least as long as, but preferably longer than, the time of concentration. A duration less than the time of concentration would not allow all parts of the basin to contribute

runoff simultaneously at the outlet during the course of the storm. Runoff from the lower parts of the basin would have left the basin before runoff from the upper parts of the basin had reached the outlet and the estimated peak discharge would be too low. A 3-day storm duration was used in the simulations because of the long time of concentration of the drainage area to the damsite. A long duration storm is required to capture the attenuation effects of the large natural storage areas in the watershed.

The time interval of storm increments should be small enough to define accurately the flood hydrograph. The time intervals of storm increments used in the study were 60 minutes.

The plotted results for rainfall DDF curves are presented in Figure 2.1 for the meteorological station. The results are also summarized in Table 2.1.

Time Distribution

A design storm developed from AES data is sometimes referred to as a 'balanced' storm because its incremental depth may be arranged in a consistent depth-frequency relationship for each duration interval of the total storm. In other words, different duration intervals, 1 hour, 6 hours, etc, produce rainfall depth with the same percent chance exceedance. This consistent relationship was used to ensure an appropriate depth of rainfall for a given frequency regardless of the time response characteristics of a particular river basin. The 'balanced' storm distribution is plotted in Figure 2.2 for the 1:100-yr storm for the Kingston meteorological station. The storms of other recurrence interval are assumed to follow the same distribution pattern.

AES DDF curves describe the variation of point rainfall with time for a given frequency. The curves do not include an adjustment for the variation with space and area. When simulations are undertaken for a watershed larger than 25 km^2, an area reduction to point rainfall is required in accordance with the Technical Guidelines for Floodplain Management in Ontario. The area reduction factors were determined for the watershed based on the

Figure 2.1: Rainfall Volume-Duration-Frequency Kingston

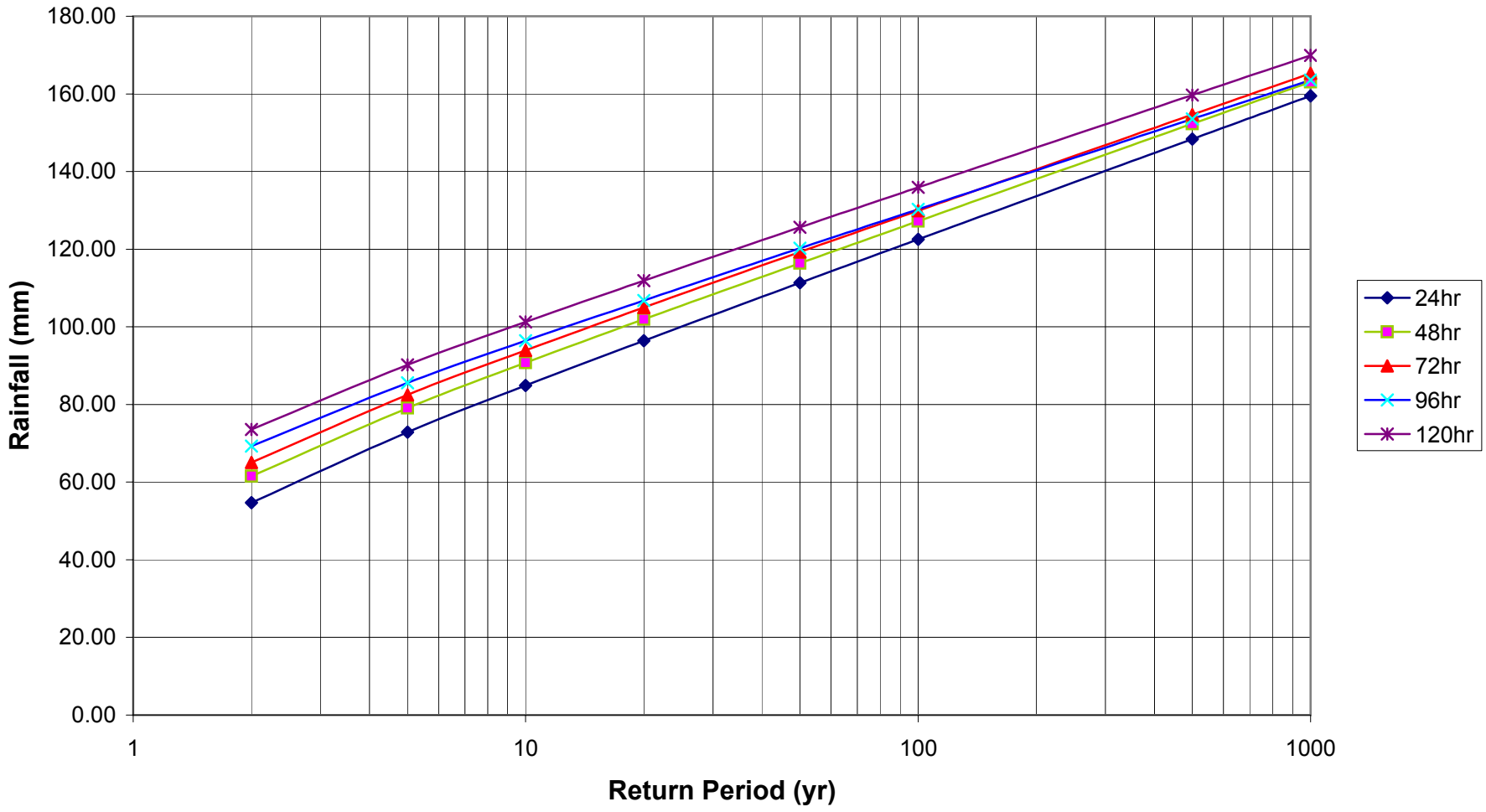


Table 2.1

Kingston Rainfall Design Events
Station 6104146

Return Period (yrs)	Rainfall Volume (mm)				
	24 Hours	48 Hours	72 Hours	96 Hours	120 Hours
2	54.70	61.51	65.09	69.26	73.53
5	72.86	79.11	82.45	85.60	90.23
10	84.88	90.76	93.95	96.42	101.29
20	96.41	101.93	104.98	106.79	111.90
50	111.33	116.40	119.26	120.23	125.63
100	122.52	127.24	129.95	130.29	135.92
500	148.36	152.29	154.68	153.55	159.70
1000	159.47	163.06	165.30	163.55	169.92

size of the basin drainage area. A reduction factor of 97.8% was applied to the Bellrock Dam.

(c) Rain-on-Snowmelt Event

Rain plus snowmelt event DDF data were derived using data obtained from AES, Environment Canada. According to the sizes and the incremental consequence category (ICC) classifications for the dam, the 3-day snowmelt event derived from AES data was applied. The rain-on-snowmelt design events were derived using daily mean temperatures, daily rainfall total and daily depth of fresh snow measurements by ruler.

Daily snowmelt estimates were calculated using degree-day type equations. Five different equations can be used, and the Model 5, which is suitable for Ontario, was selected. The snowmelt equation takes the following form:

$$SM = 3.66T_h$$

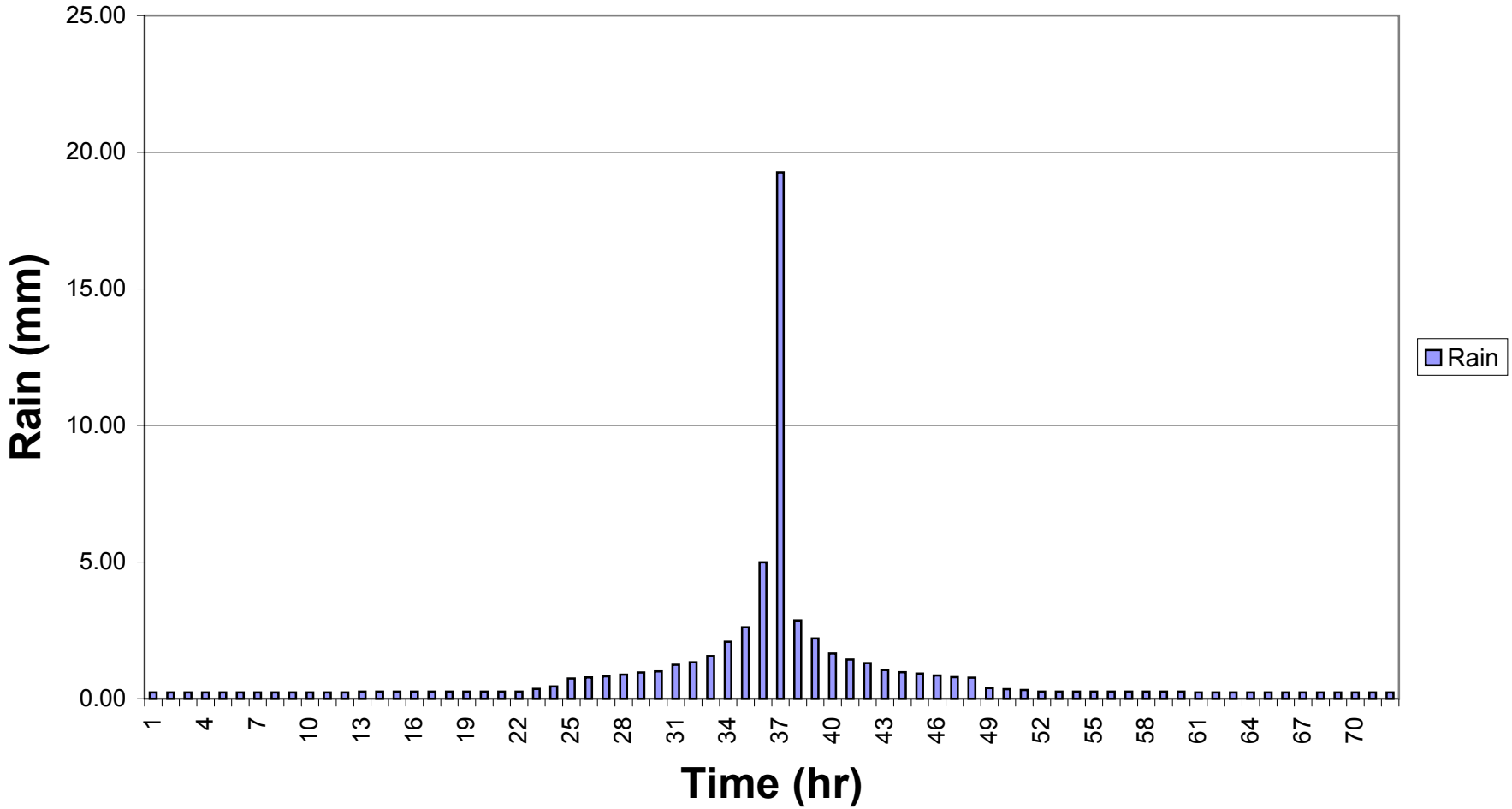
where,

SM = snowmelt (mm/day)
T_h = mean daily air temperature (°C)

The algorithm to estimate the snowmelt is based upon the synthetic snowpacks that are accumulated or reduced according to daily snowfall and snowmelt measurements. The algorithm was developed according to the historical air temperature versus snowmelt rate relationships for Ontario. The algorithm ceases to operate when the synthetic snowpack is reduced to zero. Daily rainfall is added to the daily snowmelt as calculated by the model, and the maximum of the combined rain plus snowmelt values is used to determine the annual maximum series for the different durations.

Maximum annual values for the rainfall plus snowmelt model were then estimated for different durations. Assuming a Gumbel Extreme Value distribution and using a method of moment, the data sets were fitted by the frequency curve. The results for the Kingston

Figure 2.2: 100-Yr Rainfall Distribution Kingston



meteorological station are summarized in Figure 2.3 and in Table 2.2, respectively.

The 3-day snowmelt distribution is shown in Figure 2.4.

(d) Regional Storm

The regional storm for the study area is the 100-yr storm based on the Floodplain Management Guidelines.

(e) Probable Maximum Precipitation (PMP) Storm

The Bellrock Dam is classified as a VERY LOW ICC. The inflow design flood (IDF) is, therefore, the 50-yr flood.

(f) Event Modeling

The structure was evaluated for a wide range of precipitation events, with return periods of 2, 5, 10, 20, 50 and 100 years, and the PMP. The damsite is directly associated with a regulating storage lake. This makes the volume component of a storm event more important, in comparison to the peak flow generated by the event. It is possible that a precipitation event, with a given return period, may yield different flood flow conditions with the same probability of occurrence, depending on starting water level and outlet structure manipulation, the durations, temporal patterns and intensities of the storms. Initial conditions were examined carefully in this study to ensure reasonable equivalence of overall event severity.

2.3 Hydraulic Capacity

A hydraulic analysis was performed to determine the discharge capacity of the dam. For the overflow weir section, the discharge is a function of hydraulic head, which can be estimated by the weir equation for free flow conditions:

$$Q = CLH^{1.5}$$

where,

Q = flow (m^3/s)
C = weir coefficient, $C = 1.6$ for stop logs and weir, and $C = 1.5$ for dam crest

L = the length of weir (m)
H = the head of water (m).

Table 2.2

Kingston Rain + Snowmelt Events
Station 6104146

Return Period (yrs)	Rainfall Volume (mm)				
	24 Hours	48 Hours	72 Hours	96 Hours	120 Hours
2	42.61	56.75	67.52	78.78	86.96
5	53.26	70.17	85.07	99.41	110.24
10	60.30	79.05	96.69	113.06	125.66
20	67.06	87.57	107.84	126.16	140.44
50	75.81	98.59	122.27	143.12	159.58
100	82.36	106.86	133.08	155.82	173.92
500	97.51	125.95	158.07	185.18	207.07
1000	104.03	134.16	168.81	197.81	221.32

Figure 2.3: Rain + Snow Volume-Duration-Frequency Kingston

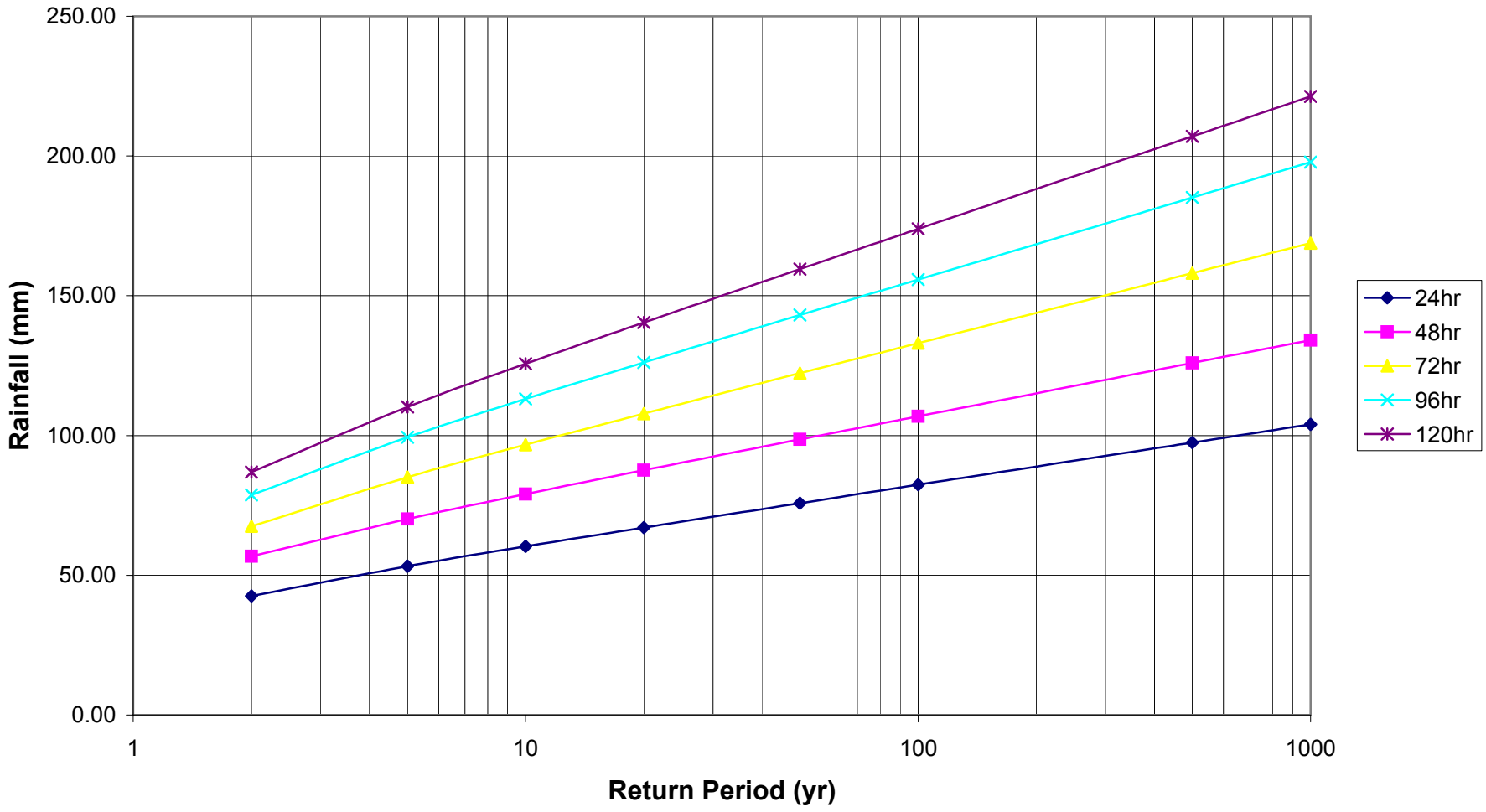
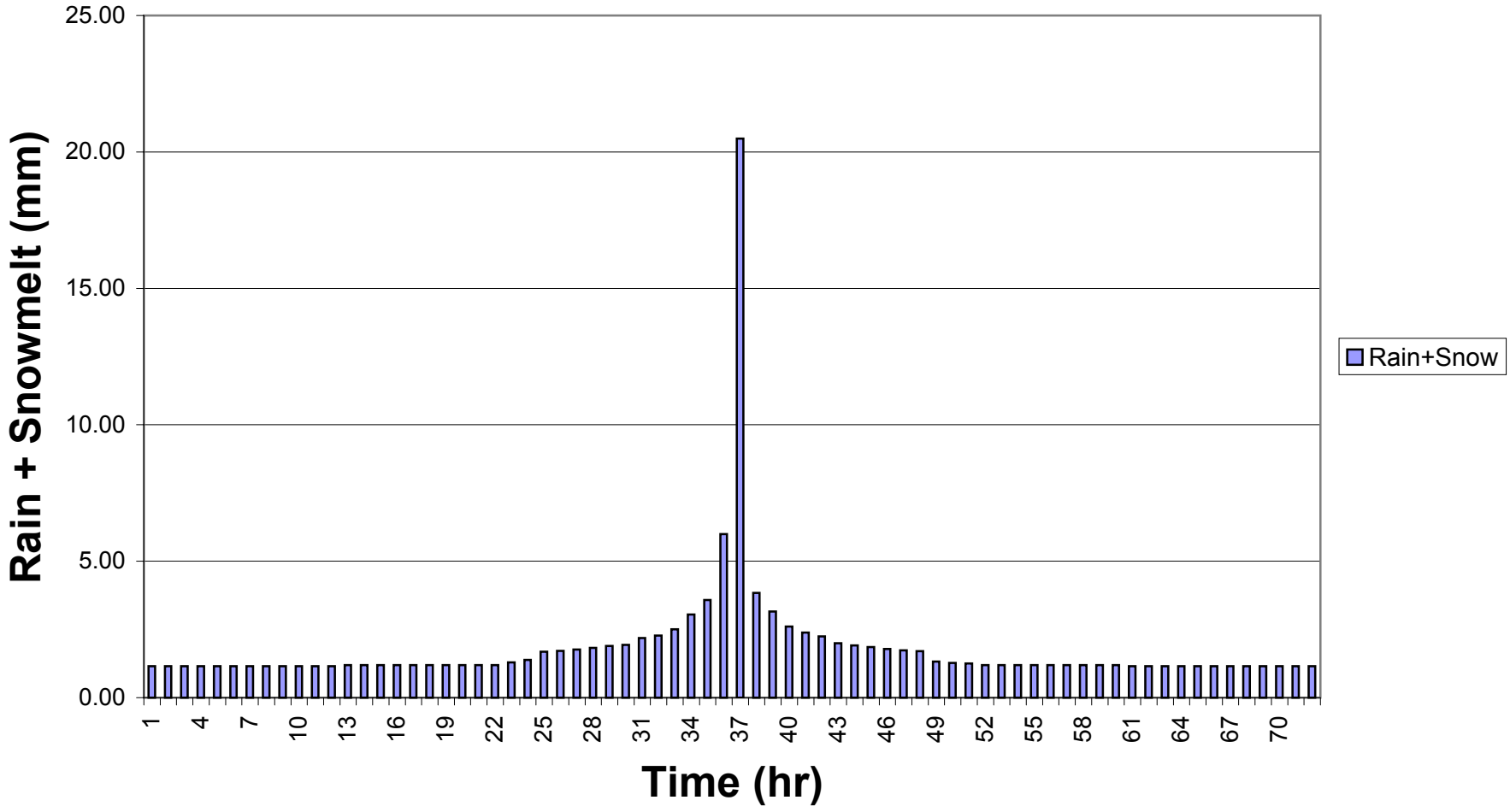


Figure 2.4: 100-Yr Rain + Snowmelt Distribution Kingston



3 Hydrological/Hydraulic Assessment

3 Hydrological/Hydraulic Assessment

3.1 Introduction

The Bellrock Dam is located in the town of Bellrock at Depot Creek. This damsite is approximately 13 km upstream of the Colebrook Dam. There is a sluiceway located near the Bellrock Dam upstream of the Mill to provide water for Mill operation. The height of the Bellrock Dam is approximately 2.5 m. The headpond at Bellrock Dam is small, which encompasses an area of approximately 11 600 m^2. The estimated storage at full supply level is approximately 28 000 m^3. The dam has an overflow weir on the left side (looking downstream), which is approximately 8.1 m wide, and one stop log bay, which is 3 m wide and contains a maximum of seven stop logs. The dam is not operated, and the stop logs are installed for both summer and spring conditions.

The schematic flow chart of a part of the GAWSER simulation model and the watershed system is presented in Figures 3.1a and 3.1b. In these figures, each subwatershed, flow routing reach and reservoir was assigned an identification number (ID). The flood routing processes from subbasin to subbasin are also indicated in the flow chart.

3.2 Basin Physiographic and Hydrologic Characteristics

The drainage area of the Bellrock Dam is approximately 186.4 km^2. The dam drains into the Cameron Swamp from the north and then discharges into the Napanee River, which empties into Lake Ontario via Quinte Bay.

Physiographic characteristics for the Bellrock Dam basin required as input to the GAWSER model were determined with the use of 1:25 000 topographic mapping (Schroeter and Associates, 2000). The drainage boundaries and the flow chart used to construct the GAWSER model are shown in Figures 3.1a and 3.1b. Details and description of the model can be found in the Moira River integrated flood forecast system technical report (Schroeter and Associates, 2000).

3.3 Storm Event Precipitation

Spring snowmelt plus rainfall events and summer design storms for the duration of 3 days at the Kingston meteorological station were used in the GAWSER model. A single precipitation event was assumed to apply to all individual basins (with proper area reduction factor) on the study area. The storms described in Tables 2.1 and 2.2 were applied in the simulations.

3.4 Reservoir Initial Conditions

Reservoir flow-elevation-volume relationships for the dam, which control impounded upstream storage, were derived for use in the GAWSER modeling. The discharge capacity of the dam was input to the model as an elevation-discharge-storage rating curve. Any additional spillway capacity at the dams, such as the overflow weir and the overtopping flow (assuming it might occur), was factored into the rating curves.

Since the dam is not operated, initial conditions at the damsite were set at el 140.98 m for spring and summer conditions.

3.5 Hydraulic Analysis

A hydraulic analysis at the Bellrock Dam was performed to evaluate existing spillway capacity and check on tailwater levels. Existing data and reports were reviewed. The upstream and downstream conveyance constraints were also evaluated.

The lake impounded behind the Bellrock Dam was reviewed, and an elevation-volume relationship was developed using the water surface area of the lake.

A summary of spillway capacity for the dam, along with the respective reservoir elevation-volume relationships, is summarized in Table 3.1 and in Figure 3.2.

3.6 GAWSER Model Flood Results

The results of the GAWSER simulations are presented in Table 3.2. Summarized are the storm rainfall return period values, as appropriate, and the peak inflow resulting from the rainfall transformation. The resulting peak inflow is presented

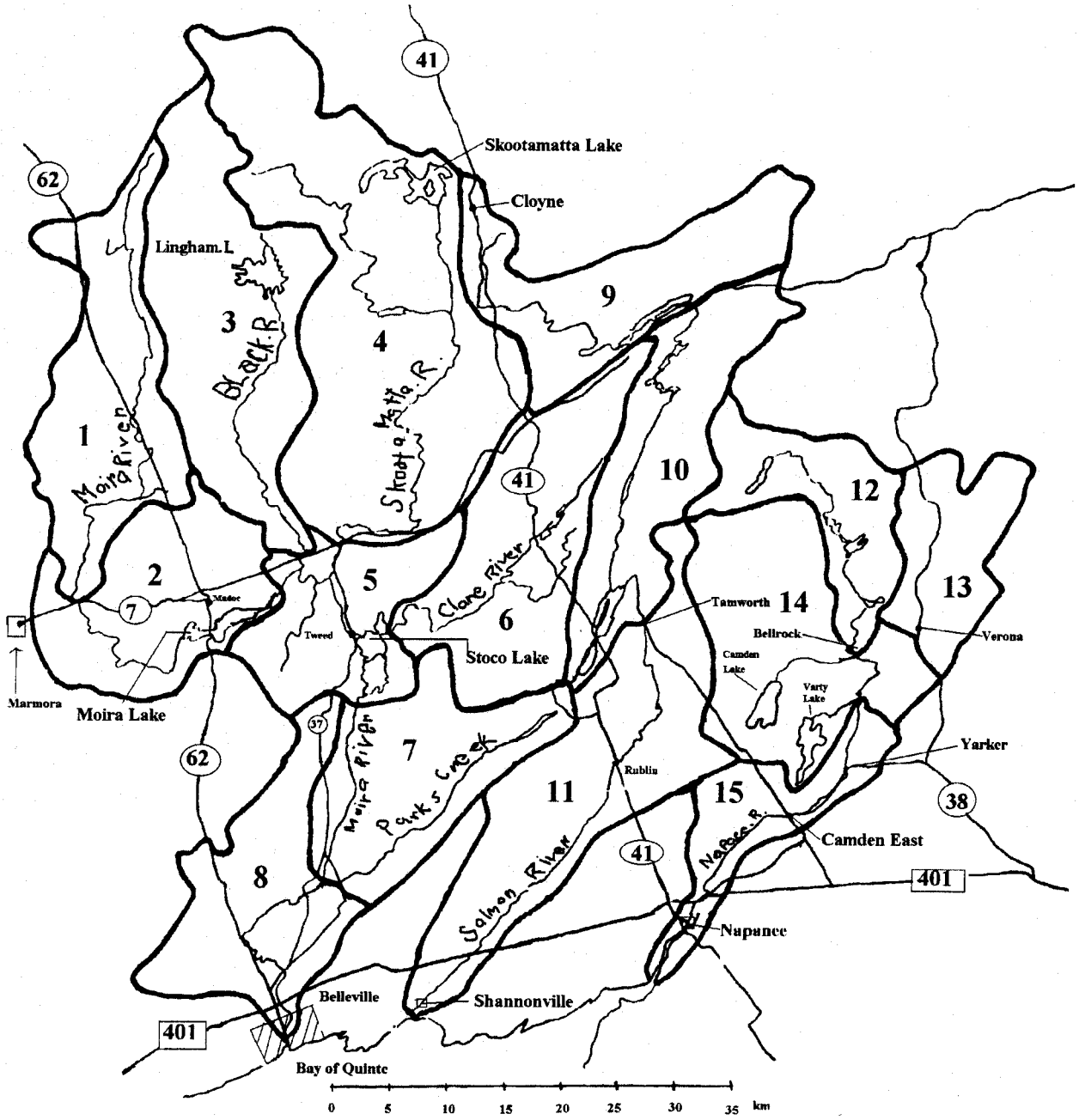


Figure 3.1a
Quinte Conservation
Moira, Salmon and Napanee Rivers

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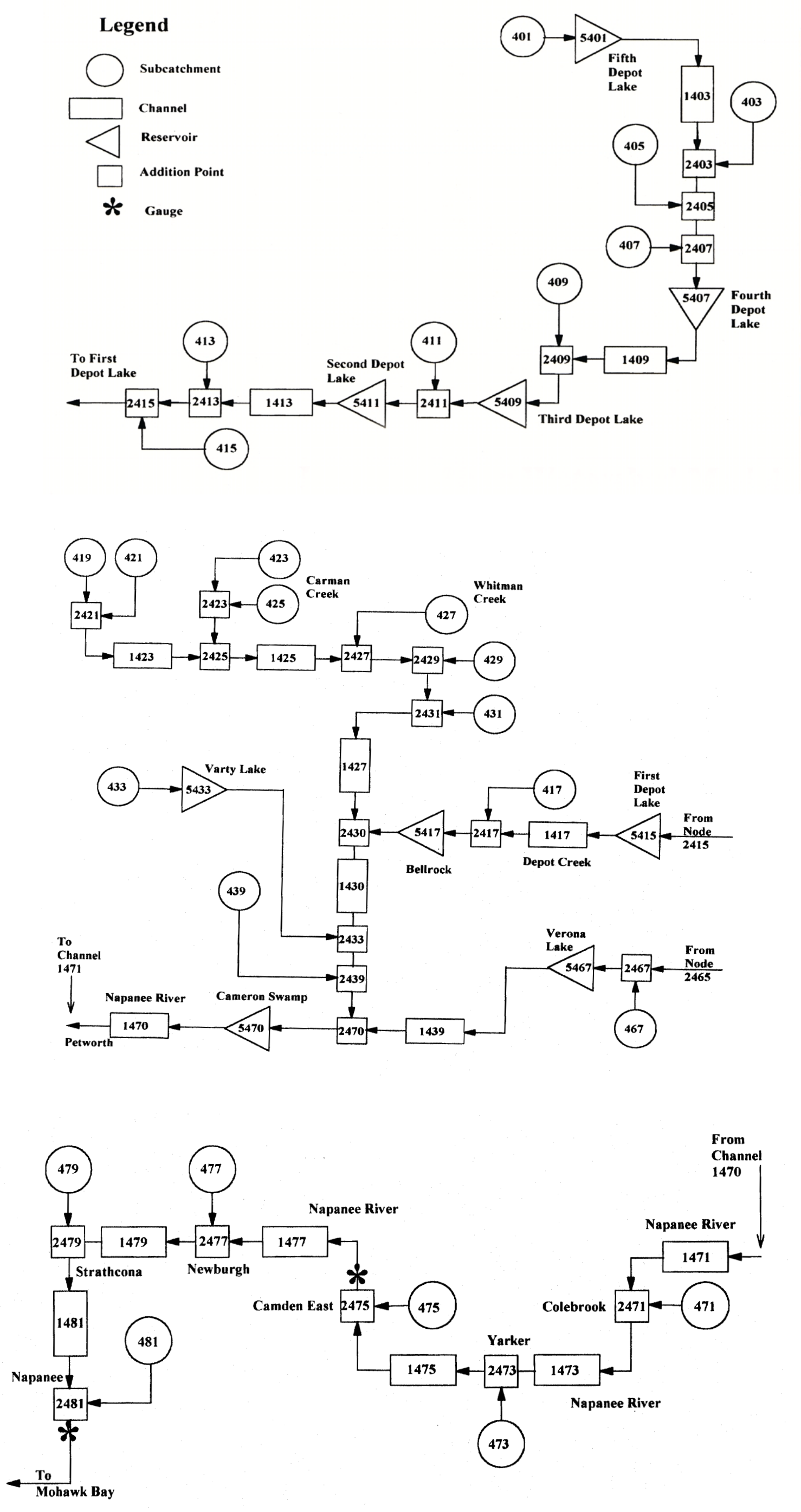


Figure 3.1b
Quinte Conservation
Schematic Representation of the Napanee River Hydrology Model

Source: Moira River Integrated Flod Forecast System/Final Technical Report/2000

Table 3.1

Bellrock Dam
Rating Curve

						Mill Dam	
Sill elevation =	139.98 m	Weir crest =	140.95 m			Stop logs crest =	140.78 m
Openning width =	3 m	Length =	8.08 m			Width =	1.53 m
Log height =	0.2 m	Retaining wall crest	141.3 m			Wall crest =	141.5 m
No. of sluices =	1	Wall length =	7.26 m	(2.66+4.6 m		Length =	27.24 m
No. of stop logs =	7	Top of piir =	141.46 m				
		Width =	2.66 m	(1.7+.96) m			

Water Elevation (m)	Discharge (m^3/s) No. of Stop Logs in Sluice								Bellrock Dam						
	0	1	2	3	4	5	6	7	Weir	Wall	Pier	Mill Logs	Mill Wall	Total (m^3/s)	Storage (ha-m)
139.98	0.00													0.00	0.00
140.18	0.43	0.00												0.00	0.61
140.38	1.21	0.43	0.00											0.00	1.22
140.58	2.23	1.21	0.43	0.00										0.00	1.83
140.78	3.43	2.23	1.21	0.43	0.00				0.00			0.00		0.00	2.44
140.98	4.80	3.43	2.23	1.21	0.43	0.00			0.06			0.22		0.28	3.05
141.18	6.31	4.80	3.43	2.23	1.21	0.43	0.00		1.34	0.00		0.62		1.96	3.66
141.38	7.95	6.31	4.80	3.43	2.23	1.21	0.43	0.00	3.42	0.25		1.14	0.00	4.80	4.27
141.58	9.71	7.95	6.31	4.80	3.43	2.23	1.21	0.43	6.06	1.61	0.00	1.75	0.92	10.78	4.88
141.78	11.59	9.71	7.95	6.31	4.80	3.43	2.23	1.21	9.16	3.62	0.72	2.45	6.05	23.22	5.49
141.98	13.58	11.59	9.71	7.95	6.31	4.80	3.43	2.23	12.67	6.11	1.50	3.22	13.59	39.31	6.1
142.18	15.66	13.58	11.59	9.71	7.95	6.31	4.80	3.43	16.53	8.99	2.44	4.06	22.91	58.36	6.71
142.38	17.85	15.66	13.58	11.59	9.71	7.95	6.31	4.80	20.73	12.22	3.52	4.95	33.73	79.95	7.32
142.58	20.12	17.85	15.66	13.58	11.59	9.71	7.95	6.31	25.22	15.77	4.73	5.91	45.86	103.80	7.93
142.78	22.49	20.12	17.85	15.66	13.58	11.59	9.71	7.95	30.00	19.61	6.05	6.92	59.17	129.71	8.54
142.98	24.94	22.49	20.12	17.85	15.66	13.58	11.59	9.71	35.05	23.71	7.48	7.99	73.57	157.52	9.15
143.18	27.48	24.94	22.49	20.12	17.85	15.66	13.58	11.59	40.36	28.07	9.00	9.10	88.97	187.10	9.76
143.38	30.09	27.48	24.94	22.49	20.12	17.85	15.66	13.58	45.91	32.67	10.62	10.26	105.33	218.36	10.37
143.58	32.79	30.09	27.48	24.94	22.49	20.12	17.85	15.66	51.69	37.49	12.32	11.47	122.57	251.21	10.98
143.78	35.56	32.79	30.09	27.48	24.94	22.49	20.12	17.85	57.70	42.53	14.10	12.72	140.67	285.57	11.59
143.98	38.40	35.56	32.79	30.09	27.48	24.94	22.49	20.12	63.92	47.78	15.96	14.01	159.58	321.38	12.2
144.18	41.32	38.40	35.56	32.79	30.09	27.48	24.94	22.49	70.36	53.23	17.90	15.35	179.27	358.58	12.81
145.18	56.92	53.67	50.48	47.36	44.30	41.32	38.40	35.56	105.44	83.23	28.63	22.59	288.45	563.90	15.86

Table 3.2

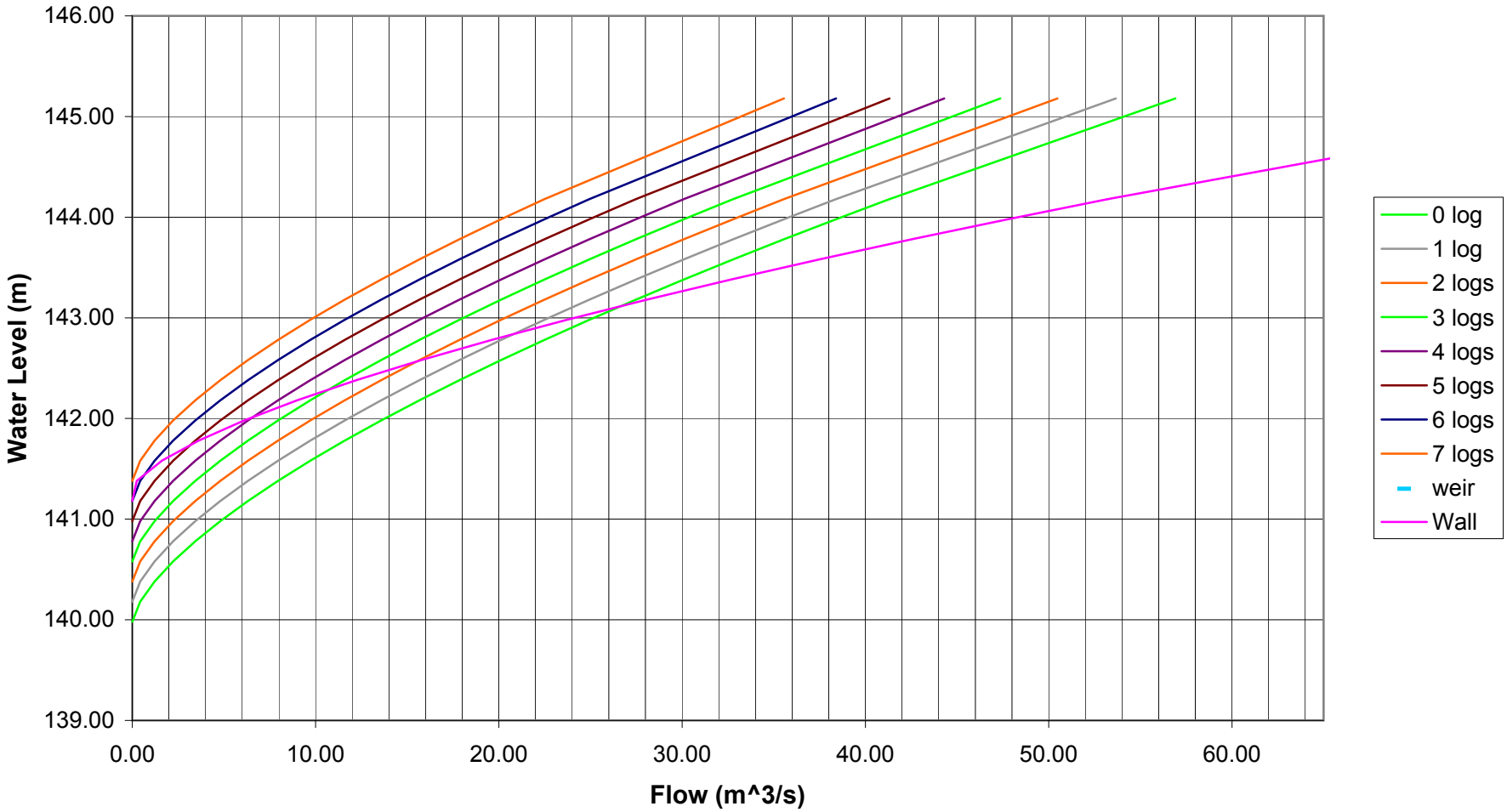
Bellrock Dam
GAWSER Simulation Results

Event Duration	Event Timing	Start Water Elevation	Return Period (yr)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Peak Water Level (m)
3 Days	Spring Snowmelt Plus Rain CN III	Typical April	2	15.5	15.5	141.65
			5	21.1	21.1	141.70
			10	24.4	24.4	141.82
			20	27.6	27.6	141.90
			50	32.2	32.2	141.93
			100	35.2	35.2	141.95
	Summer Storm CN II	Normal Water Level	2	24.2	24.1	141.81
			5	30.5	30.5	141.91
			10	34.7	34.7	141.94
			20	38.6	38.6	141.97
			50	43.4	43.4	142.10
			100	48.5	48.5	142.15
3 Days	Summer	NWL	PMF	515.0	515.0	144.20

Notes:

Spring starting water level = 140.98 m.
Summer starting water level = 140.98 m.

Figure 3.2: Bellrock Dam Rating Curve



along with peak water level at the damsite as the flood is routed through the outlet structure. The starting water level for runoff events is set at 140.98 m, and the dam is acting as an overflow weir.

As shown in Table 3.2, the peak water level for the 1:100-yr event is approximately 142.15 m with a corresponding outflow of 48.5 m^3/s. Table 3.2 indicates that the reservoir storage of this dam is rather small, so there is little peak flow reduction for a flood event higher than 1:2-yr event.

Bibliography

Bibliography

Ministry of Natural Resources. **Flood Plain Management in Ontario, Technical Guidelines**. 1986.

Ministry of Natural Resources. **Ontario Dam Safety Guidelines** (Draft). 1999.

Schroeter, H. O. and H. R. Whiteley. **GAWSER-GUELPH Agricultural Watershed Storm-Event Runoff Model, User's Manual (Version 5.1), Technical Report, School of Engineering, University of Guelph**. 1987.

Schroeter and Associates. **Moira River Integrated Flood Forecast System (MRIFFS), Final Technical Report**. 2000.

Part IV
Dam Assessment

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Part IV – Dam Assessment

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1 Introduction

1 Introduction

An inspection of the Bellrock Dam for the Quinte Conservation (QC) was performed on November 5, 2003. The results of the qualitative inspection are presented in Part I of this report. A summary of the major characteristics of the dam is presented in Table 1.1.

Table 1.1

Description of Bellrock Dam

Name of Dam	Type of Dam	Year Built	Foundation Type	Description			
				Reservoir Area (km^2)	Height (m)	Length (m)	No. of Sluices
Bellrock	Concrete gravity with integral spillway and overflow sections	Unknown; recent repairs in mid-1970s	Good quality, very strong, crystalline granite	0.25	2.5 average	21 bank to bank	One stop log bay 3 m wide with seven installed stop logs

This Part IV of the report presents the results of the stability analyses that were performed using the parameters and the general methods described herein. In performing these analyses, maps and photographs produced during the site inspection, as well as site-specific geologic data, were used to assist in the assessment of the dam structures. The results of the stability analyses were used to determine if the Bellrock Dam satisfies current draft Ontario Dam Safety Criteria, according to the criteria provided in the draft Ontario Dam Safety Guidelines (ODSG). The results from these analyses form the basis of the recommendations for remedial work.

1.1 Dam General Description

The dam, looking downstream from the right bank (west) to the left bank (east), consists of the following components:

- right wingwall, consisting of a concrete retaining wall 7.3 m long, 2.5 m high (maximum) and 2.1 m thick

- stop log opening 3 m wide, 2.5 m high, equipped with seven logs
- concrete overflow weir 8 m long, 1 m high (average), 1 m thick.

The mill dam is a sluiceway that exists about 50 m to the west along the reservoir edge. The sluice formerly provided water to the adjacent mill that is no longer in operation.

The dam is clearly founded on bedrock, for there is very little overburden, and abundant smooth bedrock outcroppings exist on the shoulder of the dam.

2 Stability Assessment

2 Stability Assessment

Stability assessments were undertaken for the QC Bellrock Dam. This section describes these assessments.

2.1 Methods of Analysis

The dam safety analyses involved the assessment of the ability of the structure to resist

- sliding at the dam-foundation interface, within the dam and at any plane in the foundation under all loading conditions
- overturning
- overstressing of the concrete dam or foundation.

This analysis is based on 'rigid body' limit equilibrium method with the various load combinations treated as static because of the relatively sustained nature of loads involved.

For critical representative sections of the structures, sliding stability in the upstream-downstream direction, the compressive and bearing stresses in the concrete and the location of the resultant were determined. Where the location, magnitude, direction and duration of computed tensile stresses were such that the stresses would be likely to produce cracking, the extent of cracking was evaluated.

Seismic analyses are typically performed at different levels of sophistication depending on the hazard potential rating of the dam and the probability of unacceptable performance. For the Bellrock Dam, pseudostatic methods of analysis were used.

2.2 Performance Indicators

The assessment of the suitability of the concrete structures was based on the following performance indicators:

- position of resultant force
- normal stresses at the heel and the toe

- calculated sliding factors and strength factors
- overturning factors.

Position of Resultant Force

The draft ODSG indicates that the position of the resultant should be within the middle third of the base for normal loading conditions and within the base for other load cases. Therefore, the intent of the guidelines is that this is a desirable, but not mandatory, requirement for the evaluation of concrete dams. On this basis, dams that satisfy the following conditions:

- existing structure with a history of service and no signs of significant distress
- low incremental consequence category
- sliding stability criteria
- compressive strength criteria

were considered to meet the intent of the dam safety requirements even if the position of the resultant was outside the middle third of the base for the normal case.

Tensile Stresses

Within the dam, tensile stresses are acceptable so long as the stresses remain within 0.1 f_c' to 0.05 f_c' (where f_c' is the compressive strength of concrete) within the mass concrete and at lift joints, respectively.

Sliding Factor

The resistance of a gravity dam against sliding on any surface is designed or assessed by comparing the net driving force with its available shear strength. The ratio of these components is the factor of safety (FOS) against sliding or sliding factor (SF).

$$SF = \frac{\text{Available Shear Strength}}{\text{Net Driving Force}}$$

The draft ODSG recognizes two states of available shear strength: 'peak' and 'residual'.

(a) Peak shear strength is based on the following components:

$$\text{Available Peak Shear Strength} = \sum A_c \{\sigma_n \tan(\phi'') + \tau_o\}$$

where,

σ_n = normal stress
N' = peak angle of internal friction
A_c = area of compression
ϑ_o = the available peak shear strength at zero normal stress.

(b) The residual or post-peak strength is defined as

$$\text{Available Residual Shear Strength} = \sum A_c \{\sigma_n \tan(\phi') + \tau_n\}$$

where,

N = residual angle of sliding friction
ϑ_n = nominal residual shear strength value at zero normal stress. According to the Ministry of Natural Resources guidelines, this value may range up to 100 kPa (15 psi), if supported by tests*. Without tests, it is assumed to be zero. For this study, the residual value was assumed to be zero for all structures since no test data was available.

2.3 Selection of Loads

The following loads were considered in the assessment of the Bellrock Dam concrete structure:

- dead loads of permanent structures, rock or soil backfill, silt deposited against the structure and any significant equipment loads

---

* As discussed, it is not strictly correct to assume a cohesive strength when considering residual shear strength as concrete-to-bedrock bonds are broken at very small strains. For the so-called 'residual' shear resistance, it is better to consider all bonding to be lost and any apparent cohesion to be a function of roughness.

- maximum flood headwater level based on the inflow design flood (IDF) with corresponding tailwater levels
- internal water pressure and foundation uplift
- static thrust created by an ice sheet
- maximum design earthquake (MDE).

Ice Loads

The thermally-driven, static, ice loads used in the design review were assessed by taking into consideration site-specific characteristics and operating information.

For ice loadings, it was assumed that horizontal thrust created by thermal expansion of ice sheets would occur 0.3 m below the headpond level. Research by Ontario Power Generation (OPG), Manitoba Hydro, Fleet Technology and others has shown that the magnitude of this ice thrust depends on factors such as the thickness of the sheet of ice, the average ambient temperature, the rate of temperature change in the ice, fluctuations in the water surface, reservoir characteristics and wind drag.

The temperature data required as part of the ice load assessment was established by considering the January 1% temperature from the Ontario Building Code. For the Bellrock Dam, the January 1% temperature was found to be -24°C.

Headpond shoreline characteristics, such as slope, were measured from current 1:20 000 Ontario Base Maps and the topographic details established during the site investigations. On the basis of procedures for estimating ice loads presented by OPG at a workshop on ice held at the annual Canadian Dam Association conference in 2000 as detailed in Table 2.1, the resulting ice thrust values were estimated. The results of this assessment showed that the following ice loads should be considered at the Bellrock Dam:

- ice load on concrete 80.2 kN/m
- ice load on stop logs 29.2 kN/m.

Table 2.1

Thermal Ice Loads on Concrete Dams

Reservoir Shoreline Characteristics	Winter Air Temperature (January 1% Temperature* from OBC)		
	Mild 0° to -20°C	Average -21° to -29°C	Severe -30°C & Lower
Flat Shore (<20° slope)	58.4 kN/m (4 kips/ft)	80.2 kN/m (5.5 kips/ft)	102.1 kN/m (7 kips/ft)
Steeper Shore (20° to 45° slope)	73.0 kN/m (5 kips/ft)	87.5 kN/m (6 kips/ft)	116.7 kN/m (8 kips/ft)
Steep Rocky Shore (>45° slope)	87.5 kN/m (6 kips/ft**)	116.7 kN/m (8 kips/ft**)	145.9 kN/m (10 kips/ft**)

Notes:

1. * The January 1% temperature is defined as the lowest temperature at or below which only 1% of the hourly exterior air temperatures in January occur. The January 1% temperatures for selected locations in Ontario are tabulated in the Ontario Building Code (OBC).

2. ** For steep rocky shoreline, careful study of the site-specific condition with regard to the shape of the headpond, snow cover data and temperature records is required to determine the design ice load magnitude, as the ice load can be larger than the values shown in the table.

3. Ice load for steel gates = 50% of the values shown in the table.

4. Ice load for timber logs = 29.2 kN/m (2.0 kips/ft).

5. Ice load reduction where timber crib remains exist at or above the waterline shall be based on the location, top elevation, and flexibility of the subject timber crib structure.

6. Minimum ice load where ice sheet existed against the structure = 29.2 kN/m (2.0 kips/ft).

7. Maximum water level in January from past records (from 30 to 80 years) shall be considered for the 'winter operating condition' in the design review. However, this water level may not be much different from the maximum headwater level given for the summer condition.

8. Site-specific conditions based on the design review inspection shall be used in selecting the appropriate design ice load.

Hydrostatic Uplift

Hydrostatic pressures within the dam and foundation are considered as follows.

- Case 1: For dams with no foundation drains or pressure relief systems, full uplift, varying linearly from 100% headwater pressure at the upstream face to 100% tailwater pressure at the downstream face, is assumed to act on the entire base area of the dam.

- Case 2: For dams equipped with an effective drainage and/or pressure relief system (based on field investigations and/or monitoring data), reduced uplift is used. The reduced uplift varies from 67% of upstream headwater pressure to 100% tailwater pressure, only if the actual recorded uplift is less.

At the Bellrock Dam, Case 1 applies.

The uplift assumption corresponds to the design water levels and does not consider any 'locked in' pressures. If base tensions exceed allowable limits (typically assumed to be one half of the threshold shear strength), it is assumed that tension cracking of the base occurs at the level, which allows full uplift pressures to be transmitted along the crack for cases not involving earthquake loadings. In the case of earthquakes, it is assumed that the motions are of such a short duration that uplift pressures will not be increased within any crack that may be theoretically induced from the earthquake loadings.

Seismic Loads

Probabilistic earthquake parameters for the damsite was established based on data obtained from the Geological Society of Canada, as summarized in Table 2.2.

These seismic loads were considered to act in a horizontal direction (increasing the driving force) and a vertical upwards direction (decreasing the horizontal resisting force). In the pseudostatic method of analysis, two thirds of the peak ground acceleration is used to simulate the sustained ground motion in combination with two thirds of that value acting in a vertically upward direction.

Table 2.2

Probabilistic Earthquake Parameters

Probability of Exceedance per Year	**0.010**	**0.005**	**0.0021**	**0.001**
Return Period (years)	1:100	1:200	1:475	1:1000
Peak Horizontal Ground Acceleration (*g*), (PGA)	0.043	0.062	0.092	0.123

The draft ODSG requires that dams

> "*. . . be designed and evaluated to withstand ground motions associated with a Maximum Design Earthquake (MDE), without release of the reservoir*"

with the selection of the MDE for a dam being based on the hazard potential classification and consequences of dam failure. As shown in Table 2.3, for any given site, the MDE increases with increasing hazard potential due to dam failure.

Table 2.3

Criteria For Design Earthquakes

Hazard Potential Classification [a]	MDE	
	Deterministically Derived	Probabilistically Derived (Annual Exceedance Probability)
High	50% to 100% MCE [b] [c] [d]	1:1000 to 1:10 000 [d]
Significant	– [e]	1:100 to 1:1000 [e]

(a) Hazard potential classification established separately for each dam.

(b) For a recognized fault or geographically defined tectonic province, the maximum credible earthquake (MCE) is the largest reasonably conceivable earthquake that appears possible. For a damsite, MCE ground motions are the most severe ground motions capable of being produced at the site under the presently known or interpreted tectonic framework. Use upper values in the range, where loss of life and property damage due to failure would be unacceptably high.

(c) An appropriate level of conservatism shall be applied to the factor of safety calculated from these loads, to reduce the risks of dam failure to tolerable values. Thus, the probability of dam failure could be much lower than the probability of extreme event loading.

(d) In the high hazard potential category, the MDE is based on the consequences of failure. Design earthquake approaching MCE would be required where loss of life and property damage due to failure would be unacceptably high.

(e) If a structure in the significant hazard potential category cannot withstand the minimum criteria, the level of upgrading may be determined by economic risk analysis, with consideration of environmental and social impacts.

Due to the VERY LOW incremental hazard potential (IHP) classification of the Bellrock Dam, the structures were not analyzed for earthquake loading conditions.

Hydrostatic Loads

Water levels used in the assessment of the various load cases were derived for the various load cases based on the IHP classification of the dam. The IDF was determined to be the 1:50-yr flood. These levels were determined to be as follows:

- normal summer/winter headwater level = 140.98 m
  tailwater level = 139.00 m
- flood headwater level = 142.10 m
  tailwater level = 139.97 m.

2.4 Selection of Shear Strength Parameters

The sliding stability of a concrete dam is a complex phenomenon. It is affected by the intact strength of the foundation material, the strength of the concrete and by planes of weakness within the foundation (rock joints), within the structure itself (lift joints) or at the interface between the structure and the foundation. When the shear resistance is governed by a definable sliding plane, the orientation, the continuity and the properties of the discontinuity affect the available shear resistance. In addition, during the shearing process, intact pieces bounded by cross-joints (or in the case of concrete, construction joints) may translate, rotate or crush in response to the stresses imposed, altering the shear resistance properties along the plane.

2.4.1 Sliding Stability of Concrete Dams - General Concepts

For a well-constructed concrete structure, situated on a well-prepared bedrock foundation, it is usually an appropriate assumption to model concrete and rock as a continuum that does not contain major defects. However, where the concrete, or the foundation, contains discontinuities or fractures, it is often necessary to adjust the shear strength relationship to account for the presence of these imperfections. This typically results in a curvi-linear normal stress-shear strength relationship. For example, as early as 1961, Grishin and Evdokimov noted that, in the initial stages of deformation, the sliding resistance of a concrete dam was indeed a combination of friction and the strength of the concrete-to-rock bond. However, they observed that when the concrete-to-rock bond was broken, a second-stage sliding (cohesion = 0) occurred. During this phase of sliding, the authors suggested that the failure mechanism was quite different.

> *" . . . there may exist a fictitious bond 'c' as a result of bond (interlocking) between concrete and rock projections which has to be removed (sheared). In this case, it is desirable to replace the frictional component in the Mohr-Coulomb equation with a*

> *parameter similar to the coefficient friction but closely related to the complex phenomenon of shear strength of two bodies with uneven surfaces caused by the destruction of bond between them."*

Deere (1976) also suggested that there might be problems associated with the use of the traditional Mohr-Coulomb equation, noting that the high factors of safety (i.e., in the order of 4) often used in the industry were a result of uncertainty with the shear strength parameters. In fact, previously, Brace and Byerlee (1967) had suggested that the coefficient of friction, for a given geological situation, could not be predicted within a factor of 2. Barton (1976) also expressed concern over the use of the traditional shear strength parameters, noting that, for conditions of low normal effective stress, the use of a cohesive intercept was

> *". . . inherently dangerous, even if the extrapolation is made from the mean effective actual stress level appropriate to the particular engineering problem."*

Despite recognition of this fact, since the early 1960s, the simplified linear Mohr-Coulomb criterion has been used in the assessment of dam sliding stability, almost unchanged, for many years. The problem with the use of the traditional linear Mohr-Coulomb strength parameters is that, in many real problems, it is often necessary to determine shear resistance for a wide range of normal stress conditions. Under such conditions, for most materials, the Coulomb concept of cohesion (c) and friction (ϕ) is really no more than a simple mathematical convenience since neither cohesion nor friction is a constant, due to the curvi-linear nature of the actual shear resistance envelope.

In addition, there is also often confusion regarding the selection of appropriate parameters. For a fully bonded discontinuity, shear strength properties are a function of the concrete alone. However, with small strains, the bond fractures. Following fracture, the shear resistance becomes a function of frictional resistance of the broken surface and dilation (roughness). For this reason, it is not appropriate to assign a cohesive strength to an unbonded surface or as a 'residual' cohesive strength since the strain required to reach the residual state would destroy any available cohesion. It, therefore, follows that it is not correct to combine the cohesion, offered by a bonded or partially bonded rough discontinuity, with the dilation component of shear strength since deformation is required to mobilize dilation.

In the case of a partially bonded surface, the total shear strength can be assessed on the basis of combining the cohesive strength of the bonded portion with the basic shear resistance (i.e., the resistance with no dilational component) of the unbonded portion on a prorata basis.

2.4.2 Shear Strength of Bonded Discontinuities

The shear strength of a bonded concrete lift joint, as defined by EPRI (1992), is defined in terms of the concrete properties only.

$$\tau_c = c + \sigma_n \tan(\phi_c)$$

where,

τ_c = shear strength of concrete
c = cohesion (or threshold shear strength) of concrete
σ_n = normal stress
ϕ_c = angle of internal friction for concrete.

The results of testing performed on 223 bonded lift joints are shown in Figure 2.1. Best fit, lower bound and absolute lower bound shear strength values* established for bonded discontinuities on the basis of these tests are summarized in Table 2.4.

Table 2.4

Summary of Results of EPRI Testing

Parameter	Best Fit	Lower Bound	Absolute Lower Bound
C (MPa)	2.10	0.97	0.83
ϕ_i (deg)	57	57	57
F_t (MPa)	1.40	0.62	0.52

* The best-fit line was determined by linear regression, and the lower bound lines were drawn by hand to include all data with a few noted exceptions. The absolute lower bound represents the lower bound to all data.

Assuming a typical concrete compressive strength (f_c'), this data suggests that the lower bound cohesive strength varies from 0.04 f_c' to 0.06 f_c', with the lower cohesive strength derived from tests performed on samples in which alkali-silica reaction effects may have reduced strength. In a Canadian Electricity Association research report prepared by Acres International Limited (Acres), a reasonable estimate of the cohesive strength, in the absence of test data, was considered to be $c = 0.05\ f_c'$ and $\phi_c = 57°$ for bonded concrete lift joints or a bonded concrete-to-rock interface. These values are consistent with the recommendations of the draft ODSG. When such values of cohesion are applied to small dams, very factors of safety are usually calculated. Unfortunately, cohesion can be lost as a result of even a very small amount of movement. In addition, it is difficult and often expensive to prove that a bond exists and the extent of bonding that exists. Therefore, for assessing the safety of the dam, it was assumed that the structure was unbonded.

2.4.3 Shear Strength of a Rough Unbonded Discontinuity

The shear resistance of intact material consists of a true cohesion (i.e., the material has a measurable tensile strength) and a component that varies with normal stress, often referred to as friction. The true definition of cohesion is beyond the scope of this report. However, it is a function of the intact characteristics of the material and is completely lost when the material fractures (Papaliangas, Personal Communication, 1998). As discussed previously, following fracture, shear resistance becomes a function of friction and dilation only. Therefore, it is not appropriate to assign a cohesive strength to an unbonded surface.

For unbonded discontinuities, either within the concrete structure, at the concrete-to-rock interface or in the rock itself, the shear resistance is a function of basic (nondilational) friction (ϕ_b), which is a material property unaffected by normal stress, and a dilation friction component (R). This latter component is a function of the nature of the roughness along the sliding plane, the strength of the material, the normal stress and the length of the sliding surface.

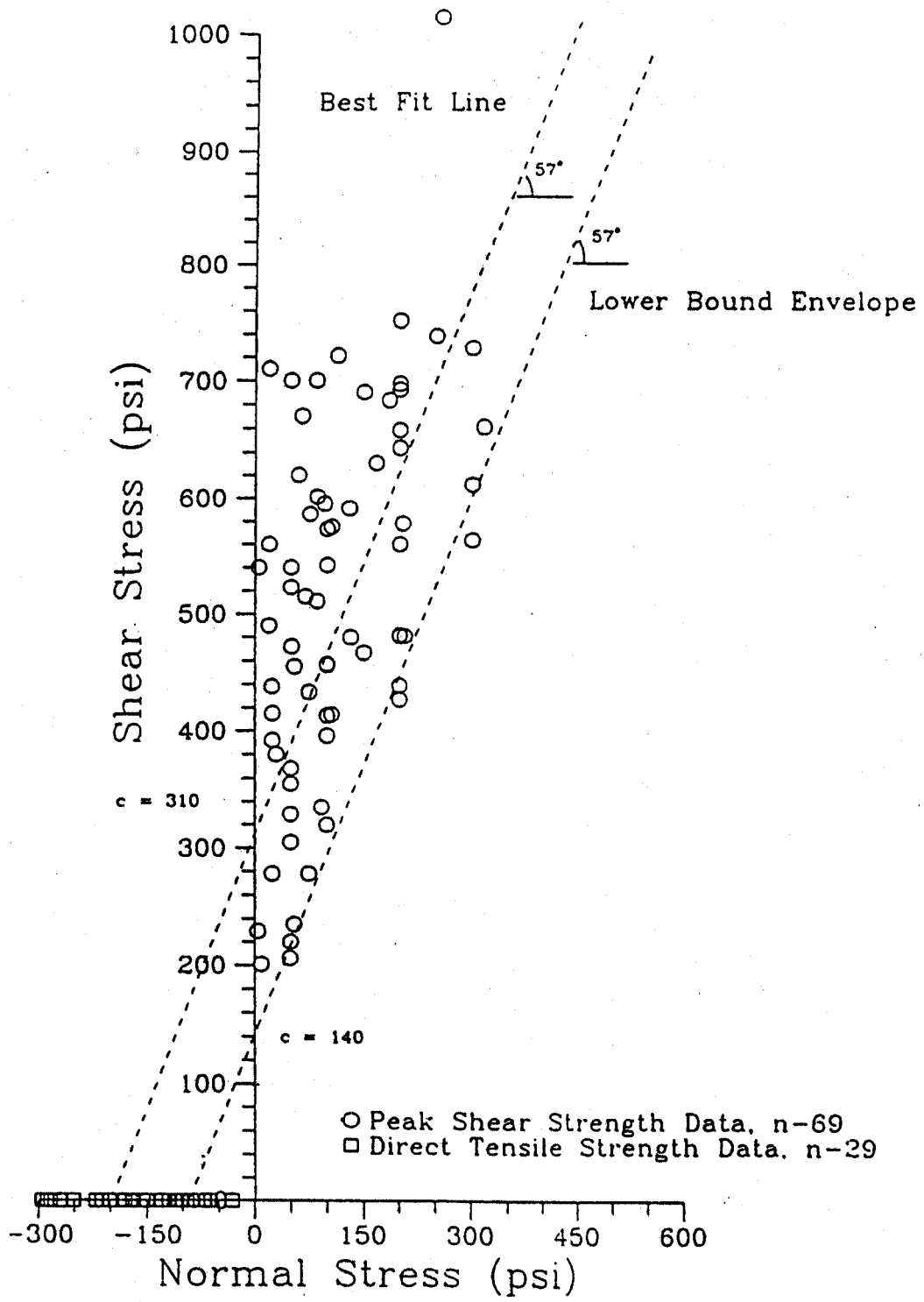

Figure 2.1
Quinte Conservation
Dam Assessment

Peak Shear Strength of Concrete-Lift Joints

ACRES International

$$\tau = \sigma_n \tan(\phi_b + R)$$

where,

R = a measure of the roughness of the interface.

The selection of the roughness parameter is entirely controlled by the nature of the contact, which must be established on the basis of assessments by an experienced engineering geologist. There is, however, some general lower bound estimates that can be made when evaluating a concrete structure built on a good quality bedrock foundation. For example, Lo et al (1991) stated that the basic friction angle of the bedrock-to-concrete contact was not sensitive to rock type* and that ϕ_b ranged from 30° to 39°. As part of a large research project, EPRI (1992) performed numerous residual shear tests strength "taken as the lowest consistent set of five or more readings at a displacement of 2.5 to 13 mm" and concluded that the shear resistance of the unbonded bedrock-to-concrete contact was in the range of 34° to 39° (excluding shale). However, these results also indicated an 'apparent' cohesion that, in reality, would be related to a foundation roughness that must be assessed on a case-by-case basis.

Both of these results are quite similar, indicating that the concrete-to-bedrock contact has a basic shear resistance in the range of 30° to 39°, likely depending on the quality of the concrete at the contact. There are, however, tests that have indicated somewhat lower ranges of values. As there remains some discrepancy in the literature regarding base friction values, care is required when selecting an appropriate value.

The base friction values are then increased, to varying degrees, depending on the roughness (or dilational shear resistance) of the actual contact that is determined in the field by experienced engineering geologists.

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* Perhaps because the frictional resistance was more related to the nature of the generally weakened concrete.

2.4.4 Generalized Shear Resistance Parameters

The generalized shear strength criteria can, therefore, be summarized as follows.

For Bonded Discontinuities

$\tau = c + \sigma_n \tan \phi_i$

where,

c = true cohesion

ϕ_i = the angle of internal friction for the intact material.

For Unbonded Discontinuities

$\tau = \sigma_n \tan (R + \phi_b + i)$

where,

ϕ_b = basic friction angle

R = dilational component of shear resistance

i = large-scale waviness of the joint.

For Partially Bonded Discontinuities

$\tau = A_b (c + \sigma_n \tan \phi_i) + (1 - A_b) (\sigma_n \tan \phi_b)$

where,

A_b = ratio of bonded area to total area.

2.4.5 Recommended Shear Resistance Parameters

The draft ODSG recommends the following shear strength parameters for assessing the sliding stability of dams situated on a competent bedrock foundation.

(a) Peak Shear Strength Values

Mass Concrete

Threshold peak shear strength (τ_o)	=	$0.17\sqrt{f_c'}$ (MPa)
Peak angle of shearing resistance (ϕ_c')	=	45°

Good Quality Lift Joints or a Bonded Concrete-to-Bedrock Contact

Threshold peak shear strength (τ_o)	=	$0.085\sqrt{f_c'}$ (MPa)
Peak angle of shearing resistance (ϕ_a')	=	55°

These values correspond well with lower bound values measured by EPRI (1992) and others. However, it is important to note that the use of a cohesive component implies a true bond and, therefore, tensile capacity which can resist cracking. This can have a significant impact on the assessments.

(b) Residual Shear Strength Values

'Residual' shear strength is, in fact, somewhat of a misnomer. In terms of the draft ODSG, it refers to the shear strength of an unbonded concrete-to-bedrock contact, or a lift joint in which enough movement has occurred in order to fracture the concrete bond. In this case, the available shear strength is a function of the basic shear resistance of the discontinuity surface, and a roughness component that is often referred to as an apparent cohesion or threshold shear strength. The draft ODSG do not specifically address this issue but do recommend the use of frictional component (ϕ''_r) for the discontinuity surface coupled with a threshold shear strength component (τ_n) of up to 100 kPa (15 psi), if the value is supported by test results. Without testing, a value of zero is recommended. For the Bellrock Dam, a value of zero was assumed for all structures. As discussed previously, this is the most appropriate approach for the analysis of any unbonded discontinuity.

(c) Tensile Strength

The draft ODSG permits tensile stresses in the range of 0.1 f_c' and 0.05 f_c' within the mass concrete and at competent bonded lift joints, respectively.

(d) Compressive Stresses

The maximum allowable compressive strength at the toe of a dam founded on competent bedrock is specified in the guidelines as not to exceed 0.45 f_c'.

(e) Position of the Resultant

The position of the resultant indicates the potential for the development of tensile stresses (and cracking) within the dam. For the usual loading case, the resultant of all forces (loads) should be in the middle third of the surface being analyzed. This ensures that the entire base of the dam remains in compression. For other load cases, the resultant may be outside the middle third, provided the other performance indicators are found to be satisfactory.

2.4.6 Selected Shear Resistance Parameters

As described previously, the Bellrock Dam is founded on good quality, very strong, crystalline granite. The critical sliding surface is the rock/concrete interface.

Specific test data were not available to confirm whether or nor a cohesive bond existed at the bedrock-to-concrete contact. The field investigations required verifying the existence of such a bond is complex and can be very expensive. For this reason, such investigations did not form part of this assessment. While a bond may exist, such an assumption is inherently unconservative for the reasons discussed previously. For these reasons, it was considered prudent to assess the structures using only the residual shear strength parameters or, in other words, to assume that the bedrock-to-concrete contact is unbonded. As a check, results were also determined for various assumptions regarding the cohesive bond strength so that the impact of the assumption of residual strength could be evaluated.

(a) Basic Shear Resistance

Acres has performed site-specific testing at a number of damsites located across Ontario. The results of all of these tests were quite similar indicating a shear resistance of 38°. This fell near the upper

range of the published results from Lo and EPRI (as described previously). Acres has, however, measured significantly lower values for basic shear resistance. To provide a conservative assessment for the review of the stability at the Bellrock Dam, a lower bound value of 30° was selected for the basic shear resistance.

(b) Roughness

Roughness components were assessed based on the nature of the geological conditions observed during the site inspections. At the Bellrock Dam, a value of minimum value of 13° was established.

This roughness value, combined with the basic shear resistance, was used to assess the stability of the structure. In addition, a parametric analysis was performed assuming that the contact was unbonded and the overall shear resistance values could vary from 10° below to 10° above the selected value.

(c) Bonded Strength

As a check, the stability of the structures under a range of peak shear strength assumptions (i.e., bonded contact) was assessed. To establish this range, it was assumed that the strength of the intact bond could be estimated assuming

$c = 0.085 \sqrt{f_c'}$ where f_c' is the strength of the concrete in the bonded section (assumed to be 50% of the measured mass concrete strength)

$\phi_c = 45°$ the frictional strength of the concrete in the bonded section.

As it would be unrealistic to assume that the entire base of the structure were bonded, a parametric assessment was performed using the equation for partially bonded discontinuities to assess the impact of the percentage of bonded contact on sliding stability by performing a parametric analysis assuming that the percentage of bonded contact ranged from 10% to 50%.

(d) Summary of Selected Parameters

A summary of the parameters used in the parametric analysis of the Bellrock Dam is provided in Table 2.5.

Table 2.5

Parameters Used in the Assessment of the Bellrock Dam

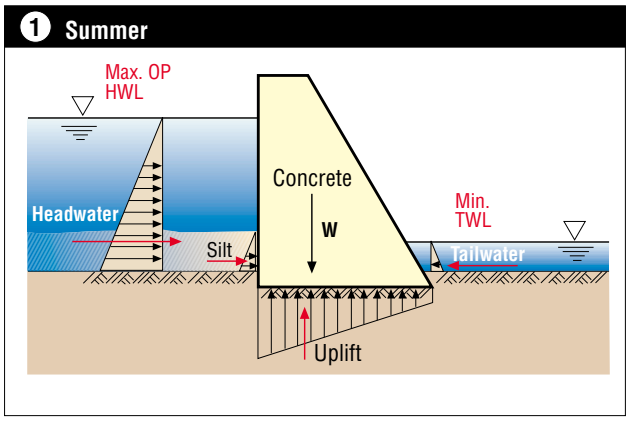
	Selected Value	Range	
		From	To
(a) Intact Concrete Properties			
Intact concrete compressive strength (f_c') (MPa)	20	-	-
Density of intact concrete (kN/m^3)	23.50	-	-
(b) Bonded Discontinuities			
Threshold shear strength (τ_o) (MPa)	0.085 $\sqrt{f_c'}$	10% bonded	50% bonded
Friction (ϕ'_c) (deg)	53	-	-
Allowable tensile stress (MPa)	0.025 f_c'	-	-
Maximum allowable compressive stress (MPa)	0.45 f_c'	-	-
(c) Unbonded Discontinuities			
Threshold shear strength (τ_n) (MPa)	0	-	-
Basic friction (ϕ'_r) (deg)	30	-	-
Roughness (I) (deg)	13	-	-
Allowable tensile stress (MPa)	0	-	-
Maximum allowable compressive stress (MPa)	0.45 f_c'	-	-

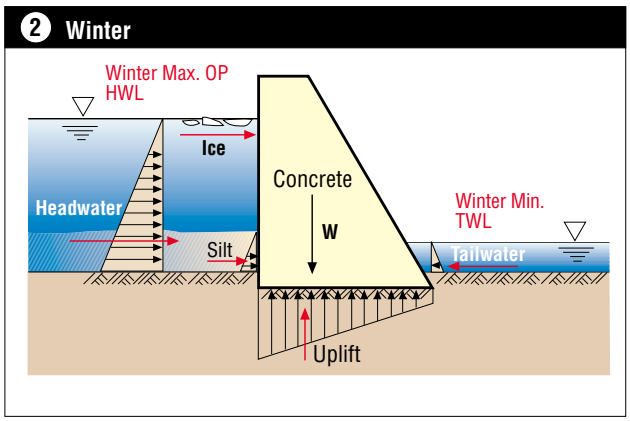
2.5 Load Combinations

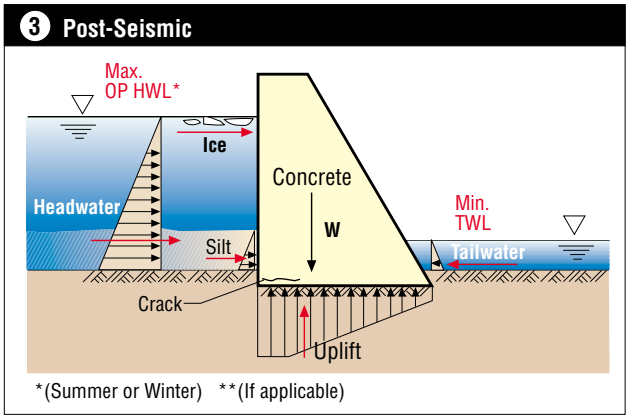
The various loading combinations are shown schematically in Figure 2.2 and are described as follows. Circled numbers refer to the numbers in Figure 2.2.

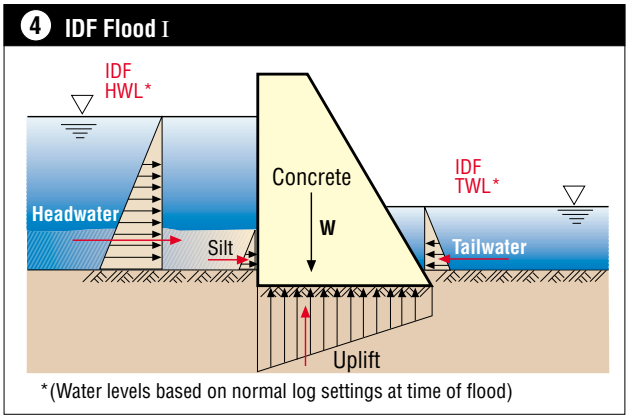
Usual Loading $\in$ and $\notin$

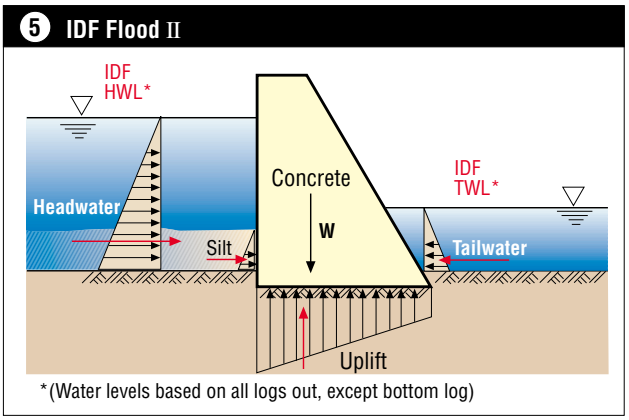
Permanent and operating loads were considered for both summer and winter conditions including self-weight, ice, silt, earth pressure, and the normal maximum operating water level with appropriate uplift pressures and tailwater level.

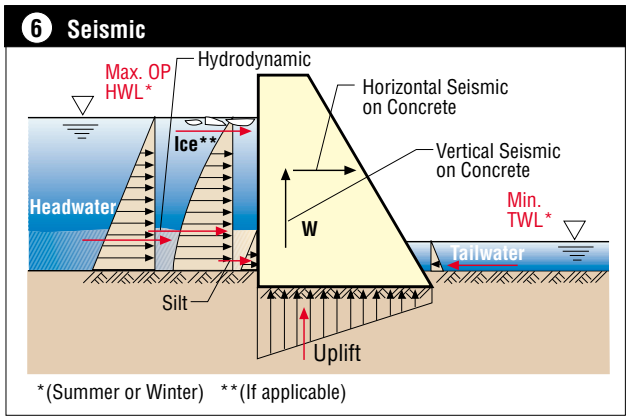












Legend

OP Operating

HWL Head Water Level

TWL Tail Water Level

IDF Inflow Design Flood

Figure 2.2
Quinte Conservation
Dam Assessment
Schematic of Load Cases

ACRES International

Unusual Loading ∠
Where earthquake-induced cracking at the rock concrete interface or any weak section was identified, a stability analysis was carried out to determine the stability of the structure, in its post-earthquake condition, under the effects of the usual loading conditions that could include concurrent ice loadings in areas where appropriate. Full reservoir pressure within the earthquake-induced cracks is assumed for the post-earthquake case.

Flood Loading I ∇
Permanent and operating loads of the usual loading case, except for ice loading, were considered in conjunction with reservoir and tailwater levels and uplift resulting during the passage of the IDF with the normal complement of logs in place. The effect of ice loads was not considered simultaneously with design flood conditions in accordance with the requirements of the draft ODSG.

Flood Loading II ®
For the Bellrock Dam, this case is identical to Flood Loading I since the structure is acting as an overflow.

Seismic Loading ©
Permanent and operating loads from the usual loading were considered in conjunction with the seismic loads that would be generated during the MDE. During this extreme load case, ice loads are also considered. Uplift pressures were assumed to be those corresponding to the normal loadings, and were not modified during the seismic event.

2.6 Acceptance Criteria

Acceptance criteria used in the analysis of Bellrock Dam concrete structures are as listed in Table 2.6.

Table 2.6

Acceptable Sliding Factors for Gravity Dams

Type of Analysis [a] [f]	Load Case			
	Usual	Unusual (Post-Earthquake)	Earthquake (MDE) [b]	Flood (IDF)
Peak Sliding Factor (PSF) – No Tests	3.0	2.0	1.3	2.0
Peak Sliding Factor (PSF) – With Tests [c]	2.0	1.5	1.1	1.5
Residual Sliding Factor (RSF) [d] [e]	1.5	1.1	1.0	1.3
Concrete Strength Factor [g]	3.0	1.5	1.1	2.0

(a) PSF is based on the peak shear strength. RSF is based on the residual or post-peak strength. See Section 7.4.1 for details.

(b) The stated value under the MDE load case is based on pseudostatic analysis. Performance evaluation of the dam should also take into consideration the time-dependent nature of earthquake excitations and the dynamic response of dam.

(c) Adequate test data must be available through rigorous investigation carried out by qualified professionals.

(d) If PSF values do not meet those listed above, the dam stability is considered acceptable provided the RSF values exceed the minimum.

(e) The minimum values of RSF may be reduced for low hazard potential dams provided data is available to support such reduction.

(f) For low hazard potential dams, if they are judged to be performing satisfactorily, based on an inspection and review of available data, and if conditions are expected to be no less favorable in the future, stability analysis may not be necessary.

(g) These values are recommended where test data is not available.

3 Results of Stability Analyses

3 Results of Stability Analyses

3.1 General Considerations

Stability analyses were undertaken for typical dam sections to establish if dam safety criteria were satisfied for the load cases listed in Table 3.1.

Table 3.1

Load Cases

Load Case	Description	Minimum FOS	
		No 'Cohesion'	With 'Cohesion'*
1	Normal reservoir loading	1.5	3.0
2	Normal winter reservoir levels + ice	1.5	3.0
3	IDF Flood I	1.3	2.0
4	IDF Flood II	1.3	2.0
5	Earthquake	1.0	1.3
6	Post-earthquake loading	1.1	2.0

* With no supporting tests.

For the above loading conditions, the dam safety indicators listed in Table 3.2 are assessed using the input parameters described herein.

Table 3.2

Acceptance Criteria

Dam Safety Indicator	Acceptance Criteria
Location of resultant	Within middle third for normal load cases and within the base for all other load cases.
Bearing stresses	Below allowable bearing capacity limits.
Sliding stability	Above minimum requirements for given load case (see above).
FOS against overturning	This parameter was calculated but there are no specific criteria given in the draft ODSG (acceptance governed by position of resultant).

As described previously, for each load case, the sliding stability was assessed for various assumptions of the threshold shear resistance (cohesion) and frictional resistance of the sliding interface as listed in Table 3.3.

Table 3.3

Range of Frictional Resistance

Shear Strength Parameter	Range of Values Assumed	Remarks
Frictional resistance (N)	43° to 53°	Basic frictional resistance of a smooth surface with a roughness component of 43° with a credible range of ±10°.
Threshold shear strength (ϑ)	0	Value for an unbonded contact, considered to be the most prudent assumption.
	$0.085*\pi f_c'$	Lower bound threshold shear strength value for a bonded discontinuity without tests (draft ODSG, 1999). Actual value of the compressive strength of the concrete in the bonded section assumed to be 50% lower than the measured strength of the intact concrete.
	% bonded area	Assumed to range from 10% to 50% of the base.

3.2 Assumptions

The Bellrock Dam has an overall IHP rating of VERY LOW. It is founded on bedrock, and has bedrock abutments that provide a competent foundation for the dam. Shear strength and concrete properties for the analysis were assumed to be as documented in Table 2.5. Loads and load combinations were taken as discussed in Sections 2.3 and 2.5.

No rock dowels into the foundation were indicated on the drawings and none were considered for any of the structures.

Ice loads were taken as 80.2 kN/m on the concrete and 29.2 kN/m on stop logs based on the discussion in Section 2.3.

Sections chosen for analysis included the sluiceway, the overflow and the gravity structures.

Based on visual inspection, properties of the concrete were taken as

f_c' = 20 MPa
γ_{conc} = 23.5 kN/m^3.

Water levels for the various load cases were as follows:

- summer — headwater level = 140.98 m, tailwater level = 139.00 m
- fall/winter — headwater level = 140.98 m, tailwater level = 139.00 m
- flood I and flood II — headwater level = 142.10 m, tailwater level = 139.97 m.

3.3 Discussion of Results

Detailed results of the stability analysis are found in Appendix A and are summarized in Table 3.4. Three sections were analyzed which included the sluiceway, overflow and gravity structures. Due to the VERY LOW IHP, the sections were not analyzed for earthquake loading.

The results indicate that the gravity section satisfied the summer normal loading conditions. The sluiceway and overflow sections developed cracks at the concrete-rock interface as well as substandard factors of safety versus sliding. This means these two sections do not meet the ODSG requirements under summer normal conditions.

All sections developed unstable cracks under winter ice load conditions and failed to meet the ODSG requirements.

The gravity section satisfied criteria under the 1:50-yr flood condition. The overflow and sluiceway both developed unstable cracks along the base and, therefore, did not satisfy the ODSG requirements.

Table 3.4

Stability Results – Bellrock Dam

Section	Residual	Peak		Load Case	FOS Against Sliding				Location of Resultant	Minimum Base Friction Angle to Satisfy Sliding Criteria (deg)	Minimum % Bonded Area to Satisfy Peak Sliding Criteria	Notes
					Residual Case		Peak					
	Phi (deg)	c (MPa)	Phi (deg)		Req'd	Actual	Req'd	Actual				
Overflow Section	43	0.38	53	Normal	1.5	1.45	3.0	25.20	Outside middle third	44.0	6.11	2, 5
				Normal with ice	1.5	0.25	3.0	0.36	Outside	79.8	38.21	uc, 3
				Flood	1.3	0.47	2.0	14.32	Outside	68.7	7.99	uc, 3
Gravity Section	43	0.38	53	Normal	1.5	6.18	3.0	61.97	Within	12.8	0.00	1
				Normal with ice	1.5	0.78	3.0	1.11	Outside	60.9	13.44	uc, 3
				Flood	1.3	1.97	2.0	23.05	Outside middle third	31.6	1.65	1
Sluiceway	43	0.38	53	Normal	1.5	1.13	3.0	36.82	Outside middle third	51.2	1.08	2, 5
				Normal with ice	1.5	0.23	3.0	0.32	Outside	80.7	4.41	uc, 3
				Flood	1.3	0.16	2.0	20.32	Outside	82.4	1.48	uc, 3

Notes:
uc = unstable crack
Note 1 = dam section satisfies dam safety criteria.
Note 2 = dam section satisfies dam safety criteria under peak strength assumptions.
Note 3 = dam section deemed to satisfy dam safety criteria for low hazard dams [Figure 7.1, Note (f) of the draft ODSG].
Note 4 = bearing stress at toe of dam exceeds criteria.
Note 5 = position of resultant does not satisfy criteria.
Note 6 = does not satisfy dam safety criteria for sliding stability.
Note 7 = rock anchor taken into account.

Since, this dam is very low hazard, then some sections may still be deemed adequate based on observations.

Anchoring of these structures would be required using passive rebar and post-tensioned anchors.

Bibliography

Bibliography

Acres International. "Sliding Resistance of Concrete Gravity Dams," **CEA Research Report CEA 9331 G 2002**. March 1998 (final draft).

Barton, N. R. "The Shear Strength of Rock and Rock Joints," **Int. J. Mech. Min, Sci. & Geomechanics Abstracts 13 (10)**. 1976. pp 1-24.

Brace, W. F. and J. D. Byerlee. "Recent Experimental Studies of Brittle Fracture of Rocks," **Proceedings of the 8th Symposium on Rock Mechanics**. University of Minnesota. 1967.

Deere, D. U. **Dams on Rock Foundations - Some Design Questions**. ASCE Geotechnical Engineering Division Specialty Conference, Rock Engineering for Foundations and Slopes. University of Boulder, Colorado. 1976.

Electric Power and Research Group (EPRI). "Uplift Pressures, Shear Strengths and Tensile Strengths for Stability Analyses of Concrete Gravity Dams", Vol 1, **Report No. EPRI TR-100345**. Denver, Colorado. 1992.

Grishin, M. M. and P. D. Evdokimov. "Shear Strength of Structures Built on Rock," **Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering**. 1961.

Lo, K. Y., T. Ogawa, B. Lukajic, and D. D. Dupak. "Measurement of Shear Strength Parameters of Concrete-Rock Contact at Dam-Foundation Interface," **ASTM Geotechnical Testing Journal**. December 1991.

Ministry of Municipal Affairs and Housing. **Ontario Building Code**. 1997.

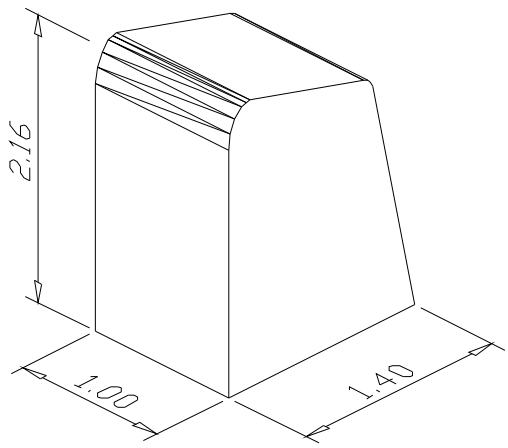
Ontario Ministry of Natural Resources. **Ontario Dam Safety Guidelines, Draft Issue**. Lands and Natural Heritage Branch. September 1999.

Papaliangas, T. Personal communication. 1998.

Appendix A

Detailed Results of Stability Analysis

Calculations	By B.Craig	Date Jan. '04	Project No. P15396.00
ACRES	Checked	Date	Calculation No. 1
Subject Bellrock Dam - Overflow Section			Page 1 of 4
Geometry and Materials			



Geometrical Definitions

Base Elevation	138.790	m
Log Top Elevation (Summer)	140.950	m
H.W.L. (Summer)	140.980	m
T.W.L. (Summer)	139.000	m
Log Top Elevation (Winter)	140.950	m
H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	140.950	m
H.W.L. (Flood I)	142.100	m
T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	140.950	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	140.950	m
Thickness of Deck	0.000	m
Ice Elevation	140.680	m
Volume of Section	2.46	m^3
Centre of Gravity X	0.794	m
Centre of Gravity Y	0.962	m
Length of Pier Section	1.400	m
Width of Pier Section	1.000	m
Length of Sluiceway #1 Section	0.000	m
Width of Sluiceway #1 Section	0.000	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	70.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	33.0	°	Angle of Friction #1
ϕ_2	38.0	°	Angle of Friction #2
ϕ_3	43.0	°	Specified Angle of Sliding Friction
ϕ_4	48.0	°	Angle of Friction #4
ϕ_5	53.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{fc'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{fc'})$
τ_3	1.00	MPa	(0.05fc')
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	30.0	°	Basic Friction Angle

Loadings

0.00	%g	Vertical Ground Acceleration
0.00	%g	Horizontal Ground Acceleration
0.00	%g	Vertical Ground Acceleration (DEice)
0.00	%g	Horizontal Ground Acceleration (DEice)
80.2	kN/m	Ice Force on Concrete
29.2	kN/m	Ice Force on Logs/Gates

Other

Winter	Season for Earthquake Calculation

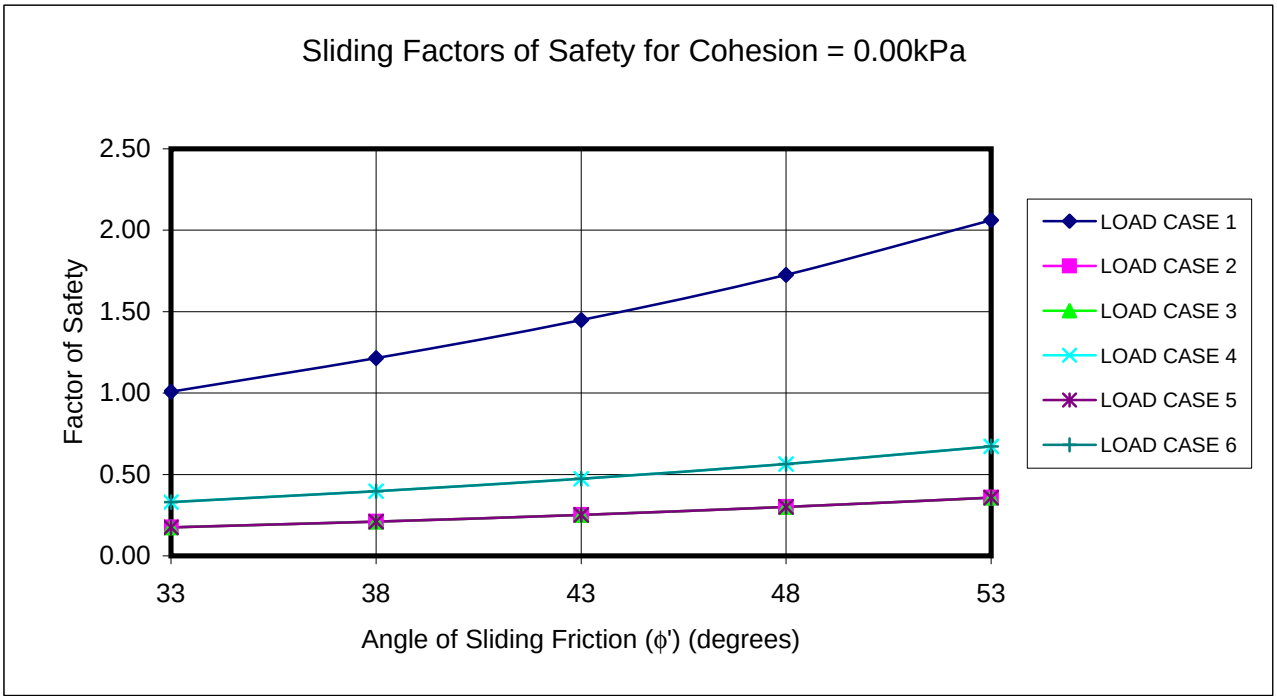
Input Summary

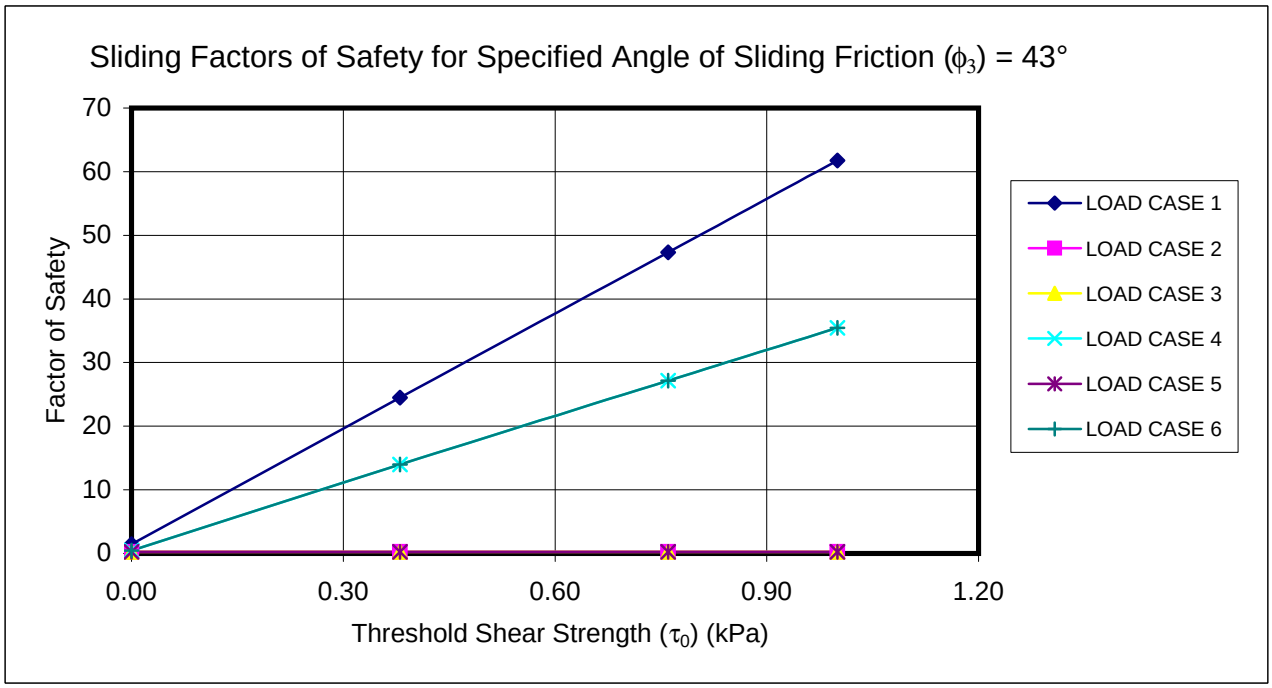
	Load Case #1	#2	#3	#4	#5	#6		
M_1	57.86	57.86	57.86	57.86	57.86	57.86	kN	Weight of Section
V_{water}	0.02	0.02	0.02	0.83	0.02	0.83	m^3	Volume of Water Over Section
M_2	0.18	0.18	0.18	8.10	0.18	8.10	kN	Weight of Water Over Section
x	1.12	1.12	1.12	0.90	1.12	0.90	m	Location of Water Force Along X-Axis
ICE	-	80.20	80.20	-	80.20	-	kN	Total Ice Force
y	-	1.89	1.89	-	1.89	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	0.00	-	kN	Westergaards Force
y	-	-	-	-	0.90	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	0.00	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	0.00	-	%g	Vertical Seismic Coefficient
w_1	23.52	23.52	23.52	47.25	23.52	47.25	kN	Hydrostatic Pressure From Headwater
y	0.73	0.73	0.73	0.91	0.73	0.91	m	Location of Headwater Force Along Y-Axis
w_2	0.22	0.22	0.22	6.83	0.22	6.83	kN	Hydrostatic Pressure From Tailwater
y	0.07	0.07	0.07	0.39	0.07	0.39	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Vertical Force
x	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Vertical Force Along X-Axis

Results (MNR)

		Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	21.83	16.48	16.48	16.48	30.08	30.08	30.08	30.08	30.08	30.08	30.08	30.08
Effective Base	%	60.7	100.0	100.0	100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Length of Base in Compression	m	0.85	1.40	1.40	1.40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Resultant	m	0.283	0.343	0.343	0.343	-5.137	-5.137	-5.137	-5.137	-5.137	-5.137	-5.137	-5.137
Stress at Heel	kPa	0.00	15.71	15.71	15.71	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable
Cracked		YES	NO	NO	NO	YES	YES	YES	YES	YES	YES	YES	YES
Stress at Toe	kPa	-85.27	-75.08	-75.08	-75.08	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		1.29	1.45	1.45	1.45	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
F.S. Sliding ϕ= 33		1.01	23.99	46.83	61.23	0.18	0.18	0.18	0.18	0.18	0.18	0.18	0.18
F.S. Sliding ϕ= 38		1.21	24.23	47.07	61.47	0.21	0.21	0.21	0.21	0.21	0.21	0.21	0.21
F.S. Sliding f= **43**		**1.45**	**24.50**	**47.34**	**61.74**	**0.25**	**0.25**	**0.25**	**0.25**	**0.25**	**0.25**	**0.25**	**0.25**
F.S. Sliding ϕ= 48		1.73	24.82	47.65	62.06	0.30	0.30	0.30	0.30	0.30	0.30	0.30	0.30
F.S. Sliding ϕ= 53		2.06	25.20	48.04	62.44	0.36	0.36	0.36	0.36	0.36	0.36	0.36	0.36
Accepted F.S. Sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Load Case #4 - Flood I				Load Case #5 - Earthquake (Winter)				Load Case #6 - Flood II			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	45.46	30.83	30.83	30.83	30.08	30.08	30.08	30.08	45.46	30.83	30.83	30.83
Effective Base	%	0.0	100.0	100.0	100.0	0.0	0.0	0.0	0.0	0.0	100.0	100.0	100.0
Length of Base in Compression	m	0.00	1.40	1.40	1.40	0.00	0.00	0.00	0.00	0.00	1.40	1.40	1.40
Resultant	m	-0.912	-0.338	-0.338	-0.338	-5.137	-5.137	-5.137	-5.137	-0.912	-0.338	-0.338	-0.338
Stress at Heel	kPa	Unstable	86.55	86.55	86.55	N/A	N/A	N/A	N/A	Unstable	86.55	86.55	86.55
Cracked		YES	NO	NO	NO	N/A	N/A	N/A	N/A	YES	NO	NO	NO
Stress at Toe	kPa	Unstable	-136.74	-136.74	-136.74	Unstable	Unstable	Unstable	Unstable	Unstable	-136.74	-136.74	-136.74
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		0.75	0.82	0.82	0.82	0.24	0.24	0.24	0.24	0.75	0.82	0.82	0.82
F.S. Sliding ϕ= 33		0.33	13.73	26.90	35.20	0.18	0.18	0.18	0.18	0.33	13.73	26.90	35.20
F.S. Sliding ϕ= 38		0.40	13.84	27.01	35.31	0.21	0.21	0.21	0.21	0.40	13.84	27.01	35.31
F.S. Sliding f= **43**		**0.47**	**13.98**	**27.14**	**35.44**	**0.25**	**0.25**	**0.25**	**0.25**	**0.47**	**13.98**	**27.14**	**35.44**
F.S. Sliding ϕ= 48		0.56	14.13	27.30	35.60	0.30	0.30	0.30	0.30	0.56	14.13	27.30	35.60
F.S. Sliding ϕ= 53		0.67	14.32	27.48	35.79	0.36	0.36	0.36	0.36	0.67	14.32	27.48	35.79
Accepted F.S. Sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00


Sliding Factors of Safety for Cohesion = 0.00kPa
Factor of Safety
2.50
2.00
1.50
1.00
0.50
0.00
33
38
43
48
53
Angle of Sliding Friction (ϕ') (degrees)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6

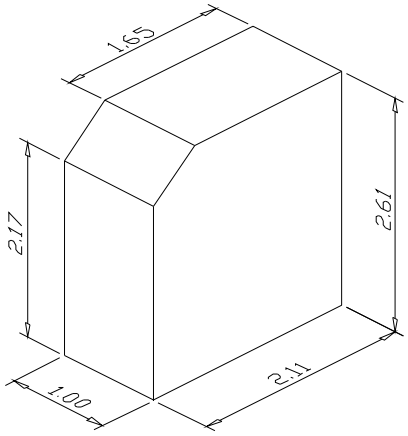

Sliding Factors of Safety for Specified Angle of Sliding Friction (ϕ_3) = 43°
Factor of Safety
70
60
50
40
30
20
10
0
0.00
0.30
0.60
0.90
1.20
Threshold Shear Strength (τ_0) (kPa)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6

Minimum Sliding Angle of Friction to Achieve Required Factor of Safety and % Bonding Results

		Case 1				Case 2				Case 3			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		43.988				79.790				76.201			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		6.43%	3.22%	2.45%		39.56%	19.82%	15.07%		25.72%	12.89%	9.80%
	ϕ_c= 38 ^{o}		6.36%	3.21%	2.44%		39.29%	19.75%	15.03%		25.55%	12.84%	9.77%
	ϕ_c= 43 ^{o}		6.29%	3.19%	2.43%		38.98%	19.67%	14.99%		25.35%	12.79%	9.75%
	ϕ_c= 48 ^{o}		6.20%	3.17%	2.42%		38.63%	19.58%	14.94%		25.12%	12.73%	9.71%
	ϕ_c= 53 ^{o}		6.11%	3.14%	2.40%		38.21%	19.47%	14.87%		24.84%	12.66%	9.67%
Desired F.S. for sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Case 4				Case 5				Case 6			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		68.685				74.881				68.685			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		8.34%	4.18%	3.18%		16.03%	8.03%	6.11%		8.34%	4.18%	3.18%
	ϕ_c= 38 ^{o}		8.27%	4.16%	3.17%		15.92%	8.00%	6.09%		8.27%	4.16%	3.17%
	ϕ_c= 43 ^{o}		8.19%	4.14%	3.16%		15.80%	7.97%	6.07%		8.19%	4.14%	3.16%
	ϕ_c= 48 ^{o}		8.10%	4.12%	3.14%		15.66%	7.94%	6.05%		8.10%	4.12%	3.14%
	ϕ_c= 53 ^{o}		7.99%	4.09%	3.13%		15.48%	7.89%	6.03%		7.99%	4.09%	3.13%
Desired F.S. for sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00

Calculations	By B.Craig	Date Jan. '04	Project No. P15396.00
ACRES	Checked	Date	Calculation No. 2
Subject Bellrock Dam - Gravity Section			Page 1 of 4
Geometry and Materials			



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H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	141.300	m
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T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	141.300	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	141.300	m
Thickness of Deck	0.000	m
Ice Elevation	140.680	m
Volume of Section	5.40	m^3
Centre of Gravity X	1.038	m
Centre of Gravity Y	1.281	m
Length of Pier Section	2.110	m
Width of Pier Section	1.000	m
Length of Sluiceway #1 Section	0.000	m
Width of Sluiceway #1 Section	0.000	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	70.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	33.0	°	Angle of Friction #1
ϕ_2	38.0	°	Angle of Friction #2
ϕ_3	43.0	°	Specified Angle of Sliding Friction
ϕ_4	48.0	°	Angle of Friction #4
ϕ_5	53.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{fc'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{fc'})$
τ_3	1.00	MPa	(0.05fc')
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	30.0	°	Basic Friction Angle

Loadings

0.00	%g	Vertical Ground Acceleration
0.00	%g	Horizontal Ground Acceleration
0.00	%g	Vertical Ground Acceleration (DEice)
0.00	%g	Horizontal Ground Acceleration (DEice)
80.2	kN/m	Ice Force on Concrete
29.2	kN/m	Ice Force on Logs/Gates

Other

Winter	Season for Earthquake Calculation

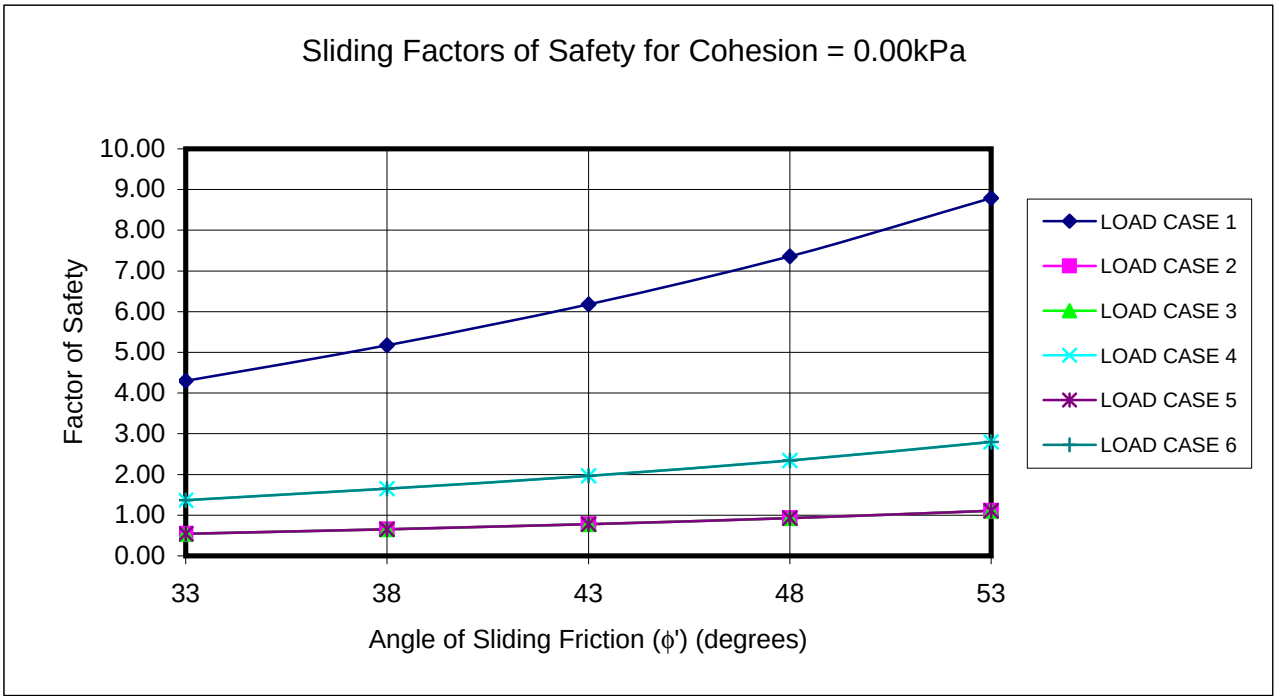
Input Summary

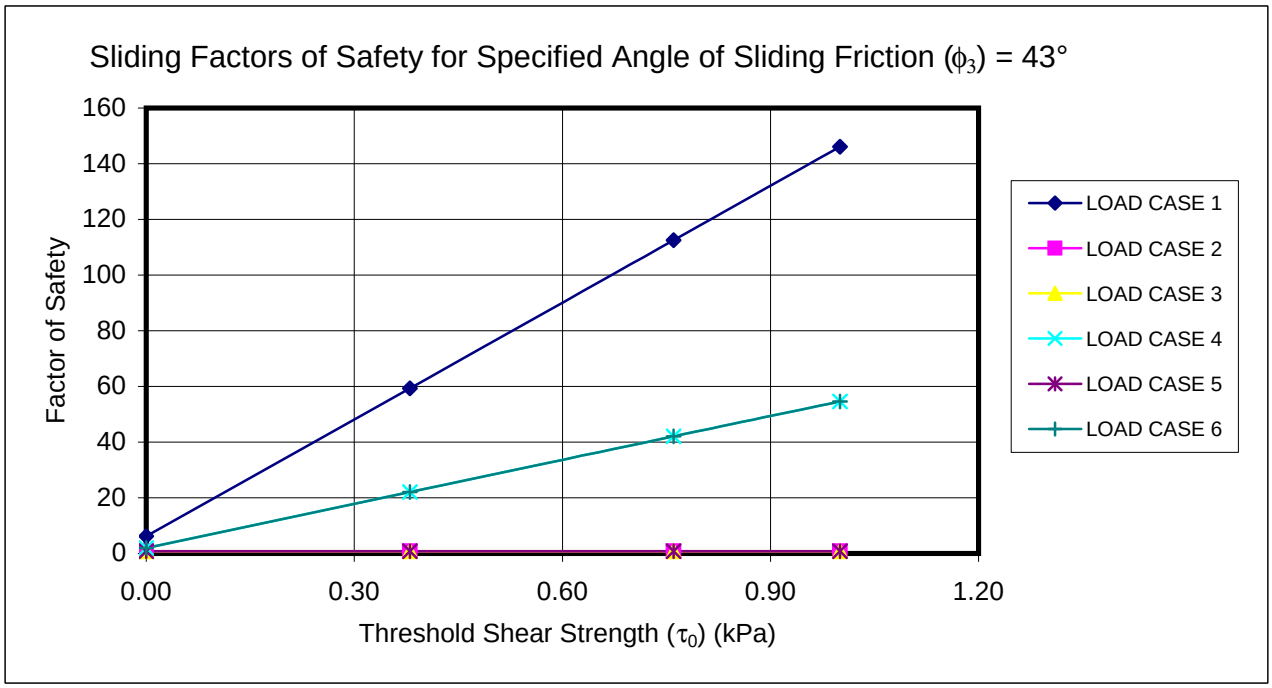
	Load Case							
	#1	#2	#3	#4	#5	#6		
M_1	126.80	126.80	126.80	126.80	126.80	126.80	kN	Weight of Section
V_{water}	0.00	0.00	0.00	1.79	0.00	1.79	m^3	Volume of Water Over Section
M_2	0.00	0.00	0.00	17.55	0.00	17.55	kN	Weight of Water Over Section
x	0.00	0.00	0.00	1.11	0.00	1.11	m	Location of Water Force Along X-Axis
ICE	-	80.20	80.20	-	80.20	-	kN	Total Ice Force
y	-	1.99	1.99	-	1.99	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	0.00	-	kN	Westergaards Force
y	-	-	-	-	0.94	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	0.00	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	0.00	-	%g	Vertical Seismic Coefficient
w_1	25.72	25.72	25.72	53.90	25.72	53.90	kN	Hydrostatic Pressure From Headwater
y	0.76	0.76	0.76	1.04	0.76	1.04	m	Location of Headwater Force Along Y-Axis
w_2	0.47	0.47	0.47	8.04	0.47	8.04	kN	Hydrostatic Pressure From Tailwater
y	0.10	0.10	0.10	0.43	0.10	0.43	m	Location of Tailwater Force Along Y-Axis
H_1	-10.17	-10.17	-10.17	-5.55	-10.17	-5.55	kN	Other Horizontal Force
y	0.45	0.45	0.45	0.45	0.45	0.45	m	Location of Other Horizontal Force Along Y-Axis
V_1	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Vertical Force
x	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Vertical Force Along X-Axis

Results (MNR)

		Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	26.91	26.91	26.91	26.91	47.40	47.40	47.40	47.40	47.40	47.40	47.40	47.40
Effective Base	%	100.0	100.0	100.0	100.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Length of Base in Compression	m	2.11	2.11	2.11	2.11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Resultant	m	0.811	0.811	0.811	0.811	-1.171	-1.171	-1.171	-1.171	-1.171	-1.171	-1.171	-1.171
Stress at Heel	kPa	-14.51	-14.51	-14.51	-14.51	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable
Cracked		NO	NO	NO	NO	YES	YES	YES	YES	YES	YES	YES	YES
Stress at Toe	kPa	-80.17	-80.17	-80.17	-80.17	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable	Unstable
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		2.60	2.60	2.60	2.60	0.59	0.59	0.59	0.59	0.59	0.59	0.59	0.59
F.S. Sliding ϕ= 33		4.30	57.49	110.67	144.21	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
F.S. Sliding ϕ= 38		5.17	58.36	111.54	145.09	0.65	0.65	0.65	0.65	0.65	0.65	0.65	0.65
F.S. Sliding f= **43**		**6.18**	**59.36**	**112.55**	**146.09**	**0.78**	**0.78**	**0.78**	**0.78**	**0.78**	**0.78**	**0.78**	**0.78**
F.S. Sliding ϕ= 48		7.36	60.54	113.73	147.27	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
F.S. Sliding ϕ= 53		8.79	61.97	115.16	148.70	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11
Accepted F.S. Sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Load Case #4 - Flood I				Load Case #5 - Earthquake (Winter)				Load Case #6 - Flood II			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	59.24	48.54	48.54	48.54	47.40	47.40	47.40	47.40	59.24	48.54	48.54	48.54
Effective Base	%	51.5	100.0	100.0	100.0	0.0	0.0	0.0	0.0	51.5	100.0	100.0	100.0
Length of Base in Compression	m	1.09	2.11	2.11	2.11	0.00	0.00	0.00	0.00	1.09	2.11	2.11	2.11
Resultant	m	0.362	0.441	0.441	0.441	-1.171	-1.171	-1.171	-1.171	0.362	0.441	0.441	0.441
Stress at Heel	kPa	0.00	33.93	33.93	33.93	N/A	N/A	N/A	N/A	0.00	33.93	33.93	33.93
Cracked		YES	NO	NO	NO	N/A	N/A	N/A	N/A	YES	NO	NO	NO
Stress at Toe	kPa	-156.75	-124.75	-124.75	-124.75	Unstable	Unstable	Unstable	Unstable	-156.75	-124.75	-124.75	-124.75
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		1.25	1.38	1.38	1.38	0.59	0.59	0.59	0.59	1.25	1.38	1.38	1.38
F.S. Sliding ϕ= 33		1.37	21.44	41.34	53.89	0.54	0.54	0.54	0.54	1.37	21.44	41.34	53.89
F.S. Sliding ϕ= 38		1.65	21.75	41.65	54.20	0.65	0.65	0.65	0.65	1.65	21.75	41.65	54.20
F.S. Sliding f= **43**		**1.97**	**22.11**	**42.01**	**54.56**	**0.78**	**0.78**	**0.78**	**0.78**	**1.97**	**22.11**	**42.01**	**54.56**
F.S. Sliding ϕ= 48		2.34	22.54	42.43	54.98	0.93	0.93	0.93	0.93	2.34	22.54	42.43	54.98
F.S. Sliding ϕ= 53		2.80	23.05	42.95	55.50	1.11	1.11	1.11	1.11	2.80	23.05	42.95	55.50
Accepted F.S. Sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00


Sliding Factors of Safety for Cohesion = 0.00kPa
Factor of Safety
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
33
38
43
48
53
Angle of Sliding Friction (ϕ') (degrees)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6

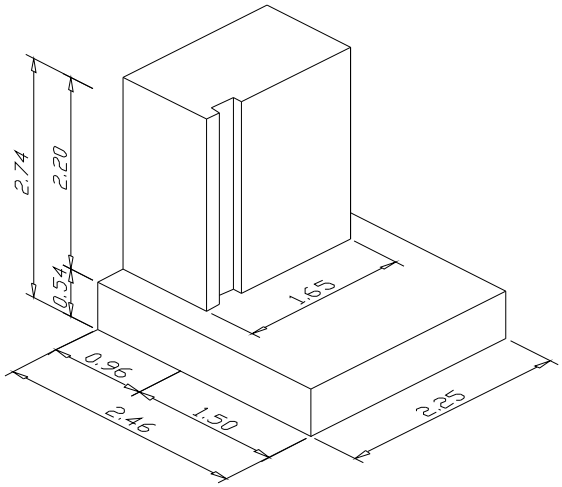

Sliding Factors of Safety for Specified Angle of Sliding Friction (ϕ_3) = 43°
Factor of Safety
160
140
120
100
80
60
40
20
0
0.00
0.30
0.60
0.90
1.20
Threshold Shear Strength (τ_0) (kPa)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6

Minimum Sliding Angle of Friction to Achieve Required Factor of Safety and % Bonding Results

		Case 1				Case 2				Case 3			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		12.760				60.946				52.854			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		0.00%	0.00%	0.00%		14.33%	7.19%	5.47%		8.74%	4.39%	3.34%
	ϕ_c= 38 ^{o}		0.00%	0.00%	0.00%		14.15%	7.14%	5.44%		8.63%	4.36%	3.32%
	ϕ_c= 43 ^{o}		0.00%	0.00%	0.00%		13.94%	7.09%	5.41%		8.50%	4.33%	3.30%
	ϕ_c= 48 ^{o}		0.00%	0.00%	0.00%		13.71%	7.03%	5.38%		8.36%	4.29%	3.28%
	ϕ_c= 53 ^{o}		0.00%	0.00%	0.00%		13.44%	6.96%	5.34%		8.20%	4.24%	3.25%
Desired F.S. for sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Case 4				Case 5				Case 6			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		31.620				50.195				31.620			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		1.78%	0.90%	0.68%		4.83%	2.42%	1.84%		1.78%	0.90%	0.68%
	ϕ_c= 38 ^{o}		1.76%	0.89%	0.68%		4.77%	2.41%	1.83%		1.76%	0.89%	0.68%
	ϕ_c= 43 ^{o}		1.73%	0.88%	0.67%		4.70%	2.39%	1.82%		1.73%	0.88%	0.67%
	ϕ_c= 48 ^{o}		1.69%	0.87%	0.67%		4.62%	2.37%	1.81%		1.69%	0.87%	0.67%
	ϕ_c= 53 ^{o}		1.65%	0.86%	0.66%		4.53%	2.34%	1.80%		1.65%	0.86%	0.66%
Desired F.S. for sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00

Calculations	By B.Craig	Date Jan. '04	Project No. P15396.00
ACRES	Checked	Date	Calculation No. 3
Subject Bellrock Dam - Spillway Section			Page 1 of 4
Geometry and Materials			



Geometrical Definitions

Base Elevation	138.720	m
Log Top Elevation (Summer)	141.010	m
H.W.L. (Summer)	140.980	m
T.W.L. (Summer)	139.000	m
Log Top Elevation (Winter)	141.010	m
H.W.L. (Winter)	140.980	m
T.W.L. (Winter)	139.000	m
Log Top Elevation (Flood I)	141.010	m
H.W.L. (Flood I)	142.100	m
T.W.L. (Flood I)	139.970	m
Log Top Elevation (Flood II)	141.010	m
H.W.L. (Flood II)	142.100	m
T.W.L. (Flood II)	139.970	m
Deck Top Elevation	141.460	m
Thickness of Deck	0.025	m
Ice Elevation	140.680	m
Volume of Section	6.42	m^3
Centre of Gravity X	1.120	m
Centre of Gravity Y	1.002	m
Length of Pier Section	2.250	m
Width of Pier Section	0.960	m
Length of Sluiceway #1 Section	2.250	m
Width of Sluiceway #1 Section	1.500	m
Distance to Edge of Sluiceway #1 Section	0.000	m
Length of Sluiceway #2 Section	0.000	m
Width of Sluiceway #2 Section	0.000	m
Distance to Edge of Sluiceway #2 Section	0.000	m

Material Properties

f_c'	20.00	MPa	Concrete Compressive Strength
f_{b1}	70.00	MPa	Rock Bearing Strength
f_{b2}	0.00	MPa	Till Bearing Strength
ϕ_1	33.0	°	Angle of Friction #1
ϕ_2	38.0	°	Angle of Friction #2
ϕ_3	43.0	°	Specified Angle of Sliding Friction
ϕ_4	48.0	°	Angle of Friction #4
ϕ_5	53.0	°	Angle of Friction #5
τ_n	0.00	MPa	Cohesion
τ_1	0.38	MPa	$(0.17\sqrt{fc'})/2$
τ_2	0.76	MPa	$(0.17\sqrt{fc'})$
τ_3	1.00	MPa	(0.05fc')
γ_{conc}	23.50	kN/m^3	Unit Weight of Concrete
γ_{water}	9.81	kN/m^3	Unit Weight of Water
ϕ_β	30.0	°	Basic Friction Angle

Loadings

0.00	%g	Vertical Ground Acceleration
0.00	%g	Horizontal Ground Acceleration
0.00	%g	Vertical Ground Acceleration (DEice)
0.00	%g	Horizontal Ground Acceleration (DEice)
80.2	kN/m	Ice Force on Concrete
29.2	kN/m	Ice Force on Logs/Gates

Other

Winter	Season for Earthquake Calculation

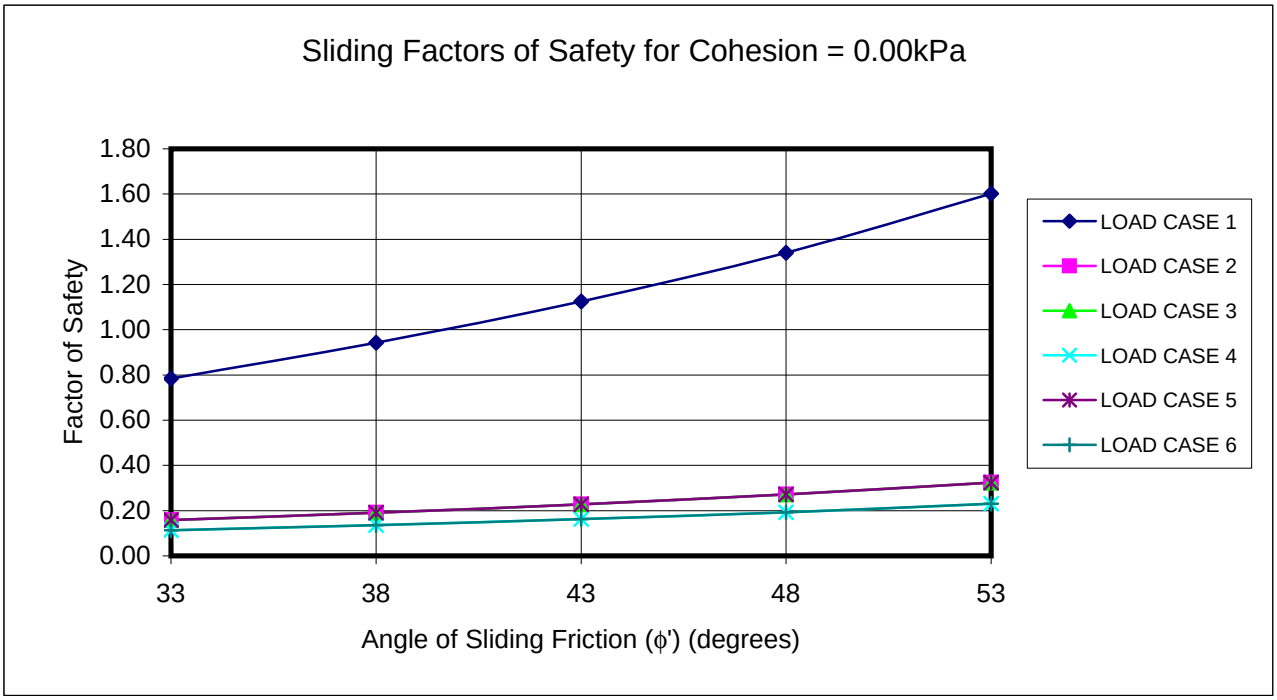
Input Summary

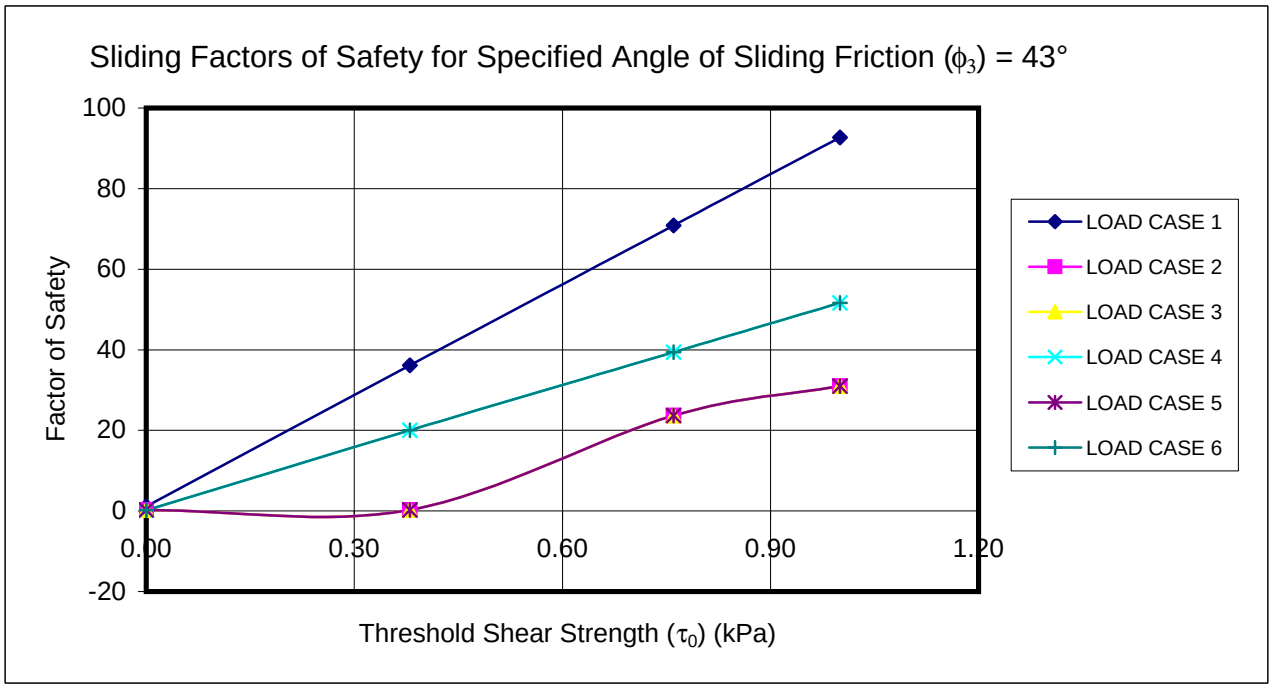
	Load Case							
	#1	#2	#3	#4	#5	#6		
M_1	150.84	150.84	150.84	150.84	150.84	150.84	kN	Weight of Section
V_{water}	1.66	1.66	1.66	5.26	1.66	5.26	m^3	Volume of Water Over Section
M_2	16.25	16.25	16.25	51.56	16.25	51.56	kN	Weight of Water Over Section
x	2.05	2.05	2.05	1.53	2.05	1.53	m	Location of Water Force Along X-Axis
ICE	-	120.79	120.79	-	120.79	-	kN	Total Ice Force
y	-	1.96	1.96	-	1.96	-	m	Location of Ice Force Along Y-Axis
W	-	-	-	-	0.00	-	kN	Westergaards Force
y	-	-	-	-	0.93	-	m	Location of Westergaards along Y-Axis
S_H	-	-	-	-	0.00	-	%g	Horizontal Seismic Coefficient
S_V	-	-	-	-	0.00	-	%g	Vertical Seismic Coefficient
w_1	61.63	61.63	61.63	127.42	61.63	127.42	kN	Hydrostatic Pressure From Headwater
y	0.75	0.75	0.75	1.00	0.75	1.00	m	Location of Headwater Force Along Y-Axis
w_2	0.95	0.95	0.95	18.85	0.95	18.85	kN	Hydrostatic Pressure From Tailwater
y	0.09	0.09	0.09	0.42	0.09	0.42	m	Location of Tailwater Force Along Y-Axis
H_1	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Horizontal Force
y	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Horizontal Force Along Y-Axis
V_1	0.00	0.00	0.00	0.00	0.00	0.00	kN	Other Vertical Force
x	0.00	0.00	0.00	0.00	0.00	0.00	m	Location of Other Vertical Force Along X-Axis

Results (MNR)

		Load Case #1 - Usual (Summer)				Load Case #2 - Usual (Winter)				Load Case #3 - Post-Earthquake (Winter)			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	93.87	68.96	68.96	68.96	122.71	122.71	68.96	68.96	122.71	122.71	68.96	68.96
Effective Base	%	53.7	100.0	100.0	100.0	0.0	0.0	100.0	100.0	0.0	0.0	100.0	100.0
Length of Base in Compression	m	1.21	2.25	2.25	2.25	0.00	0.00	2.25	2.25	0.00	0.00	2.25	2.25
Resultant	m	0.403	0.593	0.593	0.593	-4.933	-4.933	-1.820	-1.820	-4.933	-4.933	-1.820	-1.820
Stress at Heel	kPa	0.00	7.43	7.43	7.43	Unstable	Unstable	121.49	121.49	Unstable	Unstable	121.49	121.49
Cracked		YES	NO	NO	NO	YES	YES	NO	NO	YES	YES	NO	NO
Stress at Toe	kPa	-49.30	-42.89	-42.89	-42.89	Unstable	Unstable	-156.95	-156.95	Unstable	Unstable	-156.95	-156.95
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		1.17	1.40	1.40	1.40	0.48	0.48	0.53	0.53	0.48	0.48	0.53	0.53
F.S. Sliding ϕ= 33		0.78	35.72	70.39	92.26	0.16	0.16	23.54	30.85	0.16	0.16	23.54	30.85
F.S. Sliding ϕ= 38		0.94	35.94	70.61	92.47	0.19	0.19	23.61	30.92	0.19	0.19	23.61	30.92
F.S. Sliding f= 43		**1.13**	**36.18**	**70.85**	**92.72**	**0.23**	**0.23**	**23.69**	**31.00**	**0.23**	**0.23**	**23.69**	**31.00**
F.S. Sliding ϕ= 48		1.34	36.47	71.14	93.01	0.27	0.27	23.79	31.10	0.27	0.27	23.79	31.10
F.S. Sliding ϕ= 53		1.60	36.82	71.49	93.36	0.32	0.32	23.91	31.22	0.32	0.32	23.91	31.22
Accepted F.S. Sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Load Case #4 - Flood I				Load Case #5 - Earthquake (Winter)				Load Case #6 - Flood II			
Cohesion	Mpa	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
% Uplift at Upstream Face	%	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Total Uplift	kN	183.53	125.70	125.70	125.70	122.71	122.71	68.96	68.96	183.53	125.70	125.70	125.70
Effective Base	%	0.0	100.0	100.0	100.0	0.0	0.0	100.0	100.0	0.0	100.0	100.0	100.0
Length of Base in Compression	m	0.00	2.25	2.25	2.25	0.00	0.00	2.25	2.25	0.00	2.25	2.25	2.25
Resultant	m	-4.123	-0.449	-0.449	-0.449	-4.933	-4.933	-1.820	-1.820	-4.123	-0.449	-0.449	-0.449
Stress at Heel	kPa	Unstable	44.30	44.30	44.30	N/A	N/A	121.49	121.49	Unstable	44.30	44.30	44.30
Cracked		YES	NO	NO	NO	N/A	N/A	NO	NO	YES	NO	NO	NO
Stress at Toe	kPa	Unstable	-72.02	-72.02	-72.02	Unstable	Unstable	-156.95	-156.95	Unstable	-72.02	-72.02	-72.02
Allowable Stress at Toe	kPa	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000	-9000
F.S. Overturning		0.77	0.88	0.88	0.88	0.48	0.48	0.53	0.53	0.77	0.88	0.88	0.88
F.S. Sliding ϕ= 33		0.11	19.84	39.22	51.44	0.16	0.16	23.54	30.85	0.11	19.84	39.22	51.44
F.S. Sliding ϕ= 38		0.14	19.93	39.31	51.53	0.19	0.19	23.61	30.92	0.14	19.93	39.31	51.53
F.S. Sliding f= 43		**0.16**	**20.04**	**39.42**	**51.64**	**0.23**	**0.23**	**23.69**	**31.00**	**0.16**	**20.04**	**39.42**	**51.64**
F.S. Sliding ϕ= 48		0.19	20.16	39.54	51.77	0.27	0.27	23.79	31.10	0.19	20.16	39.54	51.77
F.S. Sliding ϕ= 53		0.23	20.32	39.70	51.92	0.32	0.32	23.91	31.22	0.23	20.32	39.70	51.92
Accepted F.S. Sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00


Sliding Factors of Safety for Cohesion = 0.00kPa
Factor of Safety
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
33
38
43
48
53
Angle of Sliding Friction (ϕ') (degrees)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6


Sliding Factors of Safety for Specified Angle of Sliding Friction (ϕ_3) = 43°
Factor of Safety
100
80
60
40
20
0
-20
0.00
0.30
0.60
0.90
1.20
Threshold Shear Strength (τ_0) (kPa)
LOAD CASE 1
LOAD CASE 2
LOAD CASE 3
LOAD CASE 4
LOAD CASE 5
LOAD CASE 6

Minimum Sliding Angle of Friction to Achieve Required Factor of Safety and % Bonding Results

		Case 1				Case 2				Case 3			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		51.186				80.741				77.467			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		1.12%	0.56%	0.43%		4.47%	2.11%	1.61%		2.91%	1.34%	1.02%
	ϕ_c= 38 ^{o}		1.11%	0.56%	0.42%		4.46%	2.11%	1.60%		2.90%	1.33%	1.01%
	ϕ_c= 43 ^{o}		1.10%	0.56%	0.42%		4.44%	2.10%	1.60%		2.90%	1.33%	1.01%
	ϕ_c= 48 ^{o}		1.09%	0.55%	0.42%		4.43%	2.09%	1.59%		2.88%	1.32%	1.01%
	ϕ_c= 53 ^{o}		1.08%	0.55%	0.42%		4.41%	2.08%	1.59%		2.87%	1.31%	1.00%
Desired F.S. for sliding		1.50	3.00	3.00	3.00	1.50	3.00	3.00	3.00	1.10	2.00	2.00	2.00

		Case 4				Case 5				Case 6			
	τ	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00	0.00	0.38	0.76	1.00
Minimum angle of sliding friction to satisfy F.S. against sliding (deg.)		82.384				76.260				82.384			
Percentage of bonded area to satisfy peak sliding F.S.	ϕ_c= 33 ^{o}		1.52%	0.76%	0.58%		1.82%	0.79%	0.60%		1.52%	0.76%	0.58%
	ϕ_c= 38 ^{o}		1.51%	0.76%	0.58%		1.82%	0.79%	0.60%		1.51%	0.76%	0.58%
	ϕ_c= 43 ^{o}		1.50%	0.75%	0.57%		1.81%	0.79%	0.60%		1.50%	0.75%	0.57%
	ϕ_c= 48 ^{o}		1.49%	0.75%	0.57%		1.81%	0.78%	0.60%		1.49%	0.75%	0.57%
	ϕ_c= 53 ^{o}		1.48%	0.75%	0.57%		1.80%	0.78%	0.59%		1.48%	0.75%	0.57%
Desired F.S. for sliding		1.30	2.00	2.00	2.00	1.00	1.30	1.30	1.30	1.30	2.00	2.00	2.00

Part V
Recommendations and Costs

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Part V – Recommendations and Costs

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1 Introduction

As a result of the dam safety review for Bellrock Dam, a number of recommended actions and maintenance activities were identified that are intended to address concerns related to satisfying current dam safety criteria provided in the draft Ontario Dam Safety Guidelines (ODSG), (Ministry of Natural Resources, 1999). These ranged from maintenance activities to more significant issues that may require rehabilitation work. In each case, an attempt was made to prioritize the remedial work requirements.

2 Recommendations

The dam is in fair to good condition. The dam is clearly founded on bedrock. There are no as-built drawings available. Crysler & Lathem Ltd. (CCL) conducted a study in 1977 to 1978 that examined hydraulic aspects of this site and the stability of the dam.

Three sections were analyzed which included the sluiceway, overflow and gravity structures. Due to the VERY LOW incremental consequence category, the sections were not analyzed for earthquake loading.

The results indicate that the gravity section satisfied the summer normal loading conditions. The sluiceway and overflow sections developed cracks at the concrete-rock interface as well as substandard factors of safety versus sliding. This means these two sections do not meet the ODSG requirements under summer normal conditions.

All sections developed unstable cracks under winter ice load conditions and failed to meet the ODSG requirements.

The gravity section satisfied criteria under the 1:50-yr flood condition. The overflow and sluiceway both developed unstable cracks along the base and, therefore, did not satisfy the ODSG requirements.

The following action plan is recommended:

1 Remove deteriorated concrete and apply repair material.

2 Anchoring of the structures would be required using passive rebar and post-tensioned anchors.

3 Monitor/inspect seepage at the right and left abutments.

3 Cost of Remedial Works

Priority numbers were assigned to the remedial measures with an explanation of these numbers in Table 3.1.

Table 3.1

Explanation of Priority Numbers

Priority	Description
1	Immediate - Corrective action required immediately due to safety concerns.
2	High - Corrective action required within 2 years.
3	Medium - Corrective action required within 5 years.
4	Low - Corrective action required within 10 years.
5	Monitoring - Defect should be monitored, with corrective action to be taken only when required.

Note: Each level reflects the relative importance or urgency associated with taking some form of action. In cases in which the defects were observed to be safety related (mostly Priority 1 items), action means actual construction. It is noted that some of the lower Priority items may need to be reassigned a higher priority once the areas have been monitored and investigated and any defects have been identified.

For each of the recommended issues, conceptual-level cost estimates were developed, based on an assessment of the general scope of work and typical unit price data from similar work in Ontario. It should be noted that the cost estimates that were developed were made on the basis of the actual estimated direct construction costs for the individual remedial action identified. As details of the contracting methods are not known at this time, other costs (such as mobilization,

control of water, increased access costs, contingency and engineering costs) were estimated on the basis of a percentage of the contract price according to the general guidelines summarized in Table 3.2.

Table 3.2

Summary of Additional Costs Associated with a Typical Remedial Repair Project

Item	Cost
Mobilization and demobilization	5% to 7% of capital cost.
Control of water during construction	3% to 10% of capital cost (can vary significantly, depending on complexity).
Barge access	10% to 20% of capital cost.
Permitting costs	$5,000 to $10,000.
Contingency	20% to 30% of capital cost.
Engineering and supervision	8% to 15% of capital cost.

Details of the recommended action plan and associated direct costs for Bellrock Dam are summarized in Table 3.3. An overall cost summary of the remedial repairs, including allowances for contractor mobilization/demobilization, control of water, contingency and engineering, is provided in Table 3.4.

Table 3.3

Bellrock Dam Remedial Measures

Item No.	Structure	Component	Defect/Area of Concern	Repair	Repair Type	Rate	Unit	Estimated Cost (2004 $)	Priority
1	Dam	Left and right abutments	Seepage	Leakage monitoring and piezometers installation	Dam safety	$10,000	lump sum	$10,000	2
2	Dam	All	Maintenance and stability	Concrete repair and stabilizing measures	Dam safety	$25,000	lump sum	$25,000	2
3	Damsite		Safety issues	Signage, access and boom	Public safety	$5,000	lump sum	$5,000	1

Note: Construction cost estimates do not include costs for contractor mobilization/demobilization, dewatering, contingency, engineering design or site supervision.

Table 3.4

Bellrock Dam Remedial Measures Cost Summary

Priority	1	2	3	4	5	Rounded Total
Base Estimated Construction Costs (Table 3.3)	$5,000	$35,000	$0	$0	$0	$40,000
Permitting	$0	$0	$0	$0	$0	$0
Mobilization/Demobilization - 15%	$750	$5,250	$0	$0	$0	$6,000
Control of Water - 5%	$250	$1,750	$0	$0	$0	$2,000
Barge Access - 0%	$0	$0	$0	$0	$0	$0
Subtotal	**$6,000**	**$42,000**	**$0**	**$0**	**$0**	**$48,000**
Contingency - 25%	$1,500	$10,500	$0	$0	$0	$12,000
Engineering and Supervision - 10%	$750	$5,250	$0	$0	$0	$6,000
TOTAL ESTIMATED CONSTRUCTION COSTS	**$8,250**	**$57,750**	**$0**	**$0**	**$0**	**$66,000**