APPENDICES

BEC-M12795-Appendices

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Proposal for Owner's Engineer Services:
Factorydale Turbine Project – Town of Berwick Electric Commission

[THIS ATTACHMENT IS CONFIDENTIAL]

IR-5 APPENDIX 2

Berwick Electric Commission - Request for Proposals

REQUEST FOR PROPOSALS

Berwick Electric Commission

Owner's Engineer for Factorydale Turbine Project

BER2024-003

Issue Date: April 22, 2024

Closing Date: May 13, 2024

Closing Time: 2:00 pm ADT

Address
Berwick Town Hall
236 Commercial Street, Berwick NS B0P 1P0

Contact
Name: Glen Bannon, Berwick Electric Commission
Email: adminberwick.ca

INVITATION

The Berwick Electric Commission (BEC) is a municipally owned electric utility that provides electricity to over 1,600 customers in and to the south of the Berwick Electric Commission of Berwick, Nova Scotia proper. BEC owns and operates a hundred-year-old hydroelectrical facility in Factorydale, Nova Scotia which generates on average 4% of the utility's overall energy load. In early December 2023, the wicket gates and turbine vanes at the Factorydale facility were severely damaged and the plant is now out of commission.

BEC is seeking an Engineering Firm with hydroelectric generation and construction experience to serve as Owner's Engineer (OE) for the duration of the project, to provide a costed options analysis on potential remedies to the situation, and engineering consulting services in support of all phases of implementation of the chosen option. A detailed scope of work is included in this Request for Proposals.

The OE shall work directly for BEC and it is the intent of the BEC to enter into a contract for the work described in this document with the successful Proponent. The OE's role is to ensure that implementation of the Factorydale solution is completed in full compliance with project scope documents and all applicable construction and safety practices.

There will be an optional site visit organized for May 2, 2024, at 1:00 pm ADT. Attendance at this site visit is optional, but encouraged, for consideration for this work.

BEC staff will be available at this visit and at critical junctures throughout the project.

A complete Proposal must be received before **2:00 p.m. ADT May 13, 2024** in order to be considered.

Proponents may submit one hard copy in a sealed envelope, or one PDF copy marked **"BER2024-003: BEC Owner's Engineer for Factorydale Turbine Project"** to:

Glen Bannon
Berwick Electric Commission
236 Commercial Street, PO Box 130
Berwick, NS B0P 1E0

admin@berwick.ca

SECTION 1: PROJECT OVERVIEW AND REQUIREMENTS

1.1 OVERVIEW

As described in the Invitation, the wicket gates and turbine vanes at the BEC hydroelectric plant in Factorydale, Kings County, NS were severely damaged in early December 2023. The plant is now out of commission.

1.2 SCOPE OF WORK

BEC is seeking an Engineering Firm with hydroelectric generation and construction experience to serve as Owner's Engineer (OE) for the duration of the project, to provide a costed options analysis on potential remedies to the situation, and engineering consulting services in support of all phases of implementation of the chosen option. The scope of the Owner's Engineer work shall include but not be limited to:

a. Review all relevant documentation concerning the Factorydale hydroelectric facility;

b. Inspect and assess the damaged turbine;

c. Perform a costed options analysis (indicative) of potential remedies to be completed and delivered to BEC no later than 28 June 2024. This options analysis is to include but not be limited to:

   i. Repair;

   ii. Replacement;

   iii. Decommissioning and site remediation;

d. On selection of the recommended option:

   i. Prepare relevant tender documentation for BEC approval and issue;

   ii. Create scoring criteria for review and approval by BEC. Review and score responses from Proponents, provide advice on selection;

   iii. Provide expertise and advice during contract negotiations;

   iv. Consult with regulations and regulatory officials as applicable to confirm regulatory compliance of methods and work proposed;

   v. Review all engineering drawings including architectural/landscaping drawings from pre-design through to final construction and record drawings in hard copy and/or electronic format for final delivery to BEC;

   vi. Review the Proponent's Environmental Management Plan and ensure proper implementation (supported by all permits required) during all phases of construction;

vii. Review and comment on any engineering change proposals submitted by the Proponent;

viii. Provide technical representation in support of BEC during any disputes;

ix. Provide on site monitoring when applicable and weekly reporting on project progress and prompt indications of any deviations from BEC's requirements; and

x. Provide verification of Proponent progress reports to assist authorization of progress payments.

1.3 SITE VISIT

An optional site visit will be hosted on May 2, 2024 at 1:00 pm ADT at 2545 Harmony Road, Aylesford, Nova Scotia.

1.4 REQUIREMENTS

Certificate of Compliance Workers Compensation

The successful proponent will be expected to supply a copy of their current and valid safety accreditation issued by Nova Scotia Workers' Compensation Board or Certificate of Recognition (COR) issued by Construction Safety Nova Scotia.

The successful proponent will be expected to provide a valid clearance letter issued by the Worker's Compensation Board of Nova Scotia.

Insurance and Liability Requirements

The successful Proponent shall provide the following insurance coverage:

- Commercial general liability insurance for all operations and activities of the Proponent; including listing the Berwick Electric Commission as an additional insured and providing a certificate of insurance;
- Professional insurance with exclusive limits applicable to the project in question; coverage shall commence from the start of the project and be maintained through to completion; and
- Limits of insurance will be sufficient for type of project in question. Minimum acceptable limit to be $2,000,000.

1.5 DELIVERABLES

Proposals should be detailed enough to demonstrate how the Proponent's expertise, staff, and resources best meet the needs of the Berwick Electric

Commission, as described in this Request for Proposal ("RFP"). The proposal shall include the following:

- Corporate profile, including a summary of project management and technical support services;
- List of key personnel responsible for the project;
- List of sub-contractors, if any, and the components for which they will be responsible;
- Proposed project schedule and timeline for completion; and
- At least three references for similar projects completed in the last five (5) years.

1.6 EVALUATION CRITERIA

Each proposal will be evaluated to determine the degree to which it responds to the requirements as set out in this document.

- Knowledge and Experience (40%)– Demonstrated knowledge and experience in completing similar projects.
- Overall Design (40%)- Expected production outcomes.
- Cost– (20%) Cost schedule as noted in work plan.

Whenever possible the Berwick Electric Commission will endeavor through its procurement process to open opportunities to local business. In the event where competitive submissions are judged to be equal, and where the submissions include a local firm, the award shall go to the local firm. This mechanism shall be known as *the tie-breaker provision.*

1.7 REVIEW PANEL

All submissions received prior to closing will be evaluated by a *Review Panel* consisting of the following:

- Management Representative, BEC
- Chief Administrative Officer
- Director of Finance

The *Review Panel* reserves the right to perform any of the following and take the information obtained into account in evaluating a Proposal including the right to:

Seek clarification or verify information provided by a Proponent with respect to this RFP;

Contact any or all of the references supplied by a Proponent to verify any information or data submitted by the Proponent and to obtain information about past performance;

Interview either via teleconference or at the Berwick Electric Commission’s offices any or all of the key personnel

SECTION 2 – PROPOSAL SUBMISSION

3.1 PROPOSAL SUBMISSION

A complete Proposal must be received before **2:00 p.m. local time May 13, 2024** in order to be considered.

Proponents may submit one hard copy in a sealed envelope, or one PDF copy marked **"BER2024-003: BEC Factorydale Turbine Project"** to:

Glen Bannon
Berwick Electric Commission
236 Commercial Street, PO Box 130
Berwick, NS B0P 1E0

admin@berwick.ca

3.2 CLARIFICATION AND ADDENDA

All questions concerning this Proposal shall be directed in writing to the following:

Glen Bannon, BEC
Email: admin@berwick.ca

Any changes to this RFP shall be stated in writing by Addenda. Verbal statements made by Berwick Electric Commission staff or their representatives shall not be binding.

3.3 AMENDMENT OR WITHDRAWAL OF TENDER

Proposals may only be amended or withdrawn by using the same method as proposal submission prior to the time of RFP Closing.

3.4 OFFER, ACCEPTANCE, REJECTION

The Owner reserves the right to accept or reject any tender, and to cancel the tendering process and reject all tenders at any time prior to the award of Contract without incurring any liability to affected tenderers, including without limitation the lowest tender, and to award the Contract to whomever the Owner in its sole and absolute discretion deems appropriate notwithstanding any custom of the trade nor anything contained in the Contract Documents or herein. The Owner shall not, under any circumstances, be responsible for any costs incurred by the Tenderer in preparing of its Tender.

Without limiting the generality of the foregoing, the Owner reserves the right, in its sole and absolute discretion, to accept or reject any Tender in which the view of the Owner is incomplete, obscure, or irregular, which has erasures and corrections in the documents, which contains exceptions and variations, which omits one or more prices, or which contains prices the Owner considers unbalanced.

Criteria which may be used by the Owner in evaluating tenders and awarding the Contract are in the Owner's sole and absolute discretion and, without limiting the generality of the foregoing, may include one or more of: price, total cost to the Owner, the amount of Nova Scotia content; the amount of Canadian content; reputation; claims history of Tenderer; qualifications and experience of Tenderer and its personnel; quality of services and personnel proposed by the Tenderer; ability of the Tenderer to ensure continuous availability of qualified and experienced personnel; the Construction Schedule and Plan; the proposed Labour and Equipment; and the proposed Supervisory Staff.

Should the Owner not receive any tender satisfactory to the Owner in its sole and absolute discretion, the Owner reserves the right to re-tender the Project, or negotiate a contract for the whole or any part of the Project with anyone or more persons whatsoever, including one of more of the Tenderers.

3.5 DISQUALIFICATION FOR INAPPROPRIATE CONTACT

Any attempt by the Proponent or any of its employees, agents, contractors, or representatives to contact members of Berwick Electric Commission Council, Berwick Electric Commission staff, or members of the review panel not identified in this RFP with respect to this RFP or the Services prior to awarding the contract for this RFP may lead to disqualification.

3.6 CONFIDENTIALITY

All documents provided during the RFP process may not be used for any purpose other than the submission of a proposal. Proponents shall not use information obtained through the RFP process without written permission from the Berwick Electric Commission.

3.7 FREEDOM OF INFORMATION AND PROTECTION OF PRIVACY

Proponents agree to public disclosure of the contents of its submission in response to the RFP subject to the provisions of the *Municipal Government Act* relating to Freedom of Information and Protection of Privacy. Anything in the submission that the Proponent considers to be "personal information" or "confidential information" of a proprietary nature should be marked confidential and will be subject to appropriate consideration of the *Municipal Government Act* as noted above. The work described in this RFP is being conducted with public funds, and the fees and expenses proposed in the Proponent's submission will be made public.

3.8 CONFLICT OF INTEREST

Proponents and their employees shall take all reasonable steps to ensure avoidance of all direct or indirect conflicts of interest between any of their individual interests and those of the Berwick Electric Commission. If the Proponent or any one of its personnel becomes aware of any reasonable possibility of any such conflicts, then the Proponent shall promptly disclose to the Berwick Electric Commission the facts and circumstances pertaining to same.

PO Box 130, 236 Commercial Street, Berwick, Nova Scotia
B0P 1E0 www.berwick.ca

902-538-8068

Addendum #1
10 May 2024

BEC 2024-003
Owner's Engineer For Factorydale Turbine Project

The following changes and/or modification(s) shall be made to the Tender document BEC 2024-003:

1. CLOSING DATE
Cover Page 1, Pages 2 and 7
Amend Closing Date to read:

A complete Proposal must be received before **2:00 p.m. ADT May 27, 2024** in order to be considered.

Acknowledgement of Addendum:
The proposer acknowledges receipt of this addendum and understands its contents and the respective changes to the request for tender document. Additionally, please include copies of all signed addenda with your tender.

Signed: ____________________________ Date:______________________________

Company Name:____

PO Box 130, 236 Commercial Street, Berwick, Nova Scotia
B0P 1E0 www.berwick.ca

902-538-8068

Addendum #2
24 May 2024

BEC 2024-003
Owner's Engineer For Factorydale Turbine Project

The following changes and/or modification(s) shall be made to the Tender document BEC 2024-003:

1. CLOSING DATE
Cover Page 1, Pages 2 and 7
Amend Closing Date to read:

A complete Proposal must be received before **2:00 p.m. ADT June 10, 2024** in order to be considered.

Acknowledgement of Addendum:
The proposer acknowledges receipt of this addendum and understands its contents and the respective changes to the request for tender document. Additionally, please include copies of all signed addenda with your tender.

Signed: ____________________________ Date:______________________________

Company Name:____

IR-7 APPENDIX 1

SECTION 1 – NEW TRANSFORMER PROCUREMENT PACKAGE
SECTION 2 - DECOMMISSIONING PROCUREMENT PACKAGE
SECTION 3 - NEW UNIT PROCUREMENT PACKAGE
SECTION 4 – INTAKE SYSTEM PROCUREMENT PACKAGE
SECTION 5 - LOG BOOM PROCUREMENT PACKAGE

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IR-19 APPENDIX 1

2024 Dam Safety Review

Report on

2024 Dam Safety Review

Factorydale Dam

Berwick, NS

Draft | Revision 0 | December 19, 2024

Meco Project Number: 10777

2024 DAM SAFETY REVIEW

FACTORYDALE DAM

Prepared for

BERWICK ELECTRIC COMMISSION

Berwick Town Hall
236 Commercial Street
Berwick, NS | B0P 1P0

Prepared by

MITCHELMORE ENGINEERING COMPANY LIMITED

109 Ilsley Avenue, Unit 14
Dartmouth, NS | B3B 1S8

Tel: (902) 444-3131
Fax: (902) 404-7777

Document Details			
Meco Project No.:	10777	Date Submitted:	December 19, 2024
Issue Status:	Draft	Revision No.:	0
Prepared By:	Perry Mitchelmore, P.Eng., PMP		
Reviewed By:	Hamid Goharnejad, P.Eng., PhD		

Document Revision History					
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Issue Status:		Revision Number:		Date:	
Issue Status:		Revision Number:		Date:	
Issue Status:		Revision Number:		Date:	

EXECUTIVE SUMMARY

Mitchelmore Engineering Company Ltd., (Meco) has performed the 2024 Dam Safety Review of the Factorydale Dam, located near Berwick, NS. The system has one (1) powerhouse with an installed capacity of 4.1 MW and a rated head of 80 metres.

Factorydale Dam is regulated by the Nova Scotia Department of Environment and Climate Change (NSECC) through a water withdrawal agreement. With an estimated reservoir storage capacity of less than 30,000 m^3, the dam as it is currently constructed does not meet the criteria of a dam, as defined by the CDA Guidelines, and compliance with the Guidelines is voluntary. To achieve compliance, a governance framework that integrates dam safety management into existing management structures is suggested.

The DSR reviewed the current consequence classification, and the inundation map contained in the Emergency Preparedness Plan (EPP). The review identified less flood risks than indicated in the inundation maps that suggest consequence classification can be changed from High to Significant. The recommended design criteria for a Significant consequence classification dam as presented below.

Structure Name	Previous Classification	Recommended Classification	Recommended IDF	Recommended EDGM
Factorydale Dam and Spillway	High	Significant	$Q_{1,000}$ (123 m^3/s)	$AEP_{1,000}$ (PGA 0.046g)

Factorydale Dam and Spillway was evaluated based on a visual assessment of their physical condition and a review of design adequacy August 8, 2024. The dam and spillway were observed to be in Fair condition and not under duress or at threat of imminent failure. Minor deficiencies were observed including aspects of the dam rehabilitation in 2008 that were not installed.

A design adequacy check was completed to a preliminary level. Hydraulic routing of the inflow hydrology estimated the maximum flood level (MFL) during the IDF is elevation 69.93 metres, which is 1.72 metres of surcharge at the spillway and 0.12 m higher than the previous MFL. A design adequacy assessment verified that the dam is compliant with standards-based criteria for all loading, including the higher flood level. The concrete dam will overtop frequently, but the concrete dam is not believed to be susceptible to undermining. The only vulnerable area is at the far extreme wing of the right abutment, a low-head area that was not post-tensioned, as intended in 2008.

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1 INTRODUCTION

Acting on the authorization of the Berwick Electric Commission (BEC), Mitchelmore Engineering Company Ltd., (Meco) has performed the 2024 Dam Safety Review (DSR) of Factorydale Dam, located near Berwick, NS. The work was completed in conformance with request for proposal (RFP) BER 25024-002 and Meco proposal 10777 submitted June 27, 2024.

1.1 STRUCTURE OF REPORT

The report is structured to allow the reader to review individual sections independent of the entire report. Section 2 provides a description of the system and a summary of available literature describing facilities. Section 3 provides a review of hydrology. Section 4 describes the results of the DSR. Conclusions and recommendations are provided in Section 5. Supporting analysis is included in the appendices.

1.2 SITE DATUM AND LOCAL CONTROL

The project drawings provided for the review are represented in a local datum. For project elevation data, a 1m Digital Elevation Model (DEM) was retrieved from the Government of Nova Scotia GeoNova Database (geonova.novascotia.ca) which uses the vertical datum "Canadian Geodetic Vertical Datum 2013" (CGVD2013). Map coordinates for the model were taken from the NAD83 MTM Zone 5 projection. All elevations reported are referenced to CGVD2013 datum.

2 System Description And Background

Factorydale Dam is located at 2545 Harmony Road, within the limits of the Town of Berwick. The dam and spillway were rehabilitated in 2008 with post-tensioned anchors and rockfill buttressing at the left abutment, and the spillway crest was upgraded to an ogee shape with a concrete overlay.

2.1 Factorydale Reservoir

Factorydale Dam has a surface area of three (3) hectares and an estimated live storage of 23,500 m^3 at Full Supply Level (FSL), which is defined as the concrete sill of the spillway with stoplogs removed. There is no bathymetry of the reservoir to verify the stage-storage relationship. The layout of the Factorydale Dam components are shown in Figure 2.1.

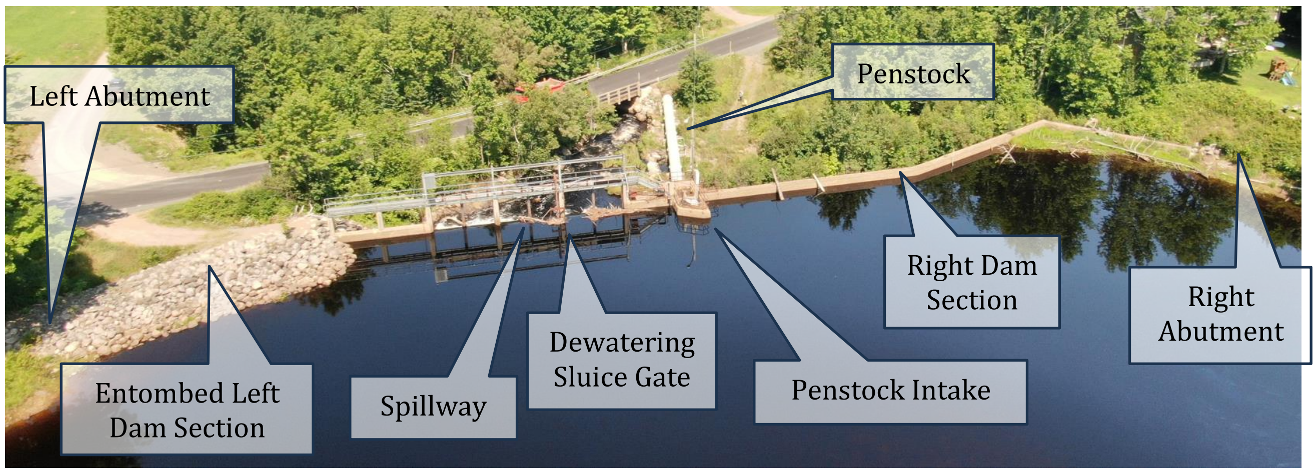

Figure 2.1 Factorydale Dam

2.1.1 Factorydale Dam

The Factorydale Dam is a concrete gravity dam measuring 113 meters in length and reaching a maximum height of 5.2 meters, although much of the structure stands around 2 meters, or less. It features an 18.7-meter-long spillway section designed for flood control, equipped with 750mm high wooden stoplog gates to regulate the water level by adjusting the forebay height. The spillway crest is modified into six (6) bays equipped with 750mm high stoplogs that are operated by a steel headframe and from a steel walkway. The spillway sill is at El. 68.21 m with the top of stoplogs at El. 68.96 m.

The left abutment is a rockfill dam with concrete core. The rockfill core is the remnant gravity dam with a concrete parapet installed in 2008 that extended the top of core to El. 69.81 m. The overlay rockfill design was to El. 70.85 m but the actual crest is at El. 70.54 m. The left entombed dam section slopes away from the spillway and extends to high ground to seal the left abutment.

To the right of the intake the dam is a post-tensioned concrete gravity structure. The 208 design included a 540 mm high timber parapet and post-tensioning of the entire crest, but the current dam does not include the parapet and four (4) anchors at the far wing of the dam were not installed.

A stage-discharge rating curve for the dam and spillway is generated by cross-referencing time of peak reservoir levels with outflow below the dam. The curve in Figure 2.2 estimates higher stages than previously predicted in large part due to potential backwater effects on discharge below the dam.

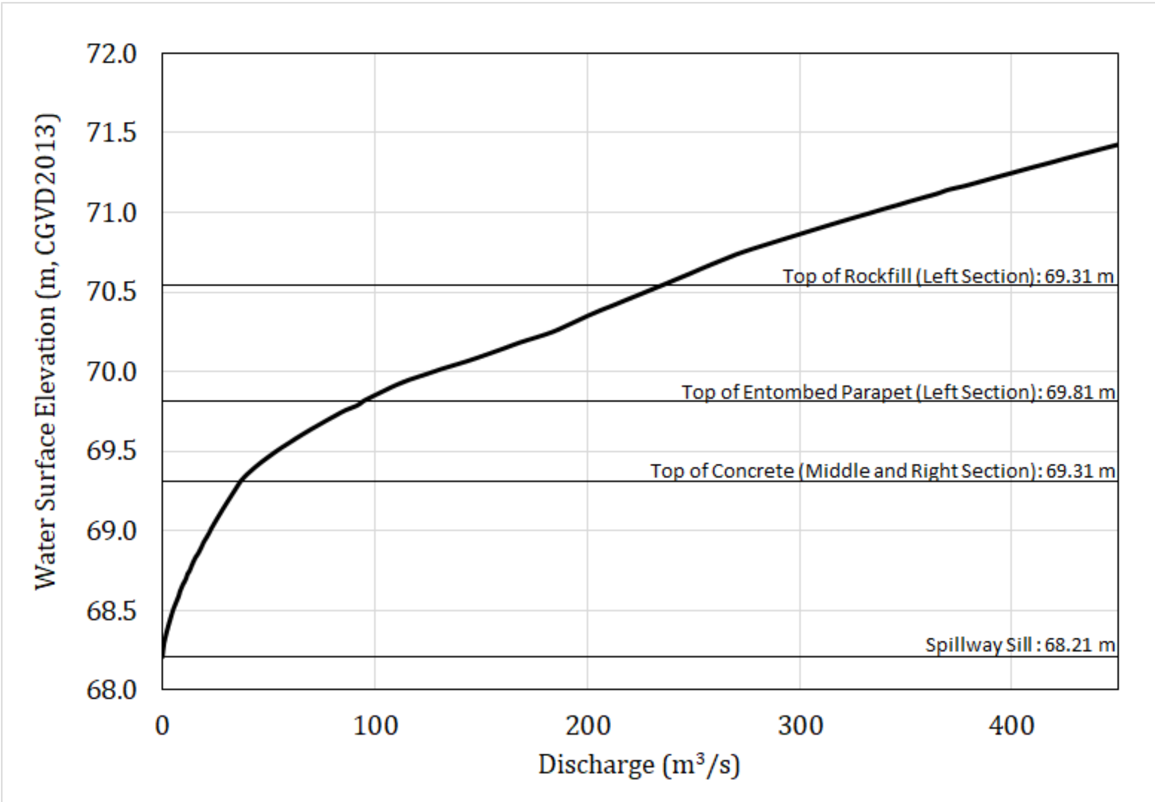

Figure 2.2 Stage-Discharge Curve for Factorydale Dam

3 HYDROTECHNICAL REVIEW

The hydrotechnical review is a summary of the technical memorandum report in Appendix C. Hydrology is developed using a combination of probabilistic and empirical methods. Routing is completed using geometry provided by Berwick Electric Commission, supplemented with field dimensioning and survey at critical infrastructure points.

3.1 WATERSHED

Factorydale Dam controls a catchment area of approximately 92.3 square kilometers with some regulation at South River Lake Dam, Burnt Flowage, and Randall Canal. This watershed area was previously measured as 88.7 km^2 with the change likely attributable to better defined topography with the LiDAR based DEM over previous topographical mapping. There may be additional inflow from Randall Lake via the Randall Canal control structure, but the structure is not well documented and the impact on flood inflows is unknown and was not included in the modelling. Runoff from this inter-basin transfer could introduce an additional 20% of runoff volume into the Factorydale watershed. Likewise, operation of control structures at South River Lake and Burnt Dam flowage are discounted for flood modelling purposes. In general, the watershed configuration is illustrated in Figure 3.1.

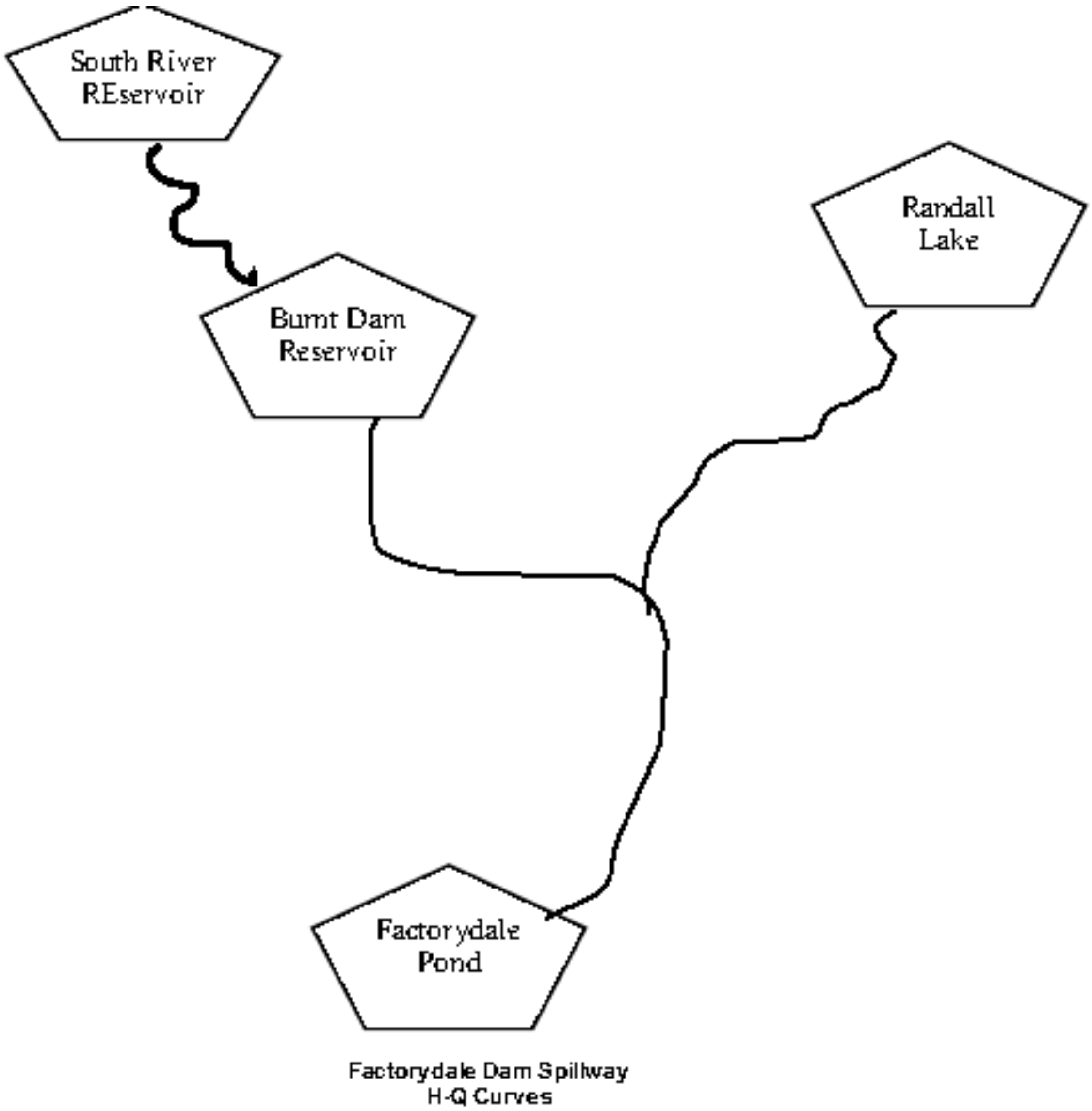

Figure 3.1 Watershed Control Areas

3.2 Inflow Design Flood

Probabilistic and deterministic methods of analysis were completed to estimate flood inflow events up to a probable maximum flood (PMF). Probabilistic analysis used a single-station transfer method and a rainfall-runoff method in a HEC-HMS model to approximate rainfall and inflow with an annual exceedance probability (AEP) up to 1,000 years. The HEC-HMS model was further used to generate an estimate of the probable maximum precipitation (PMP) with the Hershfield method. For all estimation methods, use different techniques to generate hydrographs, including the standard Soil Conservation Service (SCS) unit hydrograph.

3.2.1 Sources of Data

Rainfall data recorded by Environment Canada at CFB Greenwood, Station ID# 8202000, with continuous data from 1953-2024, was selected as the preferred source of data for rainfall. Environment Canada does not publish extreme value analysis for events with AEP's less than 100 years. To generate a suite of data suitable for inflow analysis, Meco develops Intensity-Frequency-Duration (IFD) curves with the same base data using a Gumbel Distribution for durations up to 24-hours and annual exceedance probabilities of 1,000 years. Probable maximum precipitation is estimated by the Hershfield method using the same data set.

The Water Survey of Canada (WSC) report flow data for over one hundred monitoring stations throughout Nova Scotia. The watershed attributes, size and characteristics are compared with ungauged sites for transposition. The WSC publishes the HYDAT service that includes data on average daily flows, and maximum peaks and daily flows that are used in an extreme value analysis of annual maxima for flood estimation. There were four (4) WSC sites near with drainage areas between 50% and 200%. The preferred proxy WSC watershed to develop hydrology Station 01DG003, "Beaverbank River near Kinsac" due to its similar drainage area, relative proximity to Factorydale, and extensive range of data.

3.2.2 Inflow Hydrology

Peak inflow for events with an AEP of 100-, 1,000- year and during the PMP were generated for the different methods. There was variability with the different methods, but all methods provided comparable results. The recommended design peak inflow estimated values for different events are included in Table 3.1. The flood hydrograph is included in Figure 3.2. The recommended IDF with an AEP of 1,000- years is 123 m^3/s which is a negligible change compared with the current IDF of 126 m^3/s.

Table 3.1 Peak Reservoir Level for Various Inflow Events

Flood Event	**Peak Inflow (m^3/s)**	**Peak water level (m)**
Fair Weather	1.15	69.31
Q_{100}	78.9	69.67
$Q_{1,000}$	123	69.93
$Q_{1,000}$ + 1/3[PMF-$Q_{1,000}$]	268	70.57
$Q_{1,000}$ + 2/3[PMF-$Q_{1,000}$]	413	71.10
PMF	557	71.56

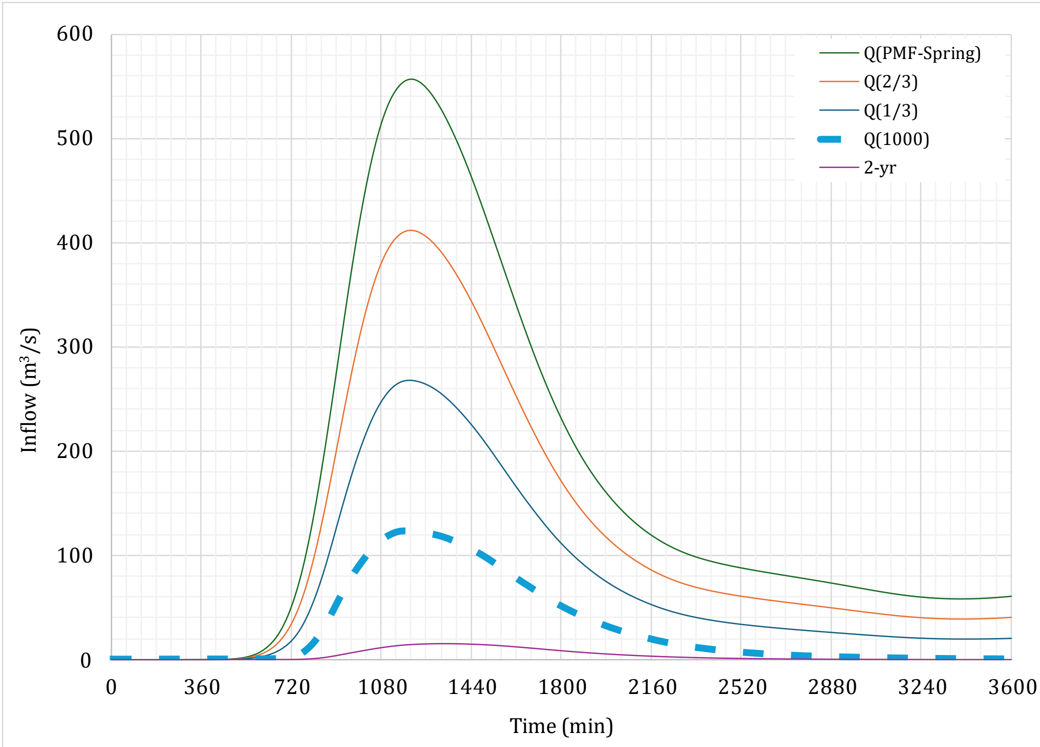

Figure 3.2 Inflow Hydrograph for Factorydale Dam

3.3 Hydraulic Analysis

A hydraulic routing model is developed in HEC-RAS2D to qualitatively assess the precision of current inundation from failure at Factorydale Dam during sunny-day and rainy-day conditions. Public domain, gridded, digital surface data for land use and landforms allowed the model to expand the downstream reach all the way to Annapolis Royal, compared with the information in the EPP that only extends 7 kilometres downstream. The results of the analysis realized at Factorydale Dam are summarized in Tables 3.2.

Table 3.2 Peak Reservoir Level for Various Inflow Events

Flood Event	Peak water level (m)	Peak Inflow (m^3/s)	Peak Outflow Non-Failure (m^3/s)	Peak Outflow Failure (m^3/s)
Fair Weather	69.31	-	-	37.6
Q_{100}	69.67	78.9	74.0	162
$Q_{1,000}$	**69.93**	**123**	**111**	**206**
$Q_{1,000}$ + 1/3[PMF-$Q_{1,000}$]	70.57	267	241	320
$Q_{1,000}$ + 2/3[PMF-$Q_{1,000}$]	71.10	412	259	452
PMF	71.56	557	489	586

4 DAM SAFETY REVIEW

The DSR was completed in general accordance with the CDA Dam Safety Guidelines, 2013. This DSR is intended as a systematic evaluation of the safety of the dam by means of inspection, assessment of performance, review of management practices and review of the original design to ensure compliance with current practices and methods.

4.1 DAM SAFETY MANAGEMENT MODEL

The CDA Guidelines describe a dam safety management system (DSMS) as part of an overall management system that provides a framework for dam safety activities, decisions and supporting policies and procedures. For utilities with a portfolio of dam assets, a standalone DSMS is a credible alternative, but for smaller utilities and municipalities where the dam asset(s) is/are a small part of the overall asset portfolio, a standalone DSMS may present administrative challenges. For the latter, a DSMS that achieves compliance with the CDA Guidelines but is familiar with current management policies has a greater likelihood of achieving a high dam safety standard. Figure 5.1 outlines a possible model for the BEC to manage dam safety.

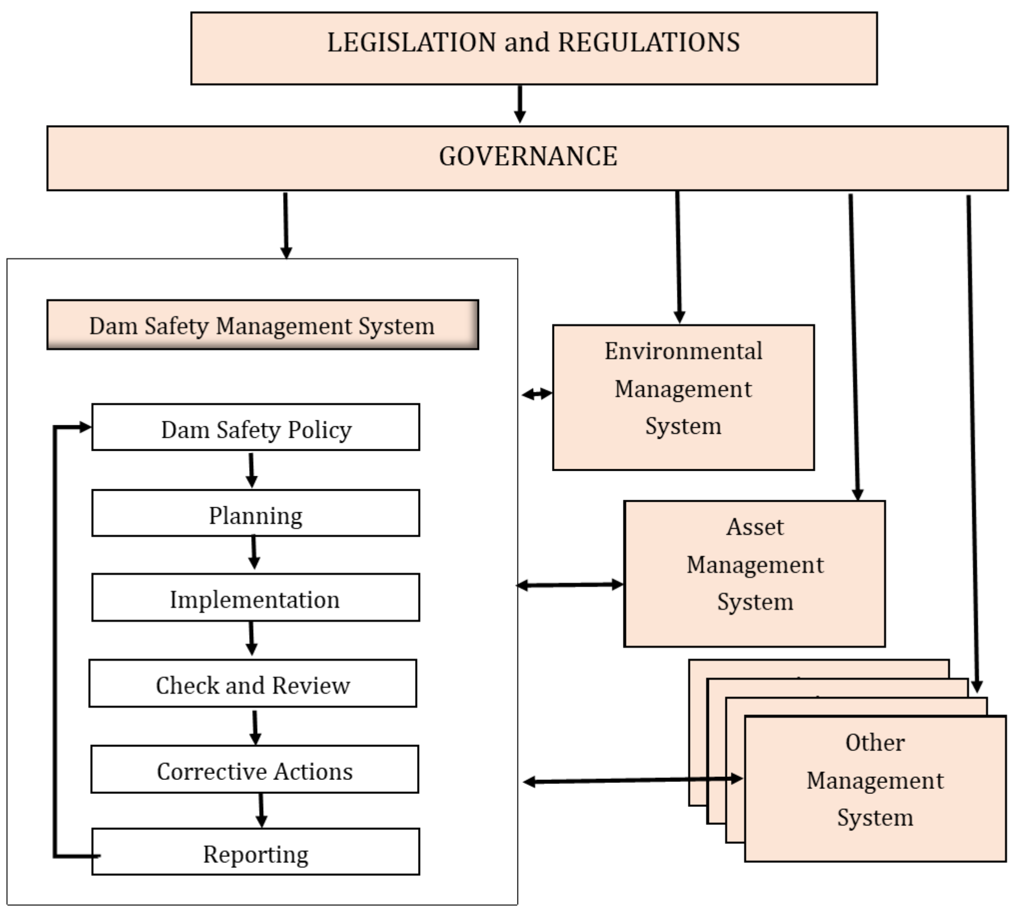

Figure 4.1 Dam Safety Management Model

4.1.1 LEGISLATION AND GOVERNANCE

Nova Scotia does not have specific legislation regulating dam safety, but dam structures are regulated by Nova Scotia Environment and Climate Change (NSECC). In lieu of specific legislation, NSECC typically specifies that dams comply with the Canadian Dam Association (CDA) Dam Safety guidelines. The CDA defines a dam as

> *A barrier which is constructed for the purpose of enabling the storage or diversion of water, water containing any other substance, fluid waste or fluid tailings, providing that such barrier could impound 30,000 m^3 or more and is 2.5 m or more in height. The height is measured vertically to the top of the barrier, as follows:*
>
> *(i) from the natural bed of the stream or watercourse at the downstream toe of the barrier, in the case of a barrier across a stream or watercourse*
> *(ii) from the lowest elevation at the outside limit of the barrier, in the case of a barrier that is not across a stream or watercourse*
>
> *"dam" is defined to include works (appurtenances) incidental to, necessary for, or in connection with, the barrier.*
>
> *For purposes of the CDA guidelines, the definition may be expanded to include "dams" under 2.5 m in height or which can impound less than 30,000 m^3, if the consequences of failure would be unacceptable to the public, such as:*
>
> - *dams with erodible foundations where a breach could lower the reservoir more than 2.5 m.*
> - *dams retaining contaminated substances.*

The reservoir at Factorydale Dam has an estimated storage of 23,500 m^3 at the spillway sill elevation and 29,000 m^3 at the current top of concrete. While the actual stage-storage relationship has not been established with a bathymetric survey, the dam as it is currently constructed does not meet the criteria of a dam, as defined by the CDA Guidelines, and compliance with the Guidelines may be considered as voluntary.

4.1.2 DAM SAFETY MANAGEMENT

The suggested approach for BEC to achieve compliance with the CDA Guidelines is to adopt a DSMS that is familiar with current management systems and policies and procedures, but that is specific to dam safety. The BEC might consider their environmental management system as a suitable proxy because of the emphasis on compliance. A compliance-based management system is a good administrative fit for dam safety that is usually achieved through documentation of (1) Operations, maintenance and surveillance practices, (2) Emergency Management Planning, and (3) Public Safety Plans.

4.1.3 OPERATIONS, MAINTENANCE & SURVEILLANCE

The BEC does not have any policies or procedures grouped in an operations, maintenance and surveillance (OMS) manual at Factorydale Dam. But there are standard practices that operators follow in operating the dam and in care and maintenance of the dam and these practices should be documented for training purposes, but also as a process to create a baseline operating standard with a focus on safety. During the interviews, coupled with observations at the dam, we noted some items that might not be a part of current standard practice that should be considered when cataloguing and documenting practice policy.

- There appears to be limited understanding of the purpose for watershed upstream controls at South River Lake, Burnt Flowage and Randall Canal. The BEC should complete a condition assessment of each control structure, document purpose and efficacy, and establish basic dimensions and operability.
- The current lake level monitoring occurs in conjunction with operating the power plant but is discontinued when the plant is not operating. Lake level monitoring is required at all times for dam safety.
- There is no monitoring for debris in the reservoir. Debris can significantly reduce spillway capacity during high-intensity flash floods and has been the cause of some recent failures. A debris boom to divert debris away from the gated structures will improve dam safety.
- The DSR did not identify defined target action response plan (TARP) levels. TARP levels establish activities such as when to remove stoplogs, when to re-install them, when to close the intake gate, etc. Pre-determined TARP levels are an effective tool for both normal operating and emergency operating environments.

The BEC relies on incident maintenance based on site observations. The system relies on routine inspections by operators to identify emerging issues are promptly addressed to prevent potential dam failures. The system works best if there are independent verification of routine inspections, either through annual, or even bi-annual inspections, by senior management or third parties. As a minimum, a sign-off procedure is required to verify and validate surveillance observations.

4.1.4 Public Safety & Security

BEC does not have a formal public safety plan and has not completed a public safety risk assessment. All dams owned by BEC are in publicly accessible areas and the public can regularly interact with the structures. At different dams, the public is exposed to some or all of the following risks;

1. Drowning
2. Trips and Falls

BEC should consider completing a public safety risk assessment as outlined in the CDA Guidelines to better understand and quantify the public safety risk and develop a public safety plan (PSP) to mitigate risks to the public.

4.1.5 Emergency Preparedness

The BEC Emergency Preparedness Plan (EPP) was issued in 2008 and has been updated periodically since, with the last update in 2011. The format of the EPP does not comply with current best practice for emergency management of dams published by the CDA Guidelines. Items the BEC may consider for achieving compliance with the CDA Guidelines include;

- Update all contact information annually,
- Establish target action response plan (TARP) water levels for normal operation, flood operation, and emergency operations.
- Update the inundation map to better assist emergency responders in the event of a potential dam failure incident.

4.2 DAM CLASSIFICATION

The classification system of the CDA Guidelines is described in Appendix A. The consequence classification defines the standard of care for a dam owner and is evaluated based on the consequences of a dam failure with respect to;

- Life Safety,
- Infrastructure and Economic Impacts, and
- Environmental and Cultural Impacts.

A comprehensive assessment of inundation and consequences of failure at the structures are included in Appendix C. The recommended classifications for all structures are presented in Table 4.1.

Table 4.1 Recommended Classification for Factorydale Dam

Structure Name	Previous Classification	Recommended Classification	Recommended IDF	Recommended EDGM
Factorydale Dam	High	Significant	$Q_{1,000}$ (123 m^3/s)	AEP 1,000 (PGA 0.054g)

The current High consequence classification was established in 2007 in the framework of the 1999 CDA Guidelines with a recommended inflow design flood that has an annual exceedance probability of 1,000 years. The CDA Guidelines were updated in 2007 and again in 2013 along with changed justification for consequence classification. Meco tested the accuracy of the EPP inundation map with a HEC-RAS2D model and a digital elevation model (DEM) using LiDAR. The model routed standards-based flood events specified in the CDA Guidelines, mapped the expected extents of flooding and compared with previous limits.

- Inundation limits from a sunny day failure does not have consequences for sedentary populations. There is a small risk to non-sedentary population from road washouts and bridge failure.
- The total flood inundation for all flood events is less than currently indicated in the EPP,
- Incremental flooding during storm events with a return period of 100 -years and more was negligible, and
- The severity of the incremental flooding in the inundated areas was negligible with depth x velocity (DV) values less that generally accepted lethal limits.

The incremental discharge from a dam failure results in negligible increases in risk to life safety, with the economic risk of highway and bridge washout likely governing the consequence classification of the dam. An evaluation of life safety risk identified an unspecified population at risk (PAR), principally travellers on the roads, that may be at risk of loss of life (lol) during a flood event from a dam failure. There were no significant cultural or environmentally sensitive areas identified in the dam breach area.

The recommended consequence classification for Factorydale Dam is Significant based on economic and life safety risk. The recommended IDF for Factorydale dam is a flood with an annual exceedance probability of 1,000 years. The recommended earthquake design ground motion (EDGM) for Factorydale dam is a seismic event with an annual exceedance probability of 1,000 years.

4.3 Condition Assessment

A dam safety inspection (DSI) was performed on August 8, 2024. Weather conditions were favorable and all exterior parts of the dam were observed. The spillway was operating during the inspection and the conveyance surface of the spillway was not visible. A comprehensive condition assessment report is included in Appendix B. Overall, Factorydale Dam and Spillway was observed to be in Fair condition with no major deterioration or structural deficiencies. The following notable deficiencies were observed.

- There is continual seepage on the right side, but the rate of seepage is not undermining the concrete. Seepage is not monitored.
- Floating debris was observed to impede spillway discharge. A debris management plan that includes a debris boom is required to ensure full operational capacity for the inflow design flood.
- The vegetation should be cleared periodically for better inspection of the abutments and downstream face of the right dam section.
- There is limited Public Safety infrastructure for the dam.

4.4 Design Adequacy

A design adequacy review is completed to a preliminary level, as defined in Appendix A. Design Criteria for the structures are summarized in Table 4.2.

Table 4.2 Design Criteria and Operating Levels

Structure Name	FSL (m)	IDF (m^3/s)	MFL (m)	Crest (m)	EDGM
Factorydale Dam Factorydale Spillway	68.21	123 m^3/s	69.93	70.54	PGA 0.046g

Site observations verified that all aspects of the 2008 rehabilitation design were not completed, namely;

- Four (4) vertical rock anchors included in design for the right concrete gravity dam were not installed.
- Install the timber parapet included in the 2008 remedial design for the right dam abutment.
- The crest of the left abutment was finished 310 mm lower than design.

4.4.1 Freeboard Analysis

Freeboard analysis is presented in detail in Appendix C. Wave characteristics and freeboard values are summarized in Table 4.3. At the left rockfill abutment, wave runup is not expected to overtop the rockfill forming the crest but the still water level during the recommended IDF will likely overtop the top of the concrete core wall and result in concentrated flow downstream of the core wall. The core wall is believed to be constructed on bedrock and can likely survive some overtopping. The maximum flood level overtops the remainder of the concrete dam.

Table 4.3 Freeboard Analysis Results

Parameter	Rockfill Abutment		Concrete Dam	
	Normal	Minimum	Normal	Minimum
Significant Wave, H_s (m)	0.47	0.28	0.47	0.28
Set-up (m)	0.05	0.01	0.03	0.01
Run-Up (m)	0.98	0.55	-	-
Required Freeboard (m)	1.03	0.56	0.50	0.29
Available Freeboard (m)	2.33	-0.12	1.10	-0.62
Adequate (Y/N)	YES	NO[1]	YES	NO

[1] Rockfill slope has adequate freeboard for wave run-up. Left dam section has inadequate available freeboard because the MFL is higher than the entombed parapet by +0.12 metres.

Freeboard compliance in establishing the MFL assumes stoplogs are removed from the spillway prior to the onset of the storm. During the IDF event, with all stoplogs removed, the concrete dam is overtopped by +0.62 meters and the entombed parapet in the left section is overtopped by +0.12 metres. If stoplogs are not removed, the reservoir maximum stage may increase by an additional 210 mm.

4.4.2 Concrete Structures

The concrete dam is a post-tensioned gravity dam with an ogee overflow spillway. Where the dam is less than 1.5 metres high, vertical rockbolt type anchors (Section A) are installed to stabilize the dam. Where the dam is greater than 1.5m high, single-corrosion protection (SCP) anchors (Section B) are installed to stabilize the dam. The analysis concluded that the dam and spillway comply with standards-based criteria for sliding and overturning. At the far end of the right abutment, there is a section of concrete dam that is less than a metre high that is not post-tensioned. This section of the dam is not compliant with the stability criteria of the CDA Guidelines. Otherwise, results from the stability analysis are summarized in Table 4.4 and Table 4.5.

Table 4.4 Results of Sliding Stability Analysis

Load Condition	Required Factor of Safety	Factor of Safety		
		Section A (SCP Anchor)	Section B (Rockbolts)	Spillway
Usual	1.5	1.9	1.8	2.1
Usual (No Ice)	1.5	3.2	22	3.1
Unusual (Flood)	1.3	1.7	4.9	1.8
Extreme (Seismic)	1.1	2.7	16	2.6

Table 4.5 Results of Overturning Stability Analysis

Load Condition	Required Position of Resultant Overturning Force	Requirement Met (Y/N)		
		Section A	Section B	Spillway
Usual	Within middle 1/3 of base	Y	Y	Y
Unusual	75% of base is in compression	Y	Y	Y
Extreme	Within the base	Y	Y	Y

The analysis concluded that the dam and spillway comply with standards-based criteria in the CDA guidelines. Sensitivity analysis determined that the structure may no longer meet criteria if rock anchors experience relaxation of load by about 10%, or to 55% of ultimate load. There were no observations to indicate relaxation is occurring. Sensitivity analysis further determined that the structure may no longer comply with criteria if the condition of the concrete-bedrock interface deteriorates such that a friction angle of less than 36° develops. There were no observations to indicate any of these conditions are present. Areas of the right gravity dam that were not post-tensioned in 2008 remain at risk of deformation. There were no observations to indicate the area is experiencing deformation to-date.

4.4.3 Left Entombed Rockfill Section

The rockfill at the left section of the dam has an upstream and downstream slope of 2H:1V, and there is a small area adjacent to where the dam emerges from the rockfill that is much steeper. Most of the downstream slope falls only 0.3 metres before it hits the access road elevation. Visually, the rockfill appears even on both the upstream and downstream slopes and unchanged from the 2008 construction site photos. Where there is an entombed concrete core and past overtopping events have not displaced the rockfill or exposed the entombed wall, there is not a stability concern.

4.5 Deficiencies and Recommended Mitigation

Three (3) categories of deficiencies were observed: 1) Physical Deficiencies; 2) Design Adequacy Deficiencies; and 3) Management Deficiencies. Each was assigned a priority and an associated urgency as described in the Dam Safety Inspection Report, Appendix B, Section 1.2. Table 4.7 is an excerpt from Appendix B.

Table 4.6 Priority Rating System

Priority	Recommended Timeline (Urgency)
Very High	≤ 1 year
High	> 1 year and ≤ 3 years
Medium	> 3 year and ≤ 5 years
Low	> 5 year and ≤ 10 years
Very Low	> 10 years

4.5.1 Physical Deficiencies

Physical deficiencies identified during the 2024 Dam Safety Inspection (DSI) and their corresponding recommended mitigation are summarized in Table 4.7. There are eleven (11) notable physical deficiencies observed; one (1) Very High priority, three (3) High priority, five (5) medium priority and two (2) Low priority.

Table 4.7 Physical Deficiencies

Index #	Location	Priority	Defect	Recommended Mitigation
1-1	Reservoir	Very High	Floating debris in the reservoir impedes spillway discharge	Develop a debris management plan that includes a debris boom in the reservoir
1-2	Multiple Locations	High	Vegetation growth obscuring abutments and right dam section	Clear vegetation and establish an annual vegetation management plan
1-3	Right Dam Section	High	Overtopping at the right dam section has caused scouring along toe	Monitor for progression. Install timber parapet as part of 2008 remedial design
1-4	Lake, abutments, road access	High	No measures in place to restrict public access on land or water	Complete a Public Safety Around Dams (PSAD) risk assessment and subsequent Public Safety Plan (PSP)
1-5	Catwalk	Medium	Loose nut on catwalk plate, second pier from left (looking downstream)	Secure nut properly to baseplate
1-6	Right Abutment	Medium	Seepage and eroded channel from Right Abutment	Reinforce the abutment with riprap to mitigate erosion
1-7	Right Dam Section	Medium	Erosion and trickling seepage along toe of right dam section	Monitor to establish rate of seepage and verify leakage rate is not escalating
1-8	Penstock Intake	Medium	Toe of dam below penstock intake is spalled and old concrete is exposed beneath overlay. Water pools beneath steps to catwalk	Repair concrete and install drainage to re-direct water away from steps to catwalk
1-9	Downstream Face Next to Penstock Intake	Medium	Mortar peeling off horizontal crack repair	Check date of repair and whether the crack was sealed with pressure injected grout. Monitor for signs of

Index #	Location	Priority	Defect	Recommended Mitigation
				progression or increasing seepage
1-10	Penstock Intake	Low	Concrete surface is weathered and messy	Once penstock repairs are complete, tidy debris of intake superstructure and resurface with new concrete
1-11	Penstock	Low	Circumference of penstock is rusted as it passes beneath Harmony Road	Extend the rust protection coating to include the portion of the penstock which is exposed to weather next to the road. Excavate road if necessary

4.5.2 Design Adequacy Deficiencies

Design adequacy deficiencies identified during the 2024 Dam Safety Review and their corresponding recommended mitigation are summarized in Table 4.8. There are two (2) notable design adequacy deficiencies observed, both Medium Priority.

Table 4.8 Design Adequacy Deficiencies at All Dams

Location	Priority	Defect	Recommended Mitigation
Right Abutment	Medium	Vertical Rock Anchors required to post-tension the concrete were not installed	Consider entombing the concrete with rockfill similar to the left abutment
Right Abutment	Medium	Timber parapet included in 2008 design was not installed	Timber parapet is intended to deflect debris and ice from impacting the penstock
Left Abutment	Low	Top of rock fill at the crest is 310 mm below design crest	Reinstate the left abutment crest to El. 70.85 m.
Watershed	High	Runoff magnitude from Randall Lake watershed into Factorydale watershed is Unknown	Complete an in-field inspection and survey of Randall Canal to understand the hydraulics of the structure and volume of flood flows due to inter-basin transfer

4.5.3 Management Deficiencies

Management deficiencies identified during the 2024 Dam Safety Review and their corresponding recommended mitigation are summarized in Table 4.9. There are four (4) notable design adequacy deficiencies observed; two (2) High priority, and three (3) medium priority.

Table 4.9 Dam Safety Management Deficiencies

Location	Priority	Defect	Recommended Mitigation
Design	High	The Consequence Classification of Factorydale Dam was High	Revise the Consequence Classification of Factorydale Dam to Significant. The recommended IDF is unchanged at an AEP of 1,000 years. The recommended EDGM is unchanged at an AEP of 1,000 years.
Dam Safety Management	High	There is no defined management system for BEC to manage the risk associated with Dams	The BEC senior management and engineering team does not have a defined process for assessing and managing risk identified through risk analysis in the framework of the CDA Guidelines
Dam Safety Management	Medium	BEC has not completed a risk assessment of public safety around dams	BEC should retain an independent opinion of public safety risks at Factorydale Dam in the framework of a Public Safety Around Dams (PSAD) process defined by the CDA
Emergency Management	Medium	The current Inundation Map in the EPP does not accurately reflect the population at risk (PAR) from a dam failure	Perform a dam break model to confirm the PAR, and update the Inundation Map in the EPP
Operations	Medium	BEC does not have any technical documentation to support decision making with respect to operations, maintenance and emergency response	BEC has a limited number of dam structures and should consider developing a single management structure to support decision in operations, maintenance, surveillance and in emergency preparedness
Emergency Management	High	The Emergency Preparedness Plan (EPP) does not contain clear guidance on when to declare an emergency	The CDA Bulletin on Emergency Management suggest establishing target action response plan (TARP) water levels to assist in emergency declaration
Operations	High	Debris was observed in the reservoir accumulating at gates	Install a debris boom to direct debris away from gates and towards the shoreline

5 CONCLUSIONS AND RECOMMENDATIONS

Factorydale dam is regulated by the Nova Scotia Department of Environment and Climate Change (NSECC) through a water withdrawal agreement. With an estimated reservoir storage capacity of less than 30,000 m^3, the dam as it is currently constructed does not meet the criteria of a dam, as defined by the CDA Guidelines, and compliance with the Guidelines is voluntary.

A review of the consequences of dam failure suggests inundation limits established in 2008 likely overestimate life safety risk. Using an updated digital terrain model and 2D floodplain software indicates negligible risk to sedentary populations and limited risk to non-sedentary populations. Based on the analysis, the recommended consequence classification for Factorydale Dam is revised to Significant, from High. The current inundation map in the EPP overestimated population at risk, and the BEC may consider updating the limits to better assist emergency panners. The recommended consequence classification and associated design criteria are as follows:

Dam	Previous Classification	Recommended Classification	Recommended IDF	Recommended EDGM
Factorydale Dam and Spillway	High	Significant	$AEP_{1,000}$ (123 m^3/s)	AEP 1,000 (PGA 0.046g)

Within its governance framework, the goal of dam safety management for the BEC should be compliance within a familiar framework. Instead of establishing dam safety as a separate process, the BEC is encouraged to integrate dam safety management into existing management structures. As part of that effort, document operating and emergency procedures that achieve compliance with the requirements of the CDA Guidelines, such as establishing target action response plan (TARP) water levels for the reservoir, and completing a public safety risk assessment, among others.

Based on current digital terrain maps, the dam controls a catchment area of approximately 92.3 square kilometers, including regulation structures at South River Lake Dam and Burnt Flowage. There is also a structure at Randall Canal that may increase the watershed area through inter-basin transfer. Further action is recommended for the BEC to evaluate upstream controls in the watershed.

Factorydale Dam and Spillway was evaluated based on a visual assessment of their physical condition and a review of design adequacy August 8, 2024. Weather conditions were favorable, and all exterior parts of the dam were observed. The spillway was operating during the inspection and the conveyance surface of the spillway was not visible. Overall, Factorydale Dam and Spillway were observed to be in Fair condition. The deficiencies listed in Section 4.5 should be addressed according to their associated priorities using Table 4.6.

A design adequacy check was completed to a preliminary level. Hydraulic routing of the inflow hydrology estimated the maximum flood level (MFL) during the IDF is elevation 69.93 metres, which is 1.72 metres of surcharge at the spillway and 0.12 m higher than the previous MFL. The MFL is dependent of BEC operating staff removing stoplogs early in the storm event. If stoplogs are not removed, the estimated MFL increases by +0.21 metres.

Concrete dams are less vulnerable to overtopping and have less stringent freeboard requirements. At Factorydale dam, the corewall at the left abutment will overtop by 130 mm but is not expected to undermine and displace the corewall. The remainder of the dam is post-tensuon anchored and is not vulnerable to overtopping. The only vulnerable area is at the far extreme wing of the right abutment, a low-head area that was not post-tensioned, as intended in 2008. Factorydale has overtopped several times in the past and to-date has not exhibited undermining of the dam.

The concrete dam is post-tensioned with rock anchors to achieve stability. Stability analysis verified the dam and spillway comply with standards-based criteria in the CDA guidelines. The extreme right abutment was not post-tensioned in 2008 remain at risk of deformation. There were no observations to indicate the area is experiencing deformation to-date. The 0.550 m high timber parapet wall at the right wing dam was suggested to deflect floating ice and debris towards the spillway and away from impacting the penstock. The wall was not constructed and debris impacts have not occurred.

6 DISCLAIMER

This report has been prepared for Berwick Electric Commission Inc. by Mitchelmore Engineering Company Ltd. (Meco) as per RFP BER2024-002 "Dam Safety Review for Factorydale Dam" issued May 2024 and Meco Proposal 10777 on July 11, 2024, subject to the following limitations, qualifications and disclaimers:

- The report is intended for the exclusive use of Berwick Electric Commission and may be shared with the Nova Scotia Utility and Review Board in their capacity as regulator. It may not be used or relied upon in any manner or for any purpose whatsoever by any other party.
- The report contains the expression of the professional opinion of Meco regarding the Factorydale Dam, 2024 Dam Safety Review. The report shall be read as a whole, with sections or parts hereof read or relied upon in context.
- The conditions of the site may change over time or may have already changed due to natural forces or human intervention, and Meco takes no responsibility for the impact that such changes may have on the accuracy or validity of the observations, conclusions and recommendations set out in this report.
- The report does not extend to any latent defect or other deficiency which could not have been reasonably discoverable or discovered within the scope of the report.
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APPENDIX A

CDA Guidelines Excerpt

TECHNICAL MEMORANDUM– CDA GUIDELINES

Project: 2024 Dam Safety Review
Factorydale Hydroelectric Development

Subject CDA Dam Safety Review Synopsis

Author: Perry Mitchelmore, P.Eng.

Date: December 10, 2024

The purpose of this Technical Memorandum Report (TMR) is to summarize application of the Canadian Dam Association (CDA) *Dam Safety Guidelines* (CDA 2013) to the current project. This TMR is intended to supplement the DSR report and should be read in conjunction with the report, not as a standalone document.

The CDA Guidelines are not prescriptive. They provide a framework to industry that allows for a methodical assessment of a dam's safety. A key component of the CDA Guidelines is a classification system based on consequences of a failure. For higher consequence dams, design criteria are more demanding. The CDA Guidelines include a condition assessment (CA) of the dam based on observation of the physical performance of the structure. There is also a design adequacy (DA) assessment that includes an evaluation of structural capacity against the design criteria established for the consequence classification. Lastly, the CDA Guidelines provide suggestions for management practices, including emergency management, operations management and protection of the public.

Assessment of dam safety is benchmarked against industry standards, suggestions on management practices and systems that promote dam safety and a variety of Bulletins that promote industry suggested practice. The CDA Guidelines provide for risk informed decision making on the safety of dams.

DAM CONSEQUENCE CLASSIFICATION

The CDA Guidelines recommend dam structures be classified based on reasonably foreseeable incremental consequences of failure at the dam. The incremental consequences of failure are those directly resulting from a dam failure, i.e.

Incremental Consequence = Damage (failure condition) – Damage (non-failure condition)

The classification system does not consider the potential for failure or probability of failure of the dam structure itself. In other words, the classification is not influenced by the structural integrity of the dam.

The CDA Guidelines recommended consequences are based on third party impacts when establishing the dam classification, i.e. BEC does not need to consider their institutional losses as part of the consequences. The consequence classification for a dam establishes the duty of care of the owner for areas that are affected by a dam failure.

The CDA Guidelines recommend evaluation of the incremental consequences of dam failure based on a hypothetical dam breach. Consequences of a dam failure are evaluated in terms of:

- Life Safety;
- Environmental and Cultural Impacts;
- Infrastructure and Economic Impacts.

Table 1 CDA Classification System

Dam Classification	Population at Risk	Incremental Losses		
		Loss of Life	Environmental and Cultural Values	Infrastructure and Economical
LOW	None	0	Minimal short-term loss No long term loss	Low economic losses; area contains limited infrastructure or services.
SIGNIFICANT	Temporary Only	Unspecified	No significant loss or deterioration. Loss of marginal. Restoration or compensation in kind highly possible.	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes.
HIGH	Permanent	10 or Fewer	Significant loss or deterioration. Restoration or compensation in kind highly possible.	High economic losses affecting infrastructure, public transportation and commercial facilities.
VERY HIGH	Permanent	100 or Fewer	Significant Loss or deterioration. Restoration or compensation in kind possible but not practical.	Very high economic losses affecting important infrastructure or services (e.g. highway, industrial facilities, storage facilities for dangerous substances).
EXTREME	Permanent	More than 100	Major loss. Restoration or compensation in kind impossible.	Extreme losses affecting critical infrastructure or services (e.g. hospital, major industrial complex, major storage facilities for dangerous substances).

Unspecified, or temporary populations include those using the roadways and bridges, as well as population at work or in commercial establishments.

The CDA Guidelines classification system uses an event based approach for two events; a Fair Weather Failure (FWF), or “Sunny Day” failure event (sudden and unexpected) and Rainy Day Failure (RDF) or “flood induced” failure event, to mimic normal and flood conditions, respectively. The classification for the flood condition is used to determine the inflow design flood (IDF) at a dam. The classification for the normal condition is used to determine the earthquake design requirement (EDGM) for a dam. For the most part, all other design criteria are based on industry practice, and are not consequence related.

The methodology to determine the IDF uses an iterative process that assesses incremental consequences for defined flood events. If the incremental consequences for a given event are less than the assigned consequence level, a lower magnitude event is selected, and the incremental consequences assessed. The process is repeated until the incremental consequences for an event is equivalent to the consequence level. Inflow design flood (IDF) and earthquake design ground motion (EDGM) used in dam analyses for the different dam classification levels are summarized in Table 2.

Table 2 Flood and Earthquake Hazards, Standards-Based Assessments

Dam Classification [1]	**Annual Exceedance Probability - IDF** [2]	**Annual Exceedance Probability - Earthquake** [3]
Low	1/100	1/100
Significant	Between 1/100 and 1/1000 [4]	Between 1/100 and 1/1000
High	1/3 between 1/1000 and PMF [5]	1/2475 [6]
Very High	2/3 between 1/1000 and PMF [5]	1/2 between 1/2475 [6] and 1/10,000 or MCE [5]
Extreme	PMF [5]	1/10,000 or MCE [5]

(Source: CDA Guidelines, Table 6-1B)
1. As defined in Table 2-1, Dam Classification (Section 2.5.4 of the Guidelines).
2. Simple extrapolation of flood statistics beyond 10-3 AEP is not acceptable.
3. Mean values of the estimated range in AEP levels for earthquakes should be used. The earthquake(s) with the AEP as defined in Table 6-1B is then input as the contributory earthquake(s) to develop the earthquake design ground motion (EDGM) parameters as described in Section 6.5 of the Guidelines.
4. Selected on basis of incremental flood analysis, exposure, and consequences of failure.
5. PMF and MCE have no associated AEP.
6. This level has been selected for consistency with seismic design levels given in the national building code of Canada.

Acronyms: PMF, probable maximum flood; AEP, annual exceedance probability; MCE, maximum credible earthquake.

LIFE SAFETY

Consequences of dam failure may include loss of life, injury and general disruption of the lives of the population in the inundated area. Consistent estimates of expected fatalities are difficult to develop, as potential for loss of life depends on several highly uncertain and variable factors. For example, a nighttime failure will likely have greater consequences than a daytime failure, the size of the breach opening and the hydraulic profile of the reach downstream of the dam will impact velocity of turbulence of the flood wave. Consistent with industry practice, life safety quantification is based on the “Population at Risk” (PAR) in the inundated area as the verifiable indicator of potential fatalities. Estimates of PAR should be developed from dam failure modelling.

The PAR from a flood event can be used to estimate expected Loss of Life (LoL) using different methods, primarily empirical, developed by various agencies. These estimates are highly variable and dependent on circumstances such as the number of residences in the inundation area, warning time, severity of the flood, type of failure, etc. The potential loss of life associated with this population at risk is assessed using the RCEM – Reclamation Consequence Estimating Methodology (USBR, 2015).

The RCEM method recommends developing a fatality rate range relative to depth of flow (D) and flow velocity (V) for a range based on whether there is little to no warning (LTNW) time or adequate warning (AW) time to the PAR. Warning times are a function of arrival time of the maximum wave relative to when residents receive notice. Each PAR can be evaluated individually, but the more common practice is to group DV ranges to estimate LoL as the product the of PAR in an area by the average DV for that area. For the FWF, no warning time is anticipated. Flood understanding is a function of the amount of training and public information available with respect to the correct response.

ENVIRONMENTAL AND CULTURAL VALUES

Environmental and cultural consequences are referred to as values and are rarely monetized for ranking purposes. Diligence is required to qualify the extent of loss, both intangible and non-direct losses. In general, losses are evaluated in terms of recovery/reversible expectations, as well as the magnitude and duration of the impact, as described in Table 3.

Cultural values are difficult to monetize because they are based on physical presence or a traditional use. Restoration after disturbance is impractical and cultural values cannot be transposed to a separate location. Cultural values are best assessed specific to a site and without assessing a monetary value.

Intangible social impacts such as damage to irreplaceable historic and cultural features, which cannot be evaluated in economic terms, should be considered on a site-specific basis. Although the exact determination of direct and indirect loss may be complex, the feasibility and practicality of restoration or compensation should be considered.

INFRASTRUCTURE AND ECONOMICS

Infrastructure and economic losses used in determining the consequence of a dam failure include only damage to third-party property, facilities, other utilities and infrastructure. Damage to the dam owner's property is typically excluded from the consequence classification, unless requested by the owner. Economic consequences to the Berwick Electric Commission in the event of a dam failure include cost of litigation and fines, structure replacement/repair costs, lost revenue, and intangibles like loss of reputation and staff effort to respond to a failure incident. These risks can be managed through risk management tools, such as insurance, that are at the disposal of the owner and are not included in the consequence assessment.

The actual limits appropriate to different levels of consequence classification are not explicit in the CDA Guidelines. While there is no standard, the proposed limits identified in Table 4 references the Ontario Guidelines for dam management, adjusted for inflation since publication, that will be used to assess consequences unless advised otherwise.

Table 3 Environmental and Cultural Values Consequence Factors

Dam Class	Environmental and Cultural Values Consequences	
Low	Minimal Short Term Loss. No long term consequences.	
Significant	No significant Loss/Deterioration of - important fish/fauna/wildlife habitat - rare and endangered species - Unique landscapes - sites having significant cultural value	Feasibility of restoration or compensation in kind is high
High	Significant Loss/Deterioration of - important fish/fauna/wildlife habitat - rare and endangered species - Unique landscapes - sites having significant cultural value	Feasibility of restoration or compensation in kind is high
Very High	Significant Loss/Deterioration of - critical fish/fauna/wildlife habitat - rare and endangered species - Unique landscapes - sites having significant cultural value	Feasibility of restoration or compensation in kind is low
Extreme	Major Loss/Deterioration of - critical fish/fauna/wildlife habitat - rare and endangered species - Unique landscapes - sites having significant cultural value	Restoration or compensation in kind is impossible

Table 4 Infrastructure and Economic Consequence Factors

Dam Class	Monetary Consequences
Extreme	N/A
Very High	>$41,500,000
High	Between $4,150,000 and $41,500,000
Significant	Between $415,000 and $4,150,000
Low	<$415,000

CONDITION ASSESSMENT (CA)

A condition assessment for dam safety inspections is completed in accordance with the Guidelines and established industry practice. Dam inspection methods are subjective and will vary depending on the professional conducting the inspection, their level of engineering judgement, and the scope of services.

Engineering inspections typically involve a detailed visual examination of the dam(s) and instrumentation and consist of a systematic walkover of the structures. The inspection team observe conditions and document any observed deficiencies. Where practical, deficiencies are photographed and assigned a condition rating reflective of the hazard. In many situations, deficiencies are assigned a priority rating as well.

CONDITION RATING

Observations and issues identified during the 2022 inspection are presented in the field inspection reports included in appendices along with site photos collected during the field inspection report. The CDA Guidelines do not prescribe a condition rating system. Meco interprets the intentions of the CDA Guidelines and uses a condition ratings system as defined in Table 5.

Table 5 Condition Rating System

Condition	Definition
Good	Will function as intended. Little to no wear, deterioration, or damage. Maintenance is up-to-date.
Fair	Will function as intended. Normal wear or deterioration. Some damage. Maintenance is generally up-to-date, however some minor non-compliance exists.
Poor	May or may not function as intended due to deficiency, abnormal wear, deterioration or damage, or due to lack of maintenance. Component may be missing or not provided.
Unsatisfactory	Will not function as intended due to deficiency, abnormal or severe wear, deterioration or damage, or due to lack of maintenance.

DESIGN ADEQUACY

Design adequacy assessments do not supersede the original design. The purpose of design adequacy assessments is to complete a design review in the present, using current industry practice, and determine if the structure is compliant with current practice. The level of comprehensiveness will depend on the accuracy of available geometry, hydrology and geotechnical conditions, as well as monitoring and surveillance records. Meco defines the following three levels of effort.

Preliminary – Design adequacy completed using geometry provided by the client, material properties are estimated using engineering judgement.

Detailed - Design adequacy completed using geometry provided by the client supplemented with field verification, material properties are estimated using limited in-situ investigation.

Comprehensive - Design adequacy completed using measured geometry, material properties are estimated using detailed investigation and measurement.

The levels of assessment can be used progressively to manage the level of effort appropriate to the risk of dam failure. A preliminary assessment can be used to screen a structure for potential failure modes, to be followed by detailed or comprehensive assessments if anomalies are identified.

EMBANKMENT STRUCTURES

Design adequacy for embankment structures is assessed for two primary modes of failure; external and internal erosion. External erosion is assessed quantitatively through assessment of slope stability. Internal erosion is assessed qualitatively by assessing composition and potential for fines migration. Other failure modes such as external stability during overtopping are only considered if required.

EXTERNAL STABILITY

External erosion through slope instability was assessed using working stress design to determine a factor of safety near the highest section, using geometry from the drawings provided. Three (3) loading conditions are specified for static assessment and seismic assessment, as outlined in Table 6 and Table 7.

Table 6 Factors of Safety for Slope Stability – Static Assessment

Loading Condition	Minimum factor of safety [1]	Slope
End of construction before reservoir filling	1.3	Upstream and downstream
Long term (steady-state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 – 1.3 [2]	Upstream

(Source: CDA Guidelines, Table 6-2)

1. Factor of safety is the factor required to reduce operational shear strength parameters to bring a potential sliding mass into a state of limiting equilibrium (using generally accepted methods of analysis)
2. Higher factors of safety may be required if drawdown occurs relatively frequently during normal operation.

Table 7 Factors of Safety for Slope Stability – Seismic Assessment

Loading Condition	Minimum factor of safety
Pseudo-Static	1.0
Post-earthquake	1.2 – 1.3

The software package GeoStudio is used to model seepage and slope stability of embankment structures using SEEP/W to model seepage and pore water pressures, and SLOPE/W to calculate the factor of safety from slope stability analysis. The critical failure surface is determined using the software search engine with only slope initiation and termination areas defined by the user. In all cases, the Spencer method of analysis was specified and the following criteria for initiation and termination points for failure planes were used.

- Slip surfaces that may pass through either the fill material alone or through the fill and the foundation materials and do not necessarily involve the crest of the dam. These are described as limited failures passing through either the foundation or the embankment slope.
- Slip surfaces that may pass through either the fill material alone or through the fill and the foundation materials and which do include the crest of the dam. These are described as critical failures passing through either the foundation or the embankment slope.
- If warranted, based on analyses 1 and 2 above, slip surfaces may be examined which pass through major zones of the fill and the foundation.

The critical failure mode assumes loss of the crest, with initiation beyond the centerline of the dam. The limited failure mode assumes the crest remains intact and initiation was on the downstream side of the crest centerline. A slope failure was defined as a slope failure that daylights in the slope of the embankment while a foundation failure is defined as a slope failure that daylight below the toe of the dam.

In certain instances, the critical slip surfaces will involve only the outer portion of the upstream or downstream slope. These are low confining pressure slope failures that typically do not put the dam at risk as long as they are managed with regular maintenance programs. In order to assess slope failures that may have an impact on overall safety of the dam, the minimum depth of failure surface was nominally set at half the crest width.

Internal Stability

A traditional assessment of piping and internal erosion potential for an earthfill dam requires conjecture and judgment to assess zoning in the dam, filters (if present), and quality of construction, foundations and performance indicators. The Guidelines do not provide quantitative criteria for protection from internal erosion, i.e., factor of safety limits, but do provide commentary on current industry practice.

Internal erosion leading to piping can occur through the embankment, the foundation, or from the embankment into the foundation (Fell et al., 1999). A qualitative review of the dam's internal geometry and the presence or lack of appropriate design features to protect against internal erosion was completed. Risk factors considered include certain types of geometry, the presence of fractured bedrock, conduits through the dam and certain types of materials.

Concrete Structures

Stability analyses are performed using rigid body mechanics for overturning and sliding. The structures are compared to acceptable criteria from the Guidelines for normal operating conditions, seismic conditions and flood conditions. The Guidelines recommend the load combinations, and the corresponding factor of safety requirements are outlined below in Table 8.

Table 8 Summary of Load Conditions

Loading Combination	Load Combinations	Type of Analysis	
		Peak FS (No Tests)	Residual FS
Usual - Normal Operating Conditions	$D+H+I+(S_h+S_V)+U$	3.0	1.5
Unusual - Flood Conditions	$D+H_f+(S_h+S_V)+U_f$	2.0	1.3
Extreme - Earthquake Conditions	$D+H+(S_h+S_V)+E+U$	1.3	1.1

Loads are described as follows:

Dead Load (D) Force of gravity calculated assuming densities of 23.5 kN/m^3, 18 kN/m^3, and 5 kN/m^3 for concrete, rock fill and timber respectively.

Hydrostatic Load (H) Horizontal component representing hydrostatic pressure of water. Load Condition one (1) (Normal operating conditions) utilize FSL and load condition two (2) (flood conditions) utilize MFL.

Silt Load (S) Silt loadings of 20 kN/m^3 dry weight were included in calculations where applicable.

Ice Load (I) Static ice loads ranging from 150 kN/m to 50 kN/m, are recommended for concrete dams with vertical faces and inclined spillways, respectively (Army Corps of Engineers, 2002).

Uplift Load (U) Is applied to the base of concrete structures.

Earthquake Loads (E) Analysis is carried out using the pseudo-static method with peak ground acceleration (PGA). Site specific PGA values are obtained from the Canadian Geological Survey of Canada.

Rotational behavior was evaluated by determining the location of the resultant of applied forces with respect to the base of the structure. Acceptance criteria for overturning are presented in Table 9.

Table 9 Overturning Criteria

Load Case	Position of resultant force (percentage of base in compression)
Usual Loading	Within the kern (middle third of the base for 100 percent compression)
Unusual Loading	75 % of the base in compression and all other acceptance criteria met.
Extreme Loading	Within the base and all other acceptance criteria must be met.

SEISMIC ANALYSIS

The Guidelines suggest that the EDGM is selected on the basis of the consequences of dam failure, as presented in Table 2. The Guidelines provide for progressively complex analyses for seismic analysis with the pseudo-static method usually applied as an initial screening tool. The pseudo static method is an empirical approach which does not consider the probability of failure. For this reason, the Guidelines state that:

> *…the annual exceedance probability (AEP) values for earthquakes in Table 6-1B (CDA Guidelines) applicable to the high, very high, and extreme classes have to be justified, to demonstrate that they conform to societal norms of acceptable risk. The justification can be provided with the help of failure modes analysis for the dam focused on the particular modes that can contribute to failure initiated by a seismic event.*

Preliminary assessments of structures use a pseudo-static analysis to estimate the minimum factor of safety during seismic events. Seismic risk is evaluated using PGA values determined from models used to derive the standard National Building Code of Canada (median) values for a Class C soil foundation. Mean spectral parameters for 1/100 AEP to 1/2,475 AEP events were obtained by request from the Geological Survey of Canada. Spectral parameters for the 1/5,000 and 1/10,000 AEP events were extrapolated by Meco. As the event return periods increase, the precision of the extrapolated values decreases. Mean spectral values are given in Table 10.

Table 10 Geological Survey of Canada - Mean Spectral Values (NBCC 2015)

Spectral Parameter	Return Period 1/500 year	1/1,000 year	1/2,475 year
PSA 0.2 sec	0.056	0.081	0.125
PSA 0.5 sec	0.042	0.060	0.089
PSA 1.0 sec	0.026	0.038	0.055
PSA 2.0 sec	0.014	0.020	0.030
PGA	0.031	0.046	0.074

In applying the pseudo-static method, the initial screening is to be completed using 100% of PGA. If seismic stability governs safety, the analysis shall be refined using a percentage of the expected PGA for a specified AEP event. The rationale for using factored values is the oscillating nature of the load. For secondary screening purposes analysis of embankment slopes will apply 50% PGA in the horizontal direction and 25% PGA in the vertical direction. For rigid body analysis, 67% PGA was used in the horizontal direction.

FREEBOARD

The freeboard requirement is a function of the type of structure. For concrete dams and other rigid structures that can withstand overtopping without erosion, the freeboard requirement can be based on economic analysis provided there is an insignificant effect on overall stability. For embankment dams, freeboard requirements are more stringent.

Freeboard allowances for the embankment dams were estimated using recommendations contained in the Guidelines. The Guidelines suggest that freeboard at a dam be enough to contain wind-generated wave effects for the following two (2) conditions:

- No overtopping by 95 % of the waves caused by the most critical wind with a frequency of 1,000 year when the reservoir is at its maximum normal elevation.

- No overtopping by 95 % of the waves caused by the most critical wind when the reservoir it at its maximum extreme level during the passage of the IDF.

The recommended wind event for the flood surcharging condition varies depending on the dam classification, from a 100-year wind for a Low consequence dam to a 2-year event for a high and higher consequence dam. Hourly wind data was obtained from Environment Canada's nearby monitoring station and by converting wind pressures in the National Building Code of Canada (2020). Frequency analysis is conducted to derive wind speeds with various annual exceedance probabilities. Calculations of wind setup, wave heights and wave run-up were performed using Coastal Engineering Manual (CEM) and procedures prescribed in the Guidelines.

APPENDIX B

Dam Safety Inspection Report

TECHNICAL MEMORANDUM REPORT

Project: Factorydale Dam

Subject Dam Safety Inspection (DSI)

Author: Oscar Moyles, EIT **Reviewer** Perry Mitchelmore, P.Eng., PMP

Date: December 3, 2024

The current Technical Memorandum Report is for a dam safety inspection (DSI) of the *Factorydale Dam* which was completed August 8 2024 for the Berwick Electric Commission.

1 INTRODUCTION

Factorydale Dam is located at 2545 Harmony Road, within the limits of the Town of Berwick. The dam controls a watershed with 88.7 km^2 that has three (3) upstream controls located at Sandy Lake Dam, Burnt Flowage and Randall Canal. The total storage capacity of the Factorydale reservoir was estimated to be 26,000 m^3 at Full Supply Level (FSL).

Factorydale Dam is a concrete gravity structure with a maximum height of 5.2 metres and an overall length of 115 metres including an 18.15 metre long concrete overflow spillway. The dam was reinforced in 2008 with post-tensioned anchors and rockfill buttressing at the left side, and the spillway crest was upgraded to an ogee shape with a concrete overlay. A steel catwalk was added to facilitate installation and removal of stoplogs. At the time of inspection, the penstock was undergoing repairs administered by Hysovent and its operations could not be observed. The individual components of the dam are shown in Figure 1.1.

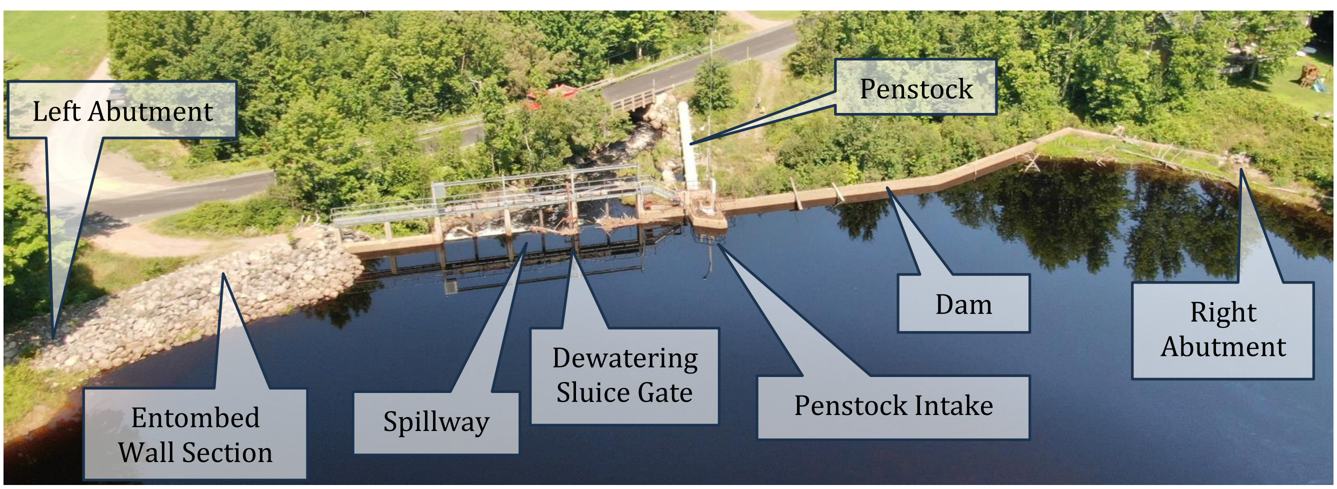


Figure 1.1 Aerial View of Dam Components

1.1 Previous Condition Assessments

The dam has not undergone a condition assessment since its reconstruction in 2008. During the 2008 construction, twenty-seven (27) post tensioned anchors were installed along the length of the dam, the spillway was resurfaced and reshaped with a conventional concrete overlay, the existing catwalk was upgraded to a "T" system with reinforced stoplogs, and a portion of the concrete dam to the left of the spillway was capped with a 500mm concrete cutoff wall, entombed in rockfill for a raised height of 1.55 metres above its previous elevation.

Since 2009, a horizontal crack repair was completed on the downstream face of the dam section between the spillway and the penstock intake. Maintenance of the Penstock turbine is on-going, and the penstock intake was dewatered and not in operation at the time of inspection.

1.2 Deficiency Management System

A dam safety deficiency is: 1) a condition, observation, or event which may threaten the safety of the dam if not addressed in a timely manner; or 2) a noncompliance with dam safety policies, guidelines, standards, regulations, etc. Deficiencies are identified during dam safety reviews, assessments and inspections, OMS activities, incident investigations, audits, EPRP testing, and/or other dam safety program activities.

Deficiencies are ranked based on Meco's priority system. Table 1.1 defines the priority rankings and the recommended timeline to rectify the deficiency.

Table 1.1 Priority Rating System

Priority	Recommended Timeline
Very High	≤ 1 year
High	> 1 year and ≤ 3 years
Medium	> 3 year and ≤ 5 years
Low	> 5 year and ≤ 10 years
Very Low	> 10 years

1.3 Condition Rating

Observations and issues identified during the 2024 inspection are presented in the inspection forms included in Attachment A. Site photos are included in Attachment B. Condition status divisions, under the condition heading in the inspection forms are defined in Table 1.2.

Table 1.2 Condition Rating System for Project Components

Condition Rating	Condition	Definition
G	Good	Will function as intended. Little wear, deterioration, or damage. Maintenance is up-to-date.
F	Fair	Will function as intended. Normal wear or deterioration. Some damage. Maintenance is generally up-to-date, however some minor non-compliance exists.
P	Poor	May or may not function as intended due to deficiency, abnormal wear, deterioration or damage, or due to lack of maintenance.
U	Unsatisfactory	Will not function as intended.

1.4 Site Inspection Overview

The inspection was carried out August 8 2024. Weather conditions were favorable during inspection and there were no missed opportunities for inspections due to the weather. The spillway was operating during the inspection and the inspection team was unable to observe the conveyance surface of the spillway directly. The inspection team members included:

Joey Foley (Berwick Electric Commission) and Company

Anthony Lewis, CET (Meco)

Oscar Moyles, EIT (Meco)

All deficiencies are described and photographed. References left and right are from the perspective of an observer looking downstream at the dam. The dam was not observed to be under duress or at threat of imminent failure.

2 Condition Assessment

Overall, Factorydale Dam is in Fair condition, with minor deficiencies, but should continue to operate for the intended purpose.

The concrete dam and spillway was in Fair condition. There is some spalling on the 2008 concrete overlay at the spillway and exposure of the original concrete not re-surfaced during the 2008 repairs. Observations of scour and flow at the toe and downstream of the dam to the right of the intake section indicate likely overtopping in this area. Seepage is not monitored but was observed at three locations, (1) at the base of the penstock, (22) at the toe of the right dam section, and (3) around the right abutment. There is a rivulet at the right abutment that is attributed to seepage.

No deficiencies were observed at the rockfill-entombed portion of the left dam section and there were no signs of overtopping. Vegetation growth at both abutments, and along the toe of the right dam section obscured observations.

The catwalk and stoplog mechanisms are in good condition, although the stoplog mechanisms were not observed in operation. The stoplogs appear weathered but sturdy. The last known operation of the dewatering sluice gate was in 2008 and-+ current functionality of the gate is unknown but there were no signs of damage to the actuator.

The penstock intake was dewatered for maintenance during the time of inspection. At the downstream base of the penstock intake, there was severe spalling around the base of the power pole and the concrete beneath the overlay was exposed, although the power pole appears structurally stable.

Floating debris was observed to partially inhibit discharge at four of the six stoplog controlled Bays at the spillway. Floating debris was observed to span the stanchions framing the bays at tow locations. Partial or full blockage of spillway operations with debris may result in unplanned overtopping of the dam. The BEC should develop operating procedures for debris management that includes a debris boom in the reservoir.

Twenty (20) of the thirty-one (31) post-tensioned anchors were observed and appeared in good condition. One (1) anchor was buried beneath the rockfill on the left dam section, and six (6) were submerged by flow over the spillway and could not be inspected. Four (4) anchors at the far end of the right dam section appear to have never been installed. A 500mm high timber parapet was intended for installation along the length of the right dam section during the 2008 construction but it was never installed. There was twig and other debris observed in the pockets that indicate splash or wave overtopping. All pockets appear to be draining properly.

The SCADA water level recorder is tied to the powerhouse generator. When the generator was disabled for repairs in February 2024, the water level gauge stopped recording data.

The BEC has not completed a Public Safety Around Dams (PSAD) risk assessment and lack features for public safety. The public can access the site and there is no signage or barriers preventing the public from accessing the dam. An operator observed canoeists upstream of the dam this year (Pers. Comm. Joey Foley, August 8 2024).

2.1 Deficiencies and Recommendations

Deficiencies identified during the 2024 Dam Safety Inspection and their corresponding recommended mitigation for Factorydale Dam are summarized in Table 2.1. Photo # refers the number in the attached Appendix A.

Table 2.1 Factorydale Dam Deficiency List

Photo #	Location	Priority	Defect	Recommended Mitigation
8 19 21	Reservoir	Very High	Floating debris in the reservoir impedes spillway discharge	Develop a debris management plan that includes a debris boom in the reservoir
8 19 21	Multiple Locations	High	Vegetation growth obscuring abutments and right dam section	Clear vegetation and establish an annual vegetation management plan

Photo #	Location	Priority	Defect	Recommended Mitigation
14	Catwalk	Medium	Loose nut on catwalk plate, second pier from left (looking downstream)	Secure nut properly to baseplate
20	Right Dam Section	High	Overtopping at the right dam section has caused scouring along toe	Monitor for progression. Install timber parapet as part of 2008 remedial design
21 22	Right Abutment	Medium	Seepage and eroded channel from Right Abutment	Reinforce the abutment with riprap to mitigate erosion
23 24	Right Dam Section	Medium	Erosion and trickling seepage along toe of right dam section	Monitor to establish the rate of seepage and verify leakage rate is not escalating
29	Penstock	Low	Circumference of penstock is rusted as it passes beneath Harmony Road	Extend the rust protection coating to include the portion of the penstock which is exposed to weather next to the road. Excavate road if necessary.
25	Penstock Intake	Medium	The toe of dam below penstock intake is spalled and old concrete is exposed beneath overlay. Water pools beneath steps to catwalk	Repair concrete and install drainage to re-direct water away from steps to catwalk
16	Penstock Intake	Low	Concrete surface is weathered and messy	Once penstock repairs are complete, tidy debris of intake superstructure and resurface with new concrete
19	Downstream Face Next to Penstock Intake	Medium	Mortar peeling off horizontal crack repair	Check date of repair and whether the crack was sealed with pressure injected grout. Monitor for signs of progression or increasing seepage.
1	Lake, abutments, road access	High	No measures in place to restrict public access on land or water.	Complete a Public Safety Around Dams (PSAD) risk assessment and subsequent Public Safety Plan (PSP)

3 Condition Rating

The condition rating of all structures was generally Fair, but varied from Good to Unsatisfactory, as summarized in Table 3.1.

Table 3.1 2024 DSI Recommended Condition Rating

Component	Condition Rating				
	Not Observed	Good	Fair	Poor	Unsatisfactory
Post-Tensioned Concrete Gravity Dam			✓		
Concrete Ogee Spillway	✓				
Abutments			✓		
Penstock		✓			
Sluice Gate			✓		
Public Safety					✓

4 Conclusion

The dam was not observed to be under duress or at threat of imminent failure. Overall, the dam is in Fair condition. Deficiencies were noted and summarized below.

For Immediate action;

- Floating debris was observed to impede spillway discharge. A debris management plan that may include a debris boom is required to ensure full operational capacity for the inflow design flood.

High Priority action;

- The vegetation should be cleared periodically for better inspection of the abutments and downstream face of the right dam section.
- A Public Safety Around Dams (PSAD) risk assessment is required to form the basis of a Public Safety Plan (PSP) for the dam.
- Install the timber parapet included in the 2008 remedial design for the right dam abutment.

Other notable deficiencies;

The horizontal crack repair on the dam section between the penstock intake and the spillway should be repaired properly with injection grouting techniques if this has not already been done. Where spalls and undermining at the toe are to be repaired, the suggested method is as per the American Concrete Institute (ACI) RAP Bulletin 4, attached in Appendix C. This should be undertaken if there is progression of the undermining.

Monitoring of seepage on the right side of the dam is encouraged. While concrete dams can sustain moderate leakage without risk of failure, increases in the rate of leakage can provide for warning of developing internal erosion that may result in failure of the dam.

ATTACHMENT A

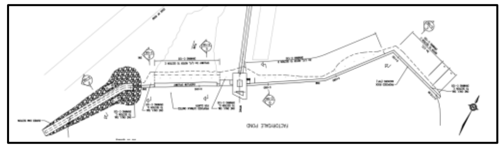
Inspection Reports

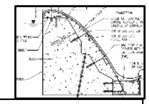
Project:	10777	Date:	08AUG2024
Location:	Aylsford, NS	Weather:	Sunny 20°C
Structure:	Factorydale Dam	Comments:	High Consequence Structure
Inspector:	O. Moyles		

Visual Description of Structure:

Concrete Post-tensioned Gravity Dam & Inline Ogee Spillway with riprap abutments
Dam section left of spillway entombed in riprap
Outlet works include dewatering sluice gate and penstock intake

Schematic Sketch:





Check List: **Comments:**

	Good	Fair	Poor	Comments
Condition of Upstream Slab	Good			Scaling Only. Silt measured x.xx m below top of concrete penstock housing
Cracking or Spalling	✓			
AAR or Paste Generation	✓			
Condition of Crest	Good			Spall beside far right spillway bay and spall for left dam top Difficult to inspect due to spilling water
Cracking	✓			
Spalling or Crumbling		✓		
AAR or paste generation	✓			
Condition of Spillway Shell	Could Not Inspect			Could not inspect due to spilling No stoplog slot for Right bay
Cracking/Spalling				
AAR or Paste Generation				
Condition of Buttress / Walkway	Good			No rust on walkway, one loose nut @ second pier from left hand side (looking downstream)
Cracking	✓			
Struts	✓			
AAR or Paste Generation	✓			
Spalling and Crumbling	✓			
Walkway Supports	✓			
Condition of Foundation	Fair			Signs of overtopping eroding foundation. Operator observed overtopping during July event (77mm July 6th, 71mm July 11th, Greenwood A (Climate ID: 8202000). 6" spall around base of power pole by penstock, old concrete visible. Base of power pole secured by bolt. Undercutting along right dam section. Visible, audible, clear seepage at toe near Anchor #14 and #17.
Abutments	✓			
Erosion		✓		
Public Safety	Poor			No fencing or warning buoys Operators spotted two canoeists on headpond in recent months
Guardrails	✓			
Fencing			✓	
Warning Buoys			✓	

Operations Checklist: **Comments:**

Reservoir Water Levels	Spilling, stoplogs removed
Gates	Good. Logs worn, some rot, but solid
Mechanical & Electrical	Not observed. Penstock chamber drained for repair work.
Channel	Bedrock. Good.
Instrumentation Check	Received SCADA readings back to February 2023. Fields, Units, and datums require clarification. Water level readings lost after Feb 25th, 2024 when Penstock was disabled

Final Comments:

Dam is in good condition generally. Signs of overtopping (likely from July rainfall event). Some erosion of d/s toe and visible seepage, audible trickle, clear. Post tensioned anchors are good condition. One loose nut on walkway, second pier from LHS (looking downstream).

Signed: ____________________ **Date:** 13SEP2024_

ATTACHMENT B

PhotoLog of Site Observations

LIST OF PHOTOS

Photo 1 Aerial View of Factorydale Dam

Photo 2 Spillway Section

Photo 3 Left Dam Section (Partially Entombed)

Photo 4 Right dam Section

Photo 5 Downstream Bridge (Harmony Rd)

Photo 6 Exposed Portion of Left Dam Section

Photo 7 Efflorescence Below Post-Tensioned Anchor at Exposed Portion of Left Dam Section

Photo 8 Entombed Portion of Left Dam Section

Photo 9 Overgrown Left Abutment

Photo 10 Spillway Looking from Left Side

Photo 11 Tailwater of Spillway Looking from Left Side

Photo 12 Stairs to Catwalk and hoist mechanisms

Photo 13 Stoplogs Resting on Catwalk

Photo 14 Actuator for Dewatering Sluice Gate

Photo 15 Loose Nut on Catwalk Plate, Second Pier from Left

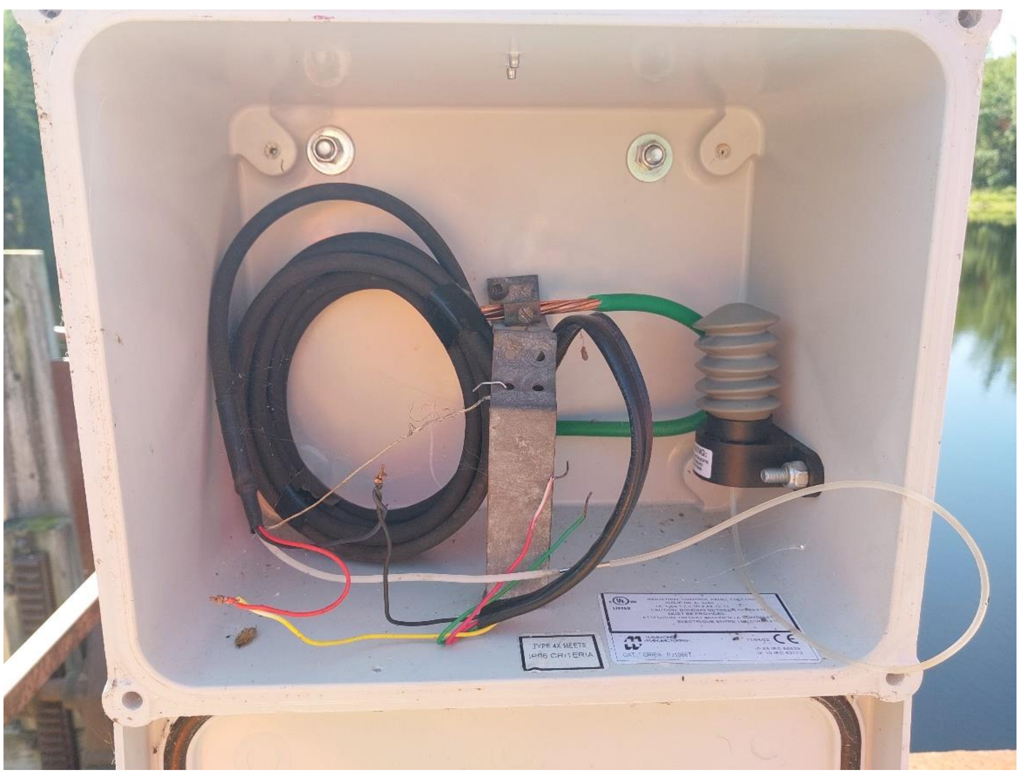


Photo 16 SCADA Water Level Transducer Next to Penstock Intake

Photo 17 Dewatered Penstock Intake Shaft During Maintenance

Photo 18 Looking into Intake Shaft Dewatered for Maintenance

Photo 19 Plastic Seal in Front of Dewatered Intake Shaft

Photo 20 Spalling and Efflorescence Next to Penstock

Photo 21 Right Dam Section Looking from Penstock Intake

Photo 22 Bolts for Stoplog Gains On Right Dam Section

Photo 23 Right Abutment

Photo 24 Runoff from Seepage at Right Abutment

Photo 25 Scour and Trickling water at toe of Right Dam Section

Photo 26 Ponded water at toe of right dam section, next to Penstock intake

Photo 27 Ponded water and scouring around base of power pole beside Penstock Intake

Photo 28 Connection Point for Penstock

Photo 29 Drain Pipe Next To Penstock

Photo 30 Penstock Alignment Looking Downstream from Intake

Photo 31 Penstock Passing Under Harmony Road Showing Rust

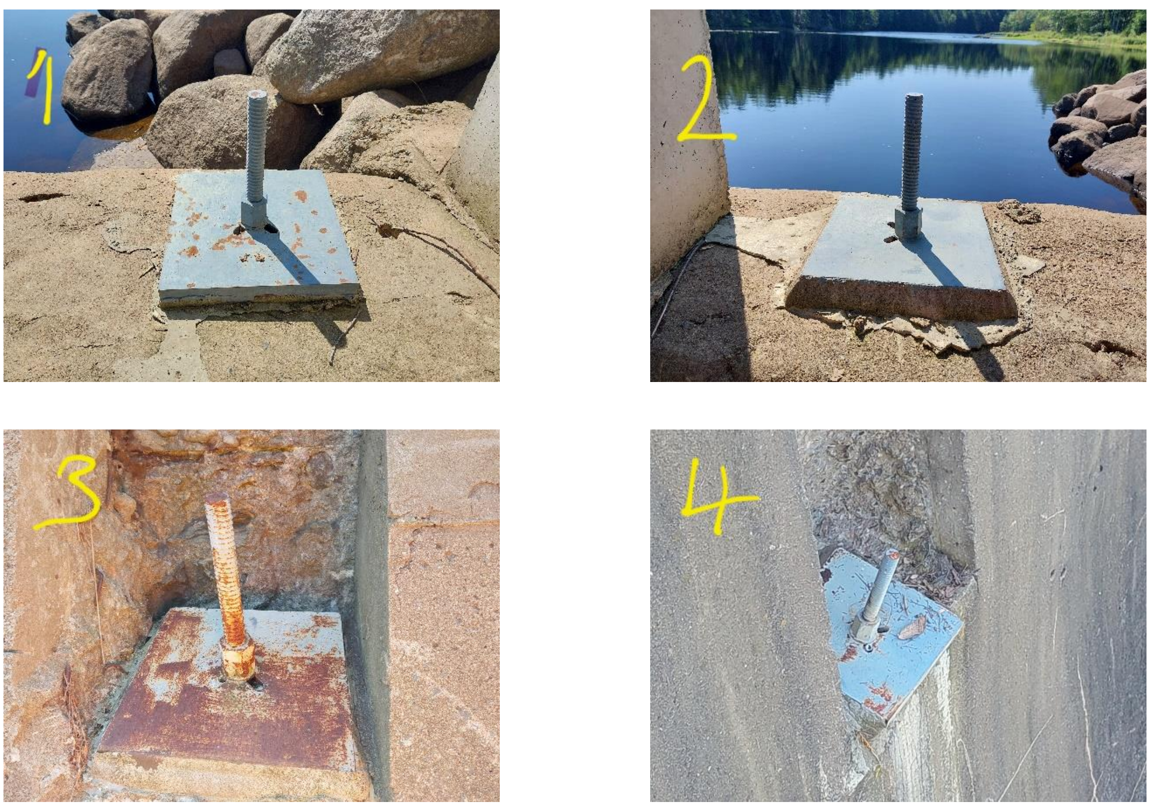


Photo 32 Post Tensioned Anchor #1-4 Starting at Exposed Part of Left Dam Section

Photo 33 Post Tensioned Anchor #5-8 in Downstream Face of Right Dam Section

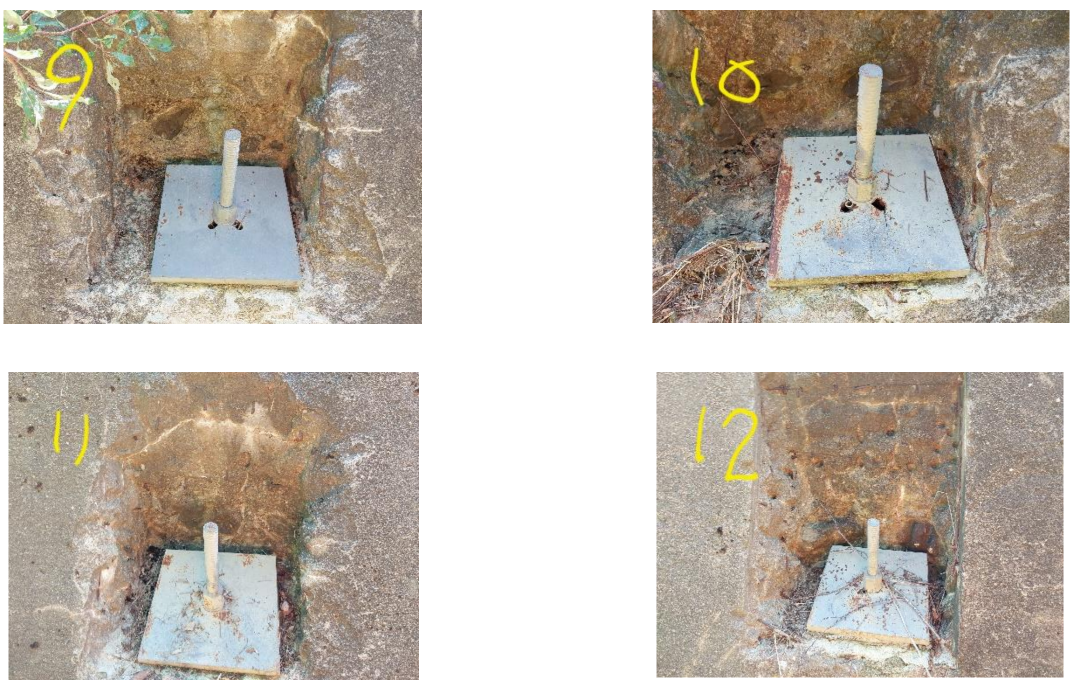


Photo 34 Post Tensioned Anchor #9-12 in Downstream Face of Right Dam Section

Photo 35 Post Tensioned Anchor #13-16 in Downstream Face of Right Dam Section

Photo 36 Post Tensioned Anchor #17-20 Ending at Top of Far Corner of Right Dam Section

APPENDIX C

Hydrotechnical Assessment

TECHNICAL MEMORANDUM REPORT

Project: 2024 DSR & Dam Break Study

Subject: Hydrotechnical Analysis

Author: Oscar Moyles, EIT

Reviewer: Hamid Goharnejad, PhD., P.Eng.

Date: December 18, 2024

The enclosed Technical Memorandum Report provides a comprehensive account of the hydrological and hydrotechnical analysis of the *Factorydale Dam*, completed as part of the *2024 Dam Safety Review Program*.

1 PREVIOUS STUDIES

The current flood and hydrology data for the study area is referenced from the Emergency Preparedness Plan Report (Meco, 2011).

1.1 Emergency Preparedness Plan Report (Meco, 2011)

An Emergency Preparedness Plan (EPP) was prepared in 2007 and updated in 2011 (Meco, 2011). The purpose of the EPP is to detect, prevent and react to an incident at Factorydale Dam. The EPP outlines emergency action instructions, including notification procedures for governmental authorities and affected individuals. It also establishes the roles and responsibilities of key personnel, such as the Incident Commander and Response Mitigation Team, and identifies evacuation zones based on inundation mapping. The plan emphasizes regular training, testing, and annual reviews to maintain readiness.

A Regional Flood Frequency Analysis (RFFA) was used to estimate the Inflow Design Flood (IDF) magnitude, and Simplified Dam Break (SMPDBK) was used to produce a dam breach inundation map with a downstream extent of 7 kilometres, terminating south of Auburn, NS. The dam is classified as a High consequence dam based on potential life safety risk. The current inflow design flood (IDF) is a flood event with an Annual Exceedance Probability (AEP) of 1,000 years ($AEP_{1,000}$) with a peak inflow estimated as 125 m^3/s.

2 SYSTEM DESCRIPTION

Factorydale Dam is located near the town of Berwick in King's County, Nova Scotia. The dam controls a catchment area of approximately 92.3 square kilometers and manages inflow from upstream sources including Burnt Dam Flowage and South River Lake. Randall Canal is at the Southwest edge of the watershed and may collect flow from the Randall Lake watershed.

The main watercourse leaving Randall Lake is Mumford Brook which drains Northwest, away from the Factorydale Dam Watershed. The Randall Lake Watershed area is approximately 18 sq-km. Depending on the draw of flow from Randall Lake and the nature of the Randall Canal – which are unknown – the peak runoff to the Factorydale Dam head pond could increase by 20%. This report does not factor in runoff from the Randall Lake Watershed.

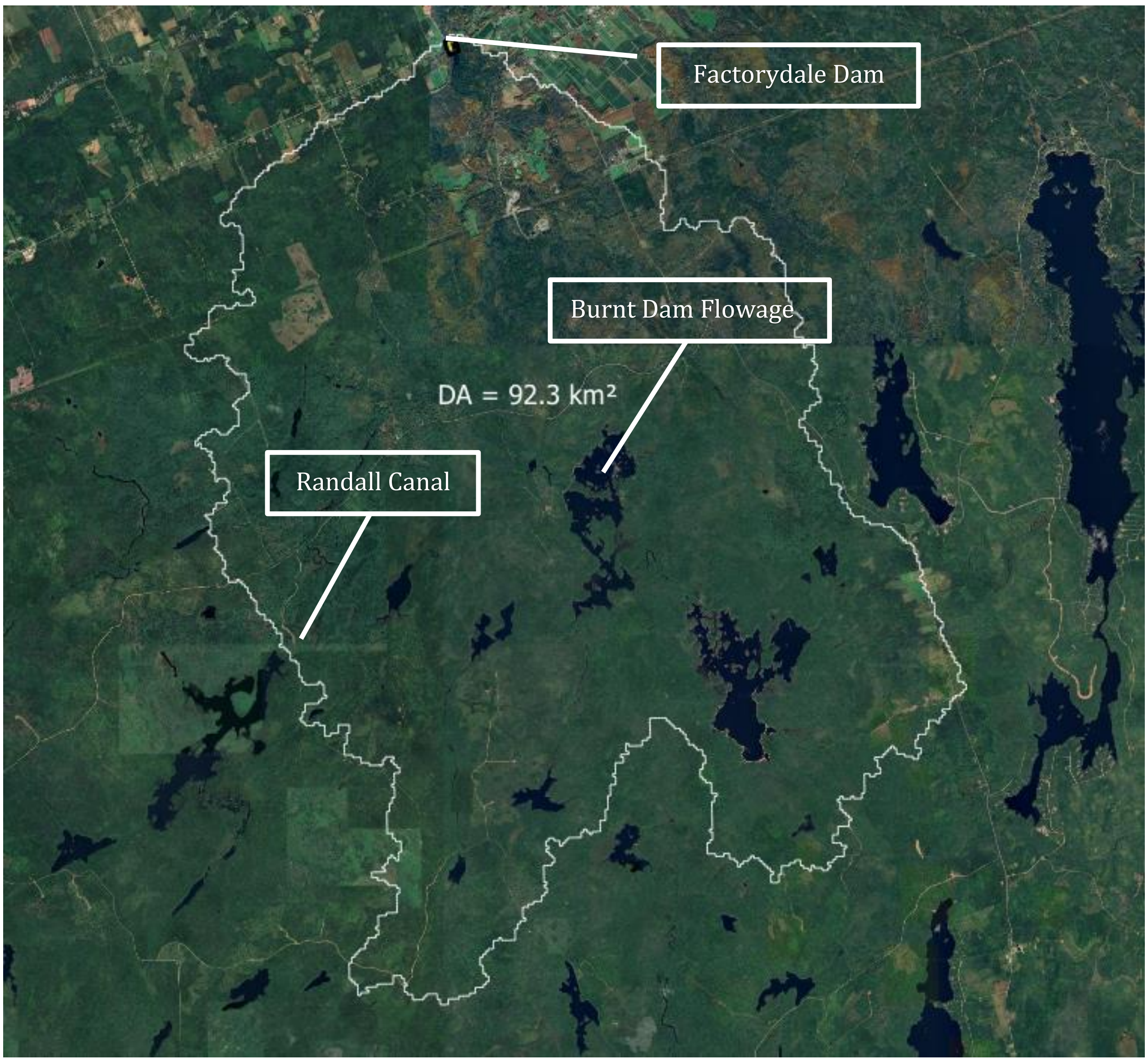


Figure 2.1 Factorydale Watershed

2.1 Elevation Datum

All structure elevations are in Canadian Geodetic Vertical Datum 2013 (CGVD2013). Some values taken from previous reports, such as dam structure elevations, were adjusted from CGVD1928 to CGVD2013. The Digital Elevation Model (DEM) is generated based on public domain LiDAR from the Government of Nova Scotia GeoNova Database (geonova.novascotia.ca). Map coordinates for the model were taken from the NAD83 MTM Zone 5 projection.

2.2 Factorydale Dam

Factorydale Reservoir has a surface area of approximately three (3) hectares. We are unaware of any previous bathymetric surveys of Factorydale reservoir. An estimate of the stage-storage relationship from 2008 indicated live storage of 27,530 m^3 to the top of the stoplogs, el. 68.96 meters above sea level (i.e., stoplogs are 750 mm high). Inflow consists of local uncontrolled flows from undeveloped terrain. The reservoir stage-storage relationship was adapted from a previously defined rating curve and is presented in Table 2.1.

Table 2.1 Stage Storage Curve for Factorydale Dam

Key Elevations	Water Surface Elevation (m, CGVD2013)	Storage (m^3)
Bottom of Dam	63.71	0
	64.00	1,016
	65.00	6,361
	66.00	11,707
	67.00	17,053
	68.00	22,398
Spillway Sill	68.21	23,521
	69.00	27,744
Top of Stoplogs (+0.75 m)	68.96	27,530
Top of Concrete Dam	69.31	29,401
Top of Entombed Parapet	69.81	32,074
Top of Left Section Rockfill	70.54	35,976
	71.00	37,997

2.2.1 Factorydale Dam /Spillway

The Factorydale Dam is a post-tensioned concrete gravity dam measuring 113 meters in length and reaching a maximum height of 5.2 meters, although much of the structure is around 2 meters tall. It features an 18.7-meter-long spillway section, equipped with wooden stop logs to regulate the water level by adjusting the forebay height. Water discharge is managed through the spillway system, though the spillway's capacity has been frequently inadequate during flood events, leading to overtopping.

A discharge rating curve for the spillway and dam top is presented in Figure 2.2.

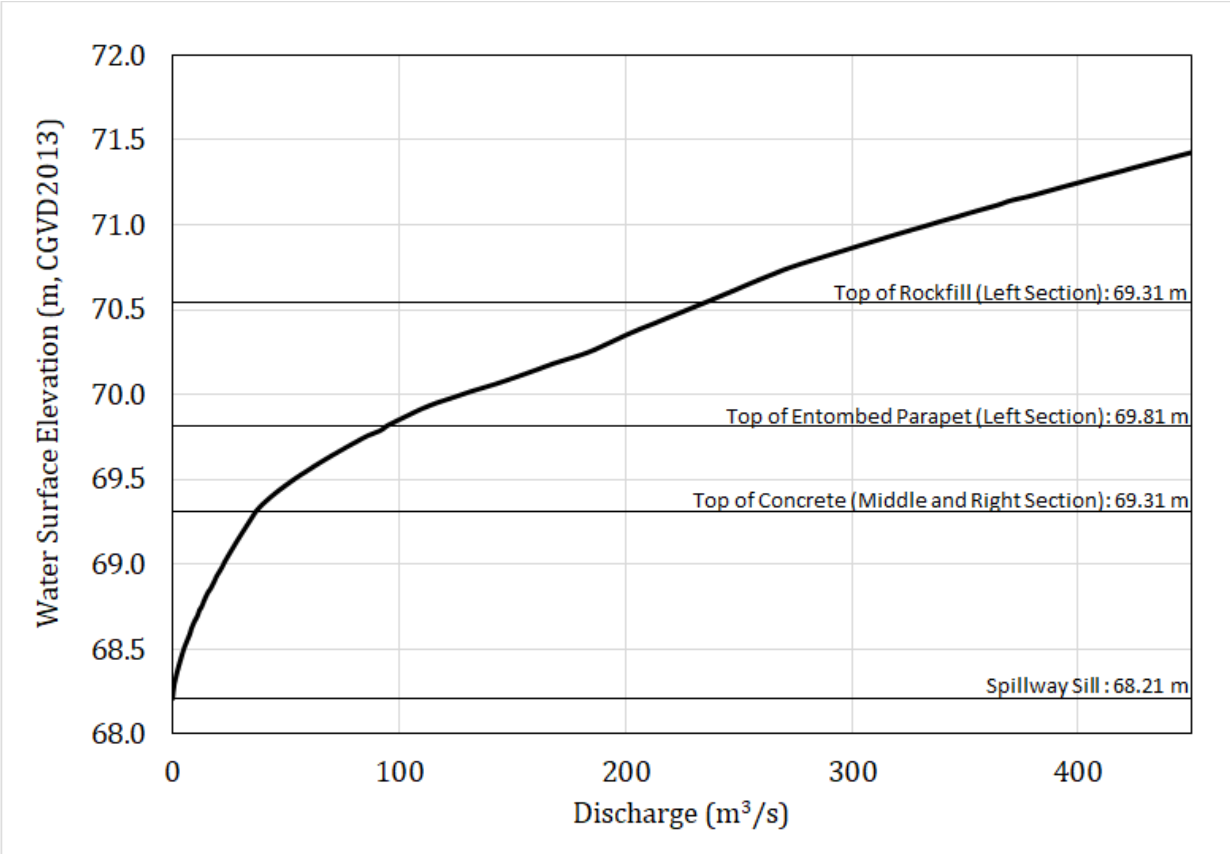


Figure 2.2 Stage-Discharge Curve for Factorydale Dam

3 HYDROLOGICAL ANALYSIS

3.1 Inflow Hydrology

A HEC-HMS routine was developed to model rainfall concentration and distribution in each sub-watershed and the overall watershed. The HEC-HMS model uses data from Greenwood Airport Station (Climate ID: 8202000) to estimate the 100- and 1,000-year for different durations using a Gumber Distribution. Environment Canada publish extreme value precipitation for frequencies as low as 100-years but does not provide guidance on frequences with 1,000 years as required for dam design. The extreme precipitation values developed by Meco are comparable to the values published by Environment Canada. Table 3.1 provides the Intensity-Frequency-Duration (IFD) data from the Greenwood Airport Climate Station in Nova Scotia (Climate ID: 8202000) with data length from 1953-2024.

The HEC-HMS "frequency storm" model was employed as the representative storm for system analysis. Several storm durations were considered, and based on the watershed's characteristics and effective area, the 24-hour storm event was the governing factor in this watershed, as it generated the highest inflow rates and reservoir water levels.

Table 3.1 Intensity-Frequency-Duration (IFD) Data for Greenwood A. Station

"Return Period, T (years)	5min	10min	15min	30min	1h	2h	6h	12h	24h
2	5.76	8.26	10.41	14.48	19.86	26.24	39.49	50.44	57.96
5	7.67	10.99	14.12	20.12	28.81	38.06	52.60	66.08	75.50
10	8.93	12.79	16.57	23.86	34.73	45.88	61.28	76.44	87.11
25	10.53	15.07	19.67	28.58	42.22	55.77	72.24	89.52	101.77
50	11.72	16.77	21.97	32.08	47.77	63.10	80.37	99.22	112.65
100	12.89	18.45	24.25	35.55	53.28	70.38	88.45	108.86	123.45
1000	16.78	24.00	31.79	47.04	71.49	94.44	115.13	140.69	159.14

3.2 Inflow Hydrograph

For this hydrotechnical analysis, diversion flow from Randall Lake was excluded. Any diversion flow from Randall Lake would serve to increase the total runoff volume at Factorydale Dam.

Deterministic analysis was completed to estimate runoff flood hydrographs for events with an annual exceedance probability (AEP) of 100, 1,000 years, a Probable Maximum Flood (PMF), the

$Q_{1/3} = Q_{1000} + \frac{1}{3} \times (Q_{PMF} - Q_{1000})$ and

$Q_{2/3} = Q_{1000} + \frac{2}{3} \times (Q_{PMF} - Q_{1000})$.

The first approach was to use the River Basin Transfer (RBT) method which makes use of a gauged "proxy" basin that is similar enough in size and has similar enough hydrologic properties so that its recorded peak runoff values may be transposed to the ungauged watershed project site. This is also called the "single station transfer" method. The second approach was to develop a rainfall-runoff model in HEC-HMS using the 24-hour HEC-HMS frequency design storm and the standard Soil Conservation Service (SCS) unit hydrograph.

3.2.1 Single Station Transfer

The Single Station Transfer can be used to estimate natural flood flows on un-gauged watersheds using a gauged-watershed located close-by and with similar watershed characteristics.

The Water Survey of Canada (WSC) has 142 flow monitoring stations installed throughout Nova Scotia. The WSC records data including average daily flows, and maximum peaks and daily flows. WSC sites near with drainage areas between 50% and 200% of the watershed were compiled in Table 3.2.

The preferred proxy WSC watershed to develop hydrology for Factorydale is WSC Station 01DG003, "Beaverbank River near Kinsac" due to its similar drainage area, relative proximity to Factorydale, and extensive range of data. Transposed peak flood values for the instantaneous flood with annual exceedance probabilities of 100- and 1,000-years are presented in Table 3.3.

Table 3.2 Representative WSC Proxy Basins for Factorydale Dam

Station ID	Station Name	Coordinate	Drainage Area (Km^2)	Years Available
01DG003	**Beaverbank River Near Kinsac**	**44° 51' 03.99" N 63° 39' 50.01" W**	**96.9**	**98 (1921 - 2018)**
01DD002	Cornwallis River at Cambridge station	45° 03' 53.21" N 64° 38' 07.19" W	90.8	18 (2000-2018)
01DL001	Kelly River (Mill Creek) at Eight Mile Road	45° 35' 12.52" N 64° 27' 02.49" W	63.2	49 (1969-2018)
01EJ001	Sackville River at Bedford	44° 43' 53.51" N 63° 39' 37.58" W	146.0	102 (1916 - 2018)

3.2.2 Rainfall-Runoff Analysis

Rainfall-Runoff Analysis was conducted using the HEC-HMS model, incorporating data from Greenwood Airport Climate Station, to estimate runoff for different return periods within the Factorydale watershed. The model accounted for runoff loss rates, baseflow, and the transformation of precipitation into runoff, using a 15-minute time increment for computations. The results are presented in Table 3.3, which compares the peak runoff values obtained using the River-Basin Transfer Method (RBTM), HEC-HMS, and a previous hydrotechnical assessment from 2011.

For the 2-year return period, the RBTM estimates a peak runoff of 29.6 m^3/s, while the HEC-HMS shows a significantly lower value of 15.63 m^3/s. The lack of data from the 2011 assessment makes further comparison challenging for this return period. However, for the 100-year return period, the RBTM yields 75.6 m^3/s, slightly lower than the HEC-HMS result of 78.9 m^3/s, suggesting that the HEC-HMS may predict more conservative peak flows. For the 1,000-year return period, the RBTM estimate of 101 m^3/s is lower than both the HEC-HMS value of 123 m^3/s and the 2011 assessment's 125 m^3/s. This comparison indicates that the HEC-HMS model predicts higher peak runoff for extreme events, aligning more closely with the previous study for the 1,000-year event, reinforcing the need for robust flood management strategies for larger return periods.

Table 3.3 Peak Runoff for RBTM, HEC-HMS and Previous Hydrotechnical Assessment for Factorydale

Return Period (years)	River-Basin Transfer (m^3/s)	HEC-HMS (m^3/s)	2011 Hyd. Assessment (m^3/s) [1]
2	29.6	15.63	---
100	75.6	78.9	---
1,000	101	123	125

[1] Meco, 2011

3.3 Probable Maximum Flood

The Probable Maximum Flood (PMF) considers the most severe ‘reasonably possible’ combination of the following events at the watershed upstream of the structure:

- Extreme Rain Event;
- Snow Accumulation combined with a Melt Rate;
- Antecedent Basin Conditions (e.g., soil moisture, lake and river levels); and
- Pre-Storm Set-Up.

The CDA Guidelines recommend probable maximum precipitation (PMP) and probable maximum snow accumulation (PMSA) models be used to estimate the PMF for the following scenarios:

- **Summer/Fall PMP** - Summer/fall flood resulting from the AEP_{100} rainfall pre- storm followed by the summer/fall PMP, with no snow on the ground;
- **Winter/Spring PMP** - Winter/spring flood resulting from the winter/spring PMP combined with the AEP_{100} snow accumulation and the AEP_{100} temperature sequence; and
- **PMSA** - Winter/spring flood resulting from the AEP_{100} rainfall combined with the PMSA and the AEP_{100} temperature sequence.

Meco used the HEC-HMS model to distribute probable maximum precipitation, utilizing values obtained through the Hershfield Method and 70 years of daily precipitation data from the Greenwood A Station in Nova Scotia. Snowmelt and temperature sequence data were sourced from for this station from the "Mainland Nova Scotia Flood Study" (Hatch, 2010). This study will be referred to hereafter as the “PMP Study”. The HEC-HMS model accounted for spring snowmelt, runoff loss rates, baseflow, and the transformation of precipitation into runoff. Probable Maximum Precipitation ordinates were distributed as a “frequency storm” similar to what was described in Section 3.1.

3.3.1 Precipitation and Snow Accumulation Extreme Events

Using information and guidance in the PMP Study (Hatch, 2010) the following parameters were established, as summarized in Table 3.4:

- Annual, Winter/Spring, and Summer/Fall Probable Maximum Precipitation (PMP);
- Probable Maximum Snow Accumulation (PMSA) and the AEP_{100} snow accumulation;
- The AEP_{100} spring and summer rainstorm; and
- The AEP_{100} mean and maximum daily temperature.

Table 3.4 Meteorological Extreme Events for the Factorydale Dam Watershed

24-hour PMP			PMSA	AEP1:100 Snowpack		AEP1:100 1-d Rain		AEP1:100 1-Day Max Temp	
Annual	**Winter/Spring**	**Summer/Fall**	**SWE (mm)**	**Ave. Density (%)**	**SWE (mm)**	**Spring (mm)**	**Summer (mm)**	**Max Daily (°C)**	**Mean Daily (°C)**
409	307	415	710	24.5	298	66.7	130.9	26.4	17.7

3.3.2 Temperature Sequence

A historic temperature sequence is used to estimate a snowmelt based PMF. The more severe of the AEP_{100} Snowfall plus the Winter/spring PMP, and the AEP_{100} winter/spring rainfall plus the PMSA, will govern. A snowmelt temperature was selected using the 1-Day AEP_{100} daily maximum and daily mean temperatures, which are 26.4°C and 17.7°C, respectively. These temperatures were derived from the Greenwood A. Meteorologic Station (ID: 8202000). The AEP_{100} temperature sequence used is presented in Table 3.5 and Table 3.6.

Table 3.5 Max-Daily-Max Temperature Sequence (Deg C)

Period	Greenwood A. Station
1 Day	26.4
2 Day	48.3
3 Day	68.9
4 Day	86.3
5 Day	105.9
10 Day	170.0
15 Day	234.7

Table 3.6 Max-Daily-Mean Temperature Sequence (Deg C)

Period	Greenwood A. Station
1 Day	17.7
2 Day	31.3
3 Day	43.8
4 Day	55.5
5 Day	65.4
10 Day	99.3
15 Day	134.3

For snowmelt modeling, the cumulative temperatures were converted into daily temperatures and each day was split into four 6-hour time steps with the following temperatures assigned as:

T_{mean}	Daily Mean Temperature	06:00 and 18:00
T_{max}	Daily Maximum Temperature	12:00 hours
$T_{min} = 2 \times T_{mean} - T_{max}$	Daily Minimum Temperature	24:00 hours

3.3.3 Snowmelt

The season was considered for two (2) PMF events, the first resulting from the combined effect of PMP plus AEP_{100} snow accumulation (SWE) with AEP_{100} melt temperatures, and the second from the AEP_{100} spring rainfall plus PMSA with AEP_{100} melt temperatures. A lapse rate of 0.6 °C/100 m, as recommended for the region, was used to reduce air temperatures of the design sequence to the mid-point of each elevation zone (Zhao et. al., 2022). The snowmelt runoff was modeled with the temperature index technique using the following default parameters and assumptions:

P_x	1.1 °C	Temperature at snow/rain threshold
Base temp	1.1 °C	Base temp above which snowmelt occurs
Melt rate	3.7 mm/deg-day	
Rain rate limit	0.5 mm/day	(Typ): Discriminates between dry and wet melt
ATI melt rate coefficient	0.98	(manual)
Cold limit	20 mm/day	(Typical value)
Cold rate coefficient	0.9	
Water capacity	5%	3 to 5 % as in the manual
Ground melt	0	(Assumed)

3.3.4 Loss Rates

A loss rate model is specified in the rainfall/snowmelt-runoff analysis for a natural watershed where loss rates determine net precipitation available for direct surface runoff. Precipitation losses are attributed to infiltration, depression storage or interception by vegetation. The curve number method is selected in HEC-HMS for estimating PMP loss rates. The method relates drainage characteristics of soil groups to a Curve Number (CN), which varies with land use description. The watershed was assumed to be comprised of soils and a curve number between 70 and 80. The CN is representative of the normal antecedent moisture condition and moderate infiltration rate.

3.3.5 SCS Unit Hydrograph

The unit hydrograph shape controls the portion of rainfall volume that will occur *before* the peak of the inflow hydrograph. The SCS has a unit hydrograph with 37.5% of unit runoff occurring before the peak flow as their *standard* unit hydrograph. This percentage is reflected in the *peak rate factor* (PRF). A catchment lag time was specified for the unit hydrograph transformation, specified as the time from the centroid of the rainstorm to the peak of the runoff hydrograph. Catchment lag, or lag time, was calculated based off geometric subbasin values using an applicable equation. Lag time can be approximated to time of concentration with the following relationship:

$$t_l = t_c \times 0.6 \qquad \text{Equation 3.1}$$

Where, t_l is catchment lag, and t_c is time of concentration. The lag time used for the Factorydale Dam Watershed is 4.6 hours.

3.4 Flood Hydrographs

Flood hydrographs were developed in HEC-HMS using data from the PMP Study (Hatch, 2010) for winter/spring PMP, summer PMP and the PMSA event. The Spring and Summer/Fall hydrographs are presented in Figure 3.1. The Winter/Spring PMP + AEP1:100 snowmelt produces the highest peak runoff and most volume and is the governing PMP event for the system. The inflow hydrographs for the PMF, $Q_{2/3}$, $Q_{1/3}$, and 1000-year return period are presented in Figure 3.2. The hydrograph for the $Q_{2/3}$, and $Q_{1/3}$ events were scaled between the $Q_{1,000}$ and Rain-On-Snow PMF event. Peak inflows from the rainfall-runoff HEC-HMS model are summarized in Table 3.7.

Table 3.7 Non-Routed Peak Inflow (m^3/s) at Factorydale Dam

Q_2	Q_{100}	$Q_{1,000}$	$Q_{1/3}$	$Q_{2/3}$	PMF
15.6	78.9	123	267	412	557

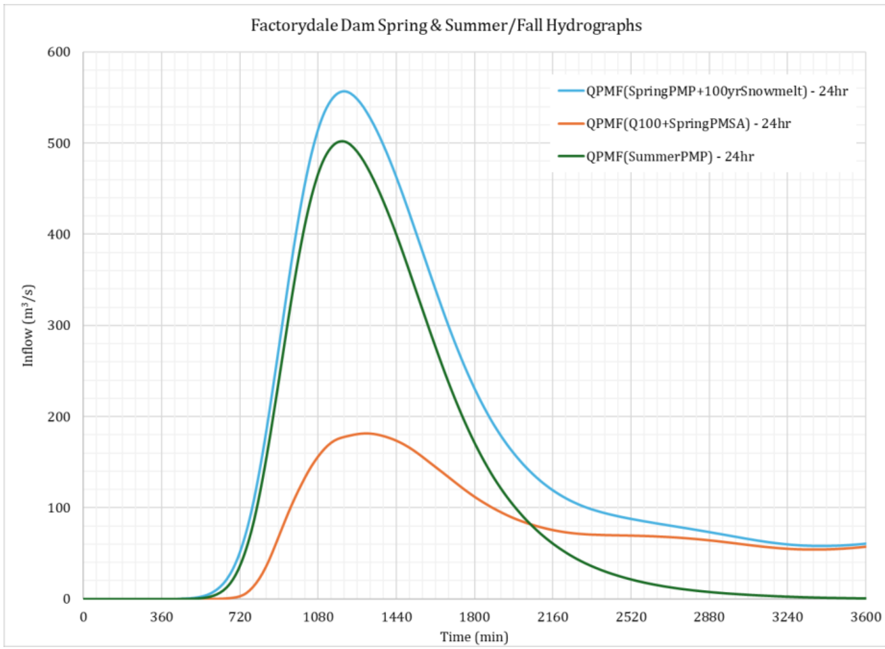


Figure 3.1 Spring and Summer/Fall PMP Inflow Hydrographs at Factorydale Dam

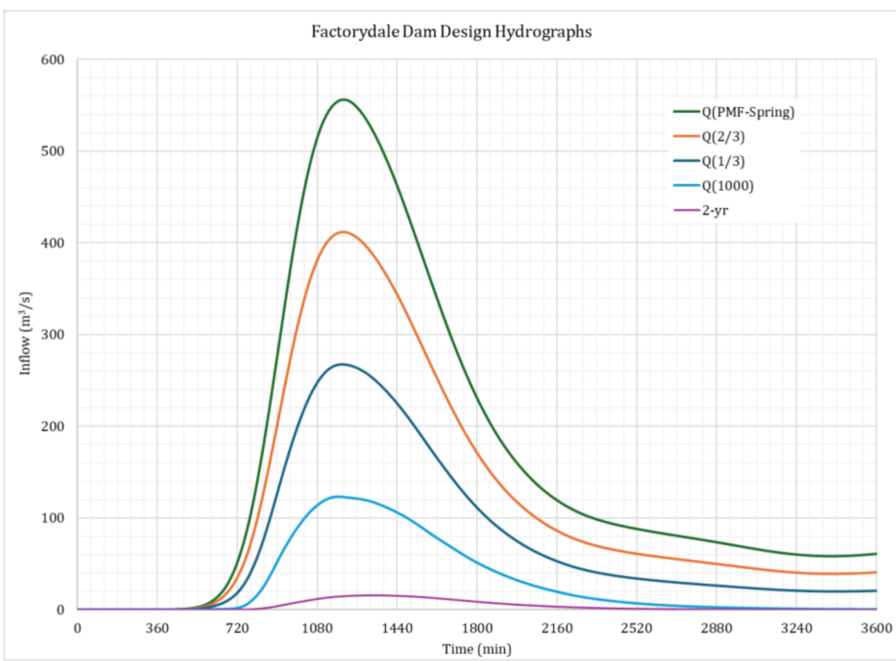


Figure 3.2 Design Storm Inflow Hydrographs at Factorydale Dam

3.5 Creager Diagram

A series of Canadian floods was plotted on the Creager Diagram (Neill, 1986) as shown in Figure 3.3. The Creager Diagram is an envelope for Probable Maximum Flood (PMF) based on gauged extreme events by relating drainage area to unit discharge. Also featured on the Creager plot are floods in US and other foreign rivers (in grey).

Factorydale Dam has a watershed area of 92.3 km^2 and a predicted PMF peak runoff of 557 m^3/s and falls within the envelope on the lower quartile but in line with other extreme Canadian flood events. This is plotted in Figure 3.3.

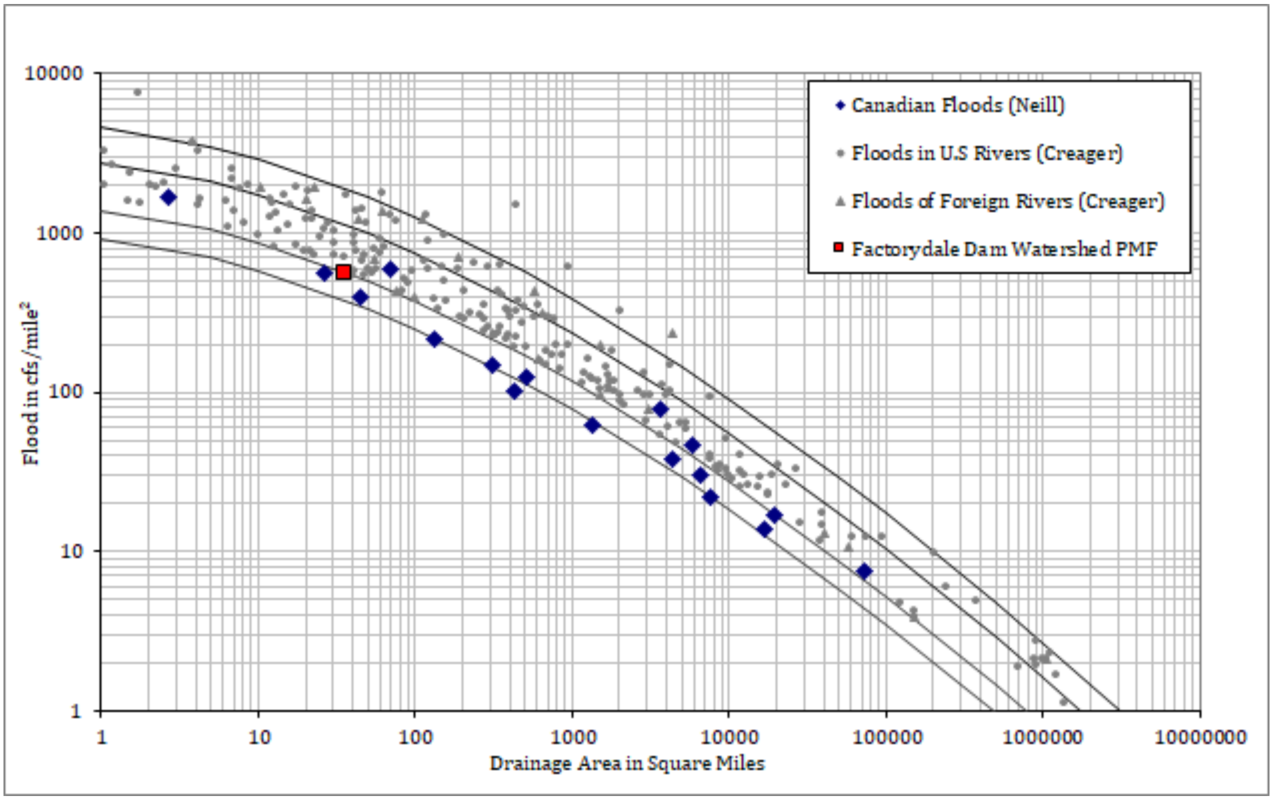


Figure 3.3 Factorydale Dam Watershed PMF On Creager Diagram

4 HYDRAULIC MODELS

A HEC-RAS 2D model was developed to define inundation extents and peak velocities. Dam failure models were generated at the Factorydale Development using HEC-RAS (2D) for three (3) scenarios:

1. Flood event(s) without dam failure – Non-Breach Event (NB),
2. Failure at Factorydale Dam with overtopping/Flood-Induced Failure (FiF),
3. Failure at Factorydale with fair-weather/Fair-Weather Failure (FwF).

Five (5) flow conditions were simulated for each flood scenario:

1. An event with a return period of 100 years (Q_{100}),
2. An event with a return period of 1,000 years ($Q_{1,000}$),
3. An event with a peak one-third between the PMF and $Q_{1,000}$ ($Q_{one\text{-}third}$),
4. An event with a peak two-thirds between the PMF and $Q_{1,000}$ ($Q_{two\text{-}third}$), and
5. A PMF event (Q_{PMF}).

4.1 Model Parameters and Methodology For HEC-RAS 2D

A hydraulic analysis of dam breach was carried out using the computational capabilities of the HEC-RAS 2D model. This 2D modeling approach for dam break models and flood wave propagation analysis captures the flow spread within the floodplains downstream of Factorydale Dam. The 2D model is comprised of various elements. The geometric data required for a 2D model set up includes a Digital Elevation Model (DEM) of the terrain, definition of 2D flow areas which are used to generate a 2D computational mesh, and geometric data on structures such as bridges, culverts, and inline structures with gates.

4.1.1 Terrain Data

The digital elevation model (DEM) used to establish terrain was obtained from the GeoNova Data Locator Tool which provides a 1 meter resolution Geo tiff for the province of Nova Scotia. The DEM was developed in 2019 (GeoNova, 2019). The dataset used is in the form of a DEM and is imported using RAS Mapper, a HEC-RAS tool that is a dedicated interface for visualization and processing of geospatial data.

4.1.2 Roughness Coefficient (Manning's n)

Manning's roughness coefficients are assigned to each of the 2D cells in the computation grid based on the land use classification according to Table 4.1. Information on land use classification was sourced from the Nova Scotia Government open-source data website https://novascotia.ca/natr/forestry/ecological/ecolandclass.asp and imported into RAS Mapper. Land zoning classification was used to accommodate the impacts of potential future development.

Table 4.1 Manning's n values based on Land Use Classification

Land Use	Description	Manning's n
0	No Data	0.04
18	Estuarine Emergent Wetland	0.04
1	Unclassified	0.04
6	Cultivated Crops	0.035
5	Developed - Open Space	0.03
10	Evergreen Forest	0.05
15	Palustrine Emergent Wetland	0.04
17	Estuarine Scrub-Shrub Wetland	0.065
14	Palustrine Scrub-Shrub Wetland	0.065
8	Grassland-Herbaceous	0.03
16	Estuarine Forested Wetland	0.08

4.1.3 Boundary Conditions

Boundary conditions are required at the upstream and downstream end of the 2D Flow Area. The runoff hydrographs generated in HEC-HMS were input as the upstream boundary condition to the headpond for each flood scenario. The downstream boundary condition was taken as constant flat elevation where the channel encounters the ocean after approximately 70 km.

For the FwF model, the AEP1:2 flood runoff hydrograph was used for model stability and as a baseline flow.

4.1.4 Initial Conditions and Breach Trigger Time

For all events, the initial water level was set at Full Supply Level (FSL), the concrete sill of the spillway. Analysis was performed assuming all stoplogs are removed before the storm event. For the FwF, the dam was breached suddenly. For the FiF events, the dam breach was triggered approximately one hour before the peak water level to allow time for the breach to develop and coincide with the MFL.

4.2 Non-Breach Hydraulic Routing

A non-breach analysis was performed to define the time and maximum level of the peak flood in the forebay and downstream. The analysis used the HEC-RAS 2D model to define the inundation extents and peak flows when the dam is in place.

Flood discharge is controlled by a concrete overflow spillway with a sill elevation of 68.21 m. The dam operates with stoplogs, but for all dam failure models, the reservoir initial water level is 68.21 m. In Table 4.2, routing results for different inflows are presented.

Table 4.2 Routing Results for Factorydale Dam (Dam Crest Elev. 69.31 m)

Flood Event	MFL (m)	Peak Inflow (m^3/s)	Peak Outflow (m^3/s)	Overtopping	Δ (MFL-Dam Crest) (m)
Q_{100}	69.67	78.9	74.0	Yes	+0.36
$Q_{1,000}$	69.93	123	111	Yes	+0.62
$Q_{1,000} + \frac{1}{3}(PMF - Q_{1,000})$	70.57	267	241	Yes	+1.26
$Q_{1,000} + \frac{2}{3}(PMF - Q_{1,000})$	71.10	412	359	Yes	+1.79
PMF	71.56	557	489	Yes	+2.25

4.3 Dam Breach Parameters

In dam safety engineering, understanding the potential breach development characteristics is paramount for risk assessment and emergency planning. The following table outlines breach development guidelines for various dam types including earthen/rockfill, concrete gravity, concrete arch, and slag/refuse. These guidelines provide average breach width, the horizontal component of the breach side slope, the expected failure time, and the standards set by agencies such as the USACE, FERC, and NWS.

Table 4.3 Dam Breach Development Guidelines by Dam Type and Regulatory Agency

Dam Type	Average Breach Width (B_{ave})	Horizontal Component of Breach Side Slope (H) (H: V)	Failure Time (t_f) (hours)	Agency
Earthen/Rockfill	(0.5 to 3.0) x HD	0 to 1.0	0.5 to 4.0	USACE 1980
	(1.0 to 5.0) x HD	0 to 1.0	0.1 to 1.0	FERC
	(2.0 to 5.0) x HD	0 to 1.0 (slightly larger)	0.1 to 1.0	NWS
	(0.5 to 5.0) x HD	0 to 1.0	0.1 to 4.0	USACE 2007
Concrete Gravity	Multiple Monoliths	Vertical	0.1 to 0.5	USACE 1980
	Usually ≤ 0.5 L	Vertical	0.1 to 0.3	FERC
	Usually ≤ 0.5 L	Vertical	0.1 to 0.2	NWS
	Multiple Monoliths	Vertical	0.1 to 0.5	USACE 2007
Concrete Arch	Entire Dam	Valley wall slope	≤ 0.1	USACE 1980
	Entire Dam	0 to valley walls	≤ 0.1	FERC
	(0.8 x L) to L	0 to valley walls	≤ 0.1	NWS
	(0.8 x L) to L	0 to valley walls	≤ 0.1	USACE 2007
Slag/Refuse	(0.8 x L) to L	1.0 to 2.0	0.1 to 0.3	FERC
	(0.8 x L) to L		≤ 0.1	NWS

The approach considers the volume of water at the time of failure, height of the final breach (from the top of the dam to natural ground elevation at the breach location) and breach side slopes. The dam breach parameters used in the HEC-RAS calculations are shown in Table 4.4.

Table 4.4 Dam Breach Input Data for All Scenarios at Factorydale Dam (Dam Crest Elev. 69.31 m)

Final Bottom Width (m)	Final Bottom Elevation (m)	Left Side Slope (xH:1V)	Right Side Slope (xH:1V)	Breach Weir Coefficient	Breach Formation Time, t_f (h)
12	64	0.1	0.1	1.5	0.17

4.4 Model Results

The following symbols represent times associated with peak water levels,

- "t_p" denotes the arrival time of the peak inflow due to a rainfall event. This pertains to non-breach peak flood times at the spillway structures,
- "t_f" denotes the breach formation time at a dam. It is the time taken for a breach to develop from initiation to a fully developed failure width, and
- "TTP" denotes the time to peak following a breach event at a specific dam/spillway, bridge, or building.

The following symbols represent flows associated with peak water levels,

- "Q_p" denotes peak flows during a non-breach,
- "Q_f" denotes peak flows during a breach.

Seven (7) failure events are routed through the HEC-RAS 2D model to estimate downstream inundation from failure events. Events include single dam failures. The output for each failure event is a maximum surface water elevation in the reservoirs, an estimate of time to failure and the expected volume of water released. The model provides a spectrum of results in the downstream inundation areas including the velocity and depth of flow at all locations, as well as the time of arrival following the initiation of a breach (TTP) at critical locations.

4.4.1 Dam Failure Modes

The principal failure mode for concrete dams for the FwF condition is initiation of internal erosion through the foundation which causes an increase in seepage rates. The seepage rate progression results in an increase in uplift pressures and undermining of the dam. The dam fails in sliding resulting in an uncontrolled release and failure to the dam foundation.

The principal failure mode at Factorydale Dam during a FiF is a buildup of headwater and tailwater, and a filling of the gallery within the dam until it loses self weight and fails due to sliding or overturning. An additional failure mode is failure along a local crack in the structure which is possible at Factorydale Dam given the poor condition of the structure.

4.4.2 Failure at Factorydale Dam/Spillway

Dam breach results are shown in Table 4.5. The results compare Factorydale Dam's performance under two primary scenarios: Fir-weather Failure and Flood-induced Failure. Each scenario includes peak breach flow rates (Q_{Br}), time of breach formation (t_{Br}), time to peak flow (TTP), and total outflow volume in 24 hours following breach initiation for a sequence of extreme flood events from Q_{100} to PMF. Failure is rapid for all cases, with TTP and t_{Br} = 0.17 hours.

- Sunny Day Failure: This scenario assumes dam failure without additional stresses from high inflows or extreme weather conditions.
- Rainy Day Failure: In this scenario, the dam fails due to a high flood water level which may or may not overtop the dam. As the flood magnitude increases from AEP_{100} to PMF, the peak breach flow rate (Q_{Br}) increases significantly.

The peak outflow from a dam failure is estimated at 2x the outflow for a non-failure at the AEP100 event, reducing in magnitude to just 20% more outflow for a failure during the PMF. A dam failure at the peak reservoir during an AEP1,000 flood event suddenly increases the outflow by approximately 70% and results in 4.5 million litres of water released to the downstream environment.

Table 4.5 Breach Results for Sunny Day Failure and Flood Induced Failure

Parameter	Sunny Day	Overtopping				
	Q_{Piping} (m³/s)	Q_{100} (m³/s)	Q_{1000} (m³/s)	$Q_{1/3}$ (m³/s)	$Q_{2/3}$ (m³/s)	Q_{PMF} (m³/s)
Q_{Br} (m³/s)	37.6	162	206	320	452	586
t_{Br} (hr)	0.17	0.17	0.17	0.17	0.17	0.17
TTP (hr)	0.17	0.17	0.17	0.17	0.17	0.17
Total Outflow Volume in 24h following Breach Initiation (x10⁶ m³)	0.024	3.11	4.57	10.2	14.8	21.0

5 CONSEQUENCE CLASSIFICATION EVALUATION

A surface level evaluation of the impacts of a dam breach and the present flood mapping from the 2011 EPP was completed as part of this assessment. When the flood mapping was completed in 2007, surveying and dimensioning was completed downstream of the dam, and the flood mapping terminated 7 km downstream of the dam, south of Auburn, NS. The assessment identified twenty (20) residences, and five (5) bridges would be impacted by the flood wave resulting from a dam breach during the $AEP_{1,000}$ event.

The 2007 consequence assessment may have over classified the dam because, 1) it did not identify zones of high flow or low flow when counting the number of impacted residences; and 2) it did not identify the *incremental* flood failure risk, which is the Canadian Dam Association (CDA) standard for assessing Dam consequence. The industry standard metric for determining life safety risk during a flood is the Depth × Velocity value, or "DV Value". It is necessary to have a standard practice in life safety assessments for determining at what point a fatality will occur. For this DSR, the Ontario Ministry of Natural Resources guideline was assumed, which assumes that for flood water that has a DV value of less than 4 ft^2/s, (2ft in depth and 2ft/s), there will not be a fatality risk for the typical person at a dwelling **(OMNR, 2011)**. To determine the incremental flood failure risk of a dam, residences that are inundated *regardless* of a dam breach are ignored. The residences that are counted include only residences that would be flooded exclusively in the event of a dam failure.

The 2007 consequence assessment may have under classified the dam because 1) it used elevation data limited to a handful of cross sections, 2) the extents of the flood mapping are only 7 km whereas the total reach to the outlet at Annapolis Royal is 71 km, and 3) it used a limited dam breach program, SMPDBK. The program is primarily limited because it only computes a peak flow, not a flood hydrograph, and it does not account for backwater effects.

The 2007 consequence assessment also did not check the consequence of the Fair-Weather Failure (FwF) or "Sunny Day Failure" where the dam breaches during a typical day. Assessment of this event is recommended by the CDA Guidelines because 1) it is typically more abrupt than a flood induced failure, and 2) there are more recreationalists during a sunny day. The incremental consequence may be larger during a FwF because where the river is not flooding preceding the dam breach, everything in the breach path will count as incremental consequence.

A surface level dam breach assessment was completed to re-assess the downstream flood extents for the Flood-Induced Failure and for the Fair-Weather Failure. The IDF used in the present assessment was that recommended by the CDA Guidelines for a high consequence dam, the $Q_{one\text{-}third}$. Figure 5.1 shows the total flood extents for the 71 km reach from Factorydale Dam to the Annapolis Basin. Figure 5.2 shows the 2024 IDF flood extents (blue) overlaying the 2007 flood extents. Bridges in the flood path are shown in (red). Residences in the flood path are shown in (green).

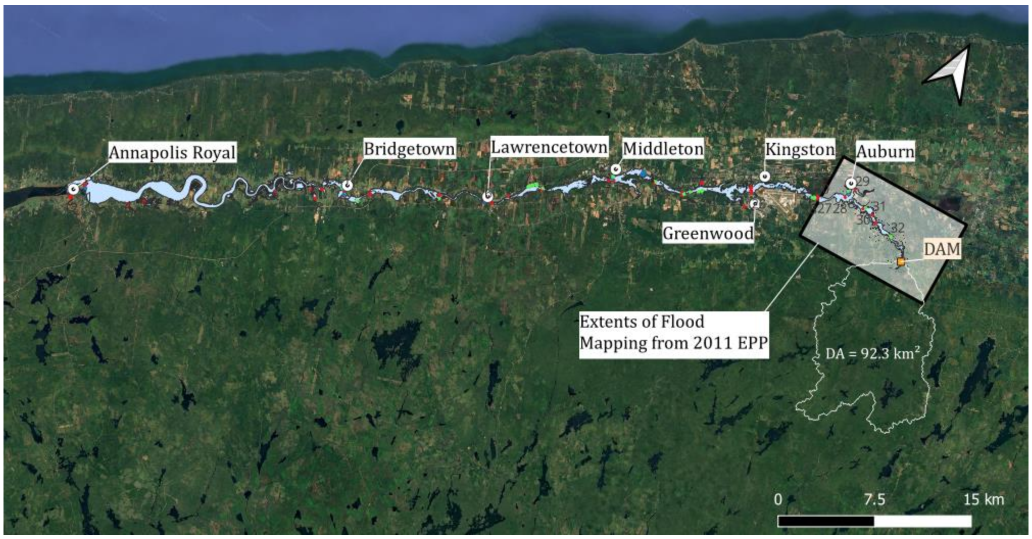


Figure 5.1 Flood Extents for Full Reach during the IDF Flood Induced Failure Scenario

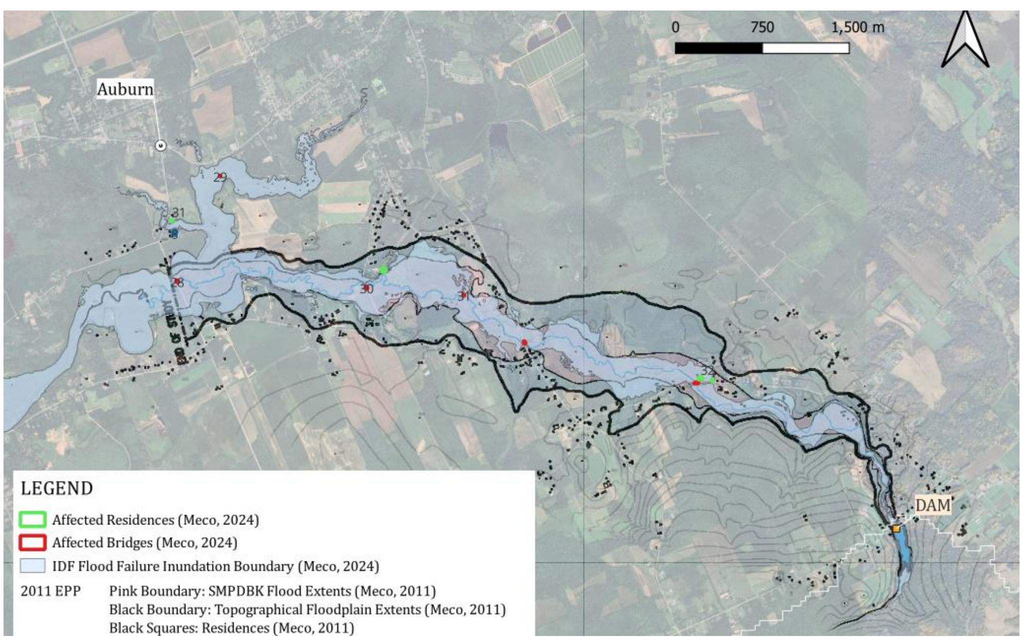


Figure 5.2 2011 Flood Map Extending to Auburn, NS, and IDF Flood Failure Extents from 2024 Study

The high level assessment concluded 1) there is almost no incremental difference in the non-failure flood extents compared to the failure flood extents. This is due to the large magnitude of the storm precipitation volume during the IDF compared to the small storage behind the dam; and 2) During the fair-weather failure, no residences are expected to be inundated in the entire downstream reach to Annapolis Royal.

Because of these conclusions, the consequence should not be governed by life safety. The BEC should conduct a more detailed assessment on 1) Temporary Population at Risk, and 2) Economic risk to the thirty (30) bridges that fall within the floodplain downstream of the dam. Economic risk due to downstream bridge infrastructure is expected to govern consequence classification of the dam.

6 FREEBOARD

As per the CDA Guidelines:

- **Normal Freeboard -** No overtopping by 95% of the waves caused by the most critical wind with a frequency of 1:1,000 year when the reservoir is at its maximum normal elevation.
- **Minimum Freeboard -** Overtopping by 95% of the waves caused by the most critical wind when the reservoir is at its maximum extreme level during the passage of the IDF. The suggested Annual Exceedance Probability (AEP) to be used for the extreme wind are: 1:100 for low consequence dams, 1:10 for significant consequence dams and 1:2 for high, very high and extreme consequence dams.

Probabilistic estimates of extreme values for instantaneous wind speeds were obtained generated from historical winds recorded at Environment Canada Climate Station "Greenwood A." (Climate ID: 8202000). A Gumbel distribution was used as representative of variation and recorded data was assumed to be recorded as overland wind at an elevation of 10 meters.

For the normal condition, the reservoir operating level is assumed to be concrete spillway sill with all flashboards removed. The design wave for normal operating conditions is generated for a windstorm with an AEP of 1,000 years, which has an estimated maximum overland wind speed of 37.9 m/s. Factorydale Dam was previously classified as high consequence and, for the minimum condition, the operating level is assumed to be the maximum flood level (MFL) during the IDF and the design wave is generated for a windstorm with AEP of 2 years, estimated as 21.6 m/s.

The US Shore Protection Manual (1978) was referenced to calculate a significant wave height, H_S, and USBR Design Standard No. 13 (2012) was referenced to calculate Wave Setup and Wave Runup at the dams. Aerial photography was taken using a drone during the site inspection (24/08/08) and stitched together to reveal shoals in the head pond not visible in satellite imagery. Accounting for these shoals reduced the calculated Significant Wave Height, H_s by 40%. The total and effective fetch distance are shown in Figure 6.1.

The dam was assessed as two (2) separate sections for freeboard deficiency; (1) the exposed concrete dam in the middle and right sections, which has a vertical face; and (2) the rockfill-entombed left section of the dam which has a 2H:1V upstream slope. The CDA Guidelines allow for some wave splash overtopping of concrete dams, so the standard for the middle and right sections of the dam is that the dam will not be overtopped by the Significant Wave, H_s , which is the threshold for the highest 1/3 of wave heights in the wave spectrum for the given design wind speed. At the rockfill section, splash overtopping is considered a risk for the rockfill. In addition, the entombed concrete parapet must not be overtopped by the MFL, otherwise there is a risk of washout of the rockfill. Freeboard assessment results are summarized in Table 6.1. Calculations are provided in Attachment B.

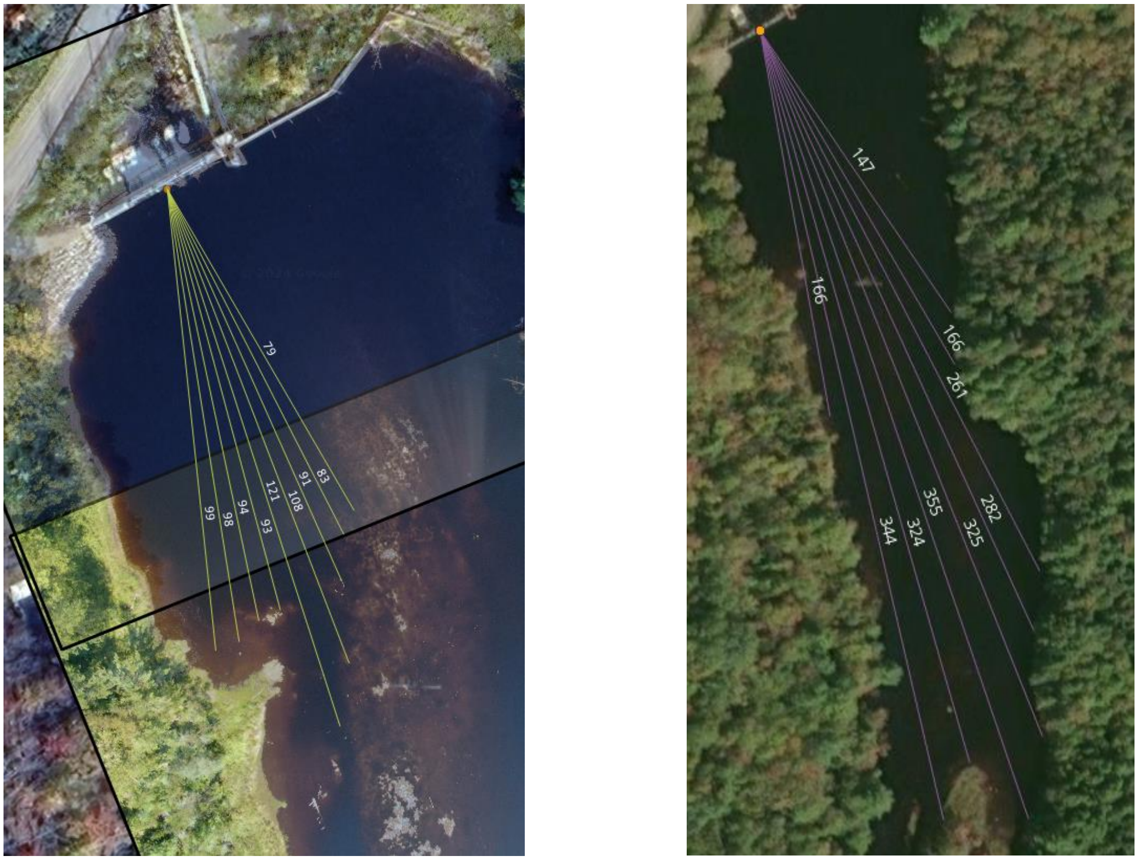


Figure 6.1 Fetch Distance at Factorydale Dam

Freeboard assessment results are summarized in Table 6.1. Calculations are provided in Attachment B. During the normal (FSL) condition, the available freeboard is sufficient at both the left entombed wall section and at the middle and right concrete dam sections. During the flood condition, there is a freeboard deficiency of 0.12 metres at the left entombed section which is driven by the entombed concrete parapet being overtopped by the IDF, and the middle and right vertical concrete dam sections are deficient by 0.62m which is governed by the maximum flood level overtopping the dam as opposed to wind-generated waves.

Table 6.1 Freeboard Results

Parameter	Factorydale Dam - Entombed Left Wall		Factorydale Dam - Vertical Concrete Dam	
	Normal (m)	Minimum (m)	Normal (m)	Minimum (m)
FSL	68.21	" "	68.21	" "
MFL ($AEP_{1,000}$)	69.93	" "	69.93	" "
Top of Dam	70.54	" "	69.31	" "
Top of Core	69.81	" "	69.31	" "
Significant Wave, H_s	0.47	0.28	0.47	0.28
Set-up	0.05	0.01	0.03	0.01
Run-Up	0.98	0.55	N/A	N/A
Required Freeboard	1.03	0.56	0.50	0.29
Available Freeboard	2.33	-0.12	1.10	-0.62
Adequate (Y/N)	YES	NO	YES	NO

ATTACHMENT A

Hydrology Calculation Report - River Basin Transfer

Project number :	10777	Calculed by :	Oscar Moyles, EIT
Client :	Berwick Electric Commission	Reviewed by :	Perry Mitchelmore, P.Eng.
Object :	River Basin Transfer	Date :	December 18, 2024

1. Purpose

These calculation notes present the river basin transfert methodology.

2. References

ROUSSELLE, Jean, WATT, W. Edgar, LANTHEM, Keith W., NEILL, Charles R. et T. Lloyd RICHARDS (1990). *Hydrologie des crues au Canada - Guide de planification et de conception*, Ottawa, Comité associé d'hydrologie, Conseil national de recherche Canada, 277 p.

3. Methodology

The first step of this methodology is to find a hydrometrical station located on a watershed that is similar to the un-gaged watershed for which the extreme floods are needed. It should be noted that for better reliability, the watersheds should be located relatively close. Also, the ratio of both watersheds area should range from 0.5 to 2.

Once the station is chosen and the data available, a statistical analysis is made to extrapolate the extreme event from the data. The HYFRAN software is used for this analysis.

Once the different extreme floods of the station are known, a transfer can be made from the gauged watersheds to the un-gauged watersheds using equation 1 (Rousselle, 1990) :

$$Q_{inst.site} = Q_{inst.station}\left(\frac{A_{site}}{A_{station}}\right)^{b} \quad \text{Eq. 1}$$

Where :
$Q_{inst.site}$ is the estimated instantaneous flow at the un-gauged site in m^3/s
$Q_{inst.station}$ is the instantenous maximum flow at the gauged site in m^3/s
$A_{station}$ is the drainage area of the gauge site in km^2
A_{site} is the drainage area of the un-gauged site in km^2

Finally, the coefficient b range between 0.6 and 0.9 based on the following assumptions :
- Upper value of b are associated with short return period and to important storage area on the watershed;
- Lower value of b are associated with long return period and to little storage area on the watershed. By default, a value of 0.8 is often used.

It should be noted that the routing effect of the gauged watershed is also transposed with equation 1. Depending on the type of flow that is needed (already routed, not routed, etc.), the calculated flow can be unrouted and routed again by using a routing coefficient that varies with the watershed's caracteristics. This method was developped by the former "Centre d'Expertise Hydrique du Québec" and is based on the "Manuel des Ponceaux" of the "Ministère des Transports du Québec" (MTQ, 2006).

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The routing coefficient depends on the percentage of lakes and swamps of the watershed and on their disposition. There is three types of arrangment :

A : Storage areas are mostly located near the gauge or studied site;
B : Storage areas are uniformly distributed throughout the watershed;
C : Storage areas are mostly located at the head of the watershed.

Figure 1 shows the variation of the routing coefficient in relation to the percentage of the lakes and swamps for each arrangment.

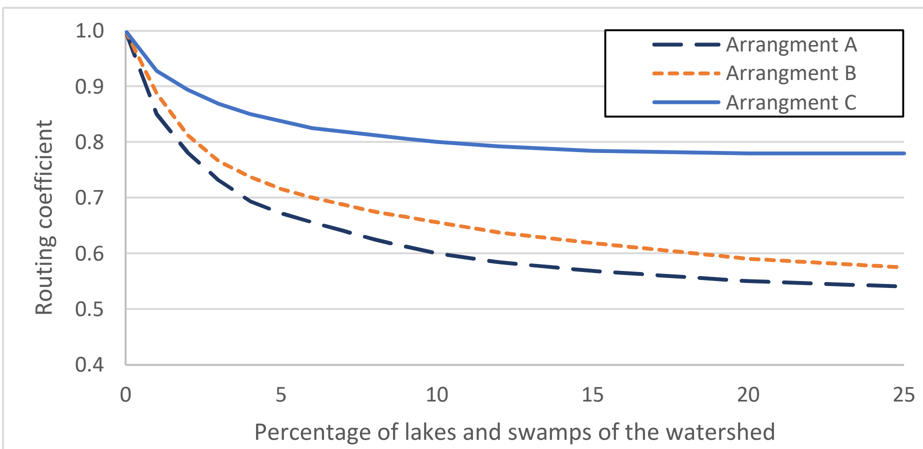

Figure 1 - Routing coefficient of a watershed in relation with the percentage of lake and swamps on its drainage area and of their arrangment

4. Data and Results

4.1 Un-gauged watershed

Table 1 presents the caracteristics of the un-gauged watershed which is showed on figure 2.

Table 1 - Un-gauged watershed caracteristics

Caracteristics		Comments
Drainage area (km^2)	92.3	
Area of lakes and swamps (km^2)	5.6	
Percentage of lakes and swamps	6.1	
Reservoir area (km^2)	0.0296	
Main Channel Length (km)	17.698	
Main Channel Average Slope	0.008	
Arrangment of lakes and swamps	C	
Routing coefficient	0.82	

initials : ____________

Project number : 10777
Client : Berwick Electric Commission
Object : River Basin Transfer
Calculed by : Oscar Moyles, EIT
Reviewed by : Perry Mitchelmore, P.Eng.
Date : December 18, 2024

4.2 Gauged watersheds

Different stations were considered for the river basin transfer which are presented in table 2.

Table 2 - Un-gauged sites consedered

Station number	01DG003	01DD002	01DL001	01EJ001
Drainage area	96.9sqkm	90.8sqkm	63.2sqkm	146.0sqkm
Latitude	44.85110855	45.06478119	45.58681107	44.73152924
Longitude	-63.66389084	-64.6353302	-64.45069122	-63.66043854
Active period	1921-2020	2000-2020	1969-2020	1916-2020
Ratio of areas	0.95	1.02	1.46	0.63
Status	Accepted	Rejected	Rejected	Rejected

01DG003 Little Sackville River Station has the smallest DA of the four selected stations, and is nearby to the site. In order to assess its similarities with the un-gauged watershed, its caracteristics are presented in table 3.

Table 3 - Station 01DG003 caracteristics summary

Drainage area (km^2)	96.9sqkm
Area of lakes and swamps (km^2)	2
Percentage of lakes and swamps	2
Main Channel Length (km)	15
Main Channel Average Slope	0.00856
Arrangment of lakes-swamps	C
Routing coefficient	0.89

As this station is similar to the un-gauged watershed, it was retained to perform the river basin transfer on the un-gauged watershed. A map of the station's watershed in presented at the end of this Good Practices note.

4.3 River Basin Transfer

Table 4 presents the extreme floods of the chosen station that were obtained through the software HYFRAN using a 3-Parameters lognormal regression curve. The results obtained through equation 1 are also shown in table 4.

It should be noted that these floods have been un-routed with the routing coefficient of the un-gauged watershed, but only partially routed again in Lake Major watershed, since they are going be manually routed later in the engineering process.

initials : ____________

Project number :	10777	Calculed by :	Oscar Moyles, EIT
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Object :	River Basin Transfer	Date :	December 18, 2024

Table 4 - Results

T	Gauged-site	Un-gauged site
2	33.5	29.6
100	85.5	75.6
1000	113.9	101
10,000		0

4. Conclusions

The use of this method has permitted to obtain different extreme floods for the un-gauged watersheds of Lake Major. The PMF also needs to be calculated in order to obtain the design flood of Lake Major dam. This calculation is presented in another calculations notes.

Perry Mitchelmore, P.Eng.

Oscar Moyles, EIT

EVA

Station: BEAVERBANK RIVER NEAR KINSAC

Flow Type

- Max Flow
- Mean Flow
- Min Flow
- Peak Flow

Season

- Winter
- Spring
- Summer
- Fall

Re-Fit GEV Starting with Default Parameters

Re-Fit GEV Starting with Existing Parameters

Flow Rates m³/s

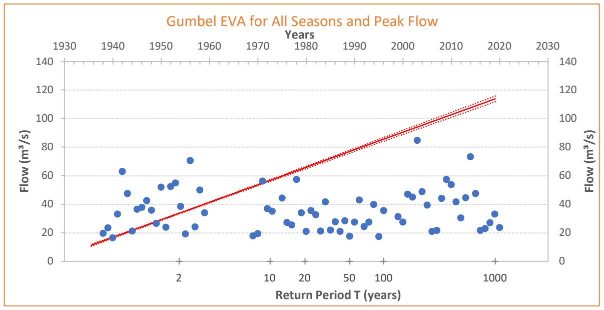
	Gumbell		GEV	
T (Years)	Flow	95 % CI +/-	Flow	95 % CI +/-
2	33.5	0.4	10.4	6.3
10	56.6	0.6	20.5	10.6
20	65.5	0.8	25.0	14.4
50	76.9	1.1	31.4	20.3
100	85.5	1.3	36.8	25.4
1000	113.9	2.0	58.0	46.0

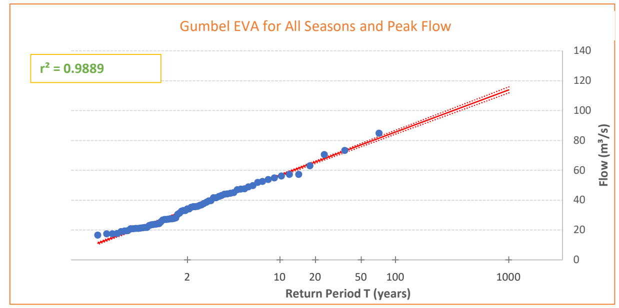
Statistics

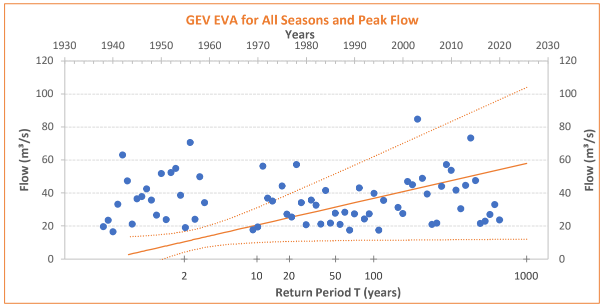
Statistic	
Num Years	71.0
Mean Yr Max	35.8
StDev Yr Max	14.6
Max	84.7
Min	16.5

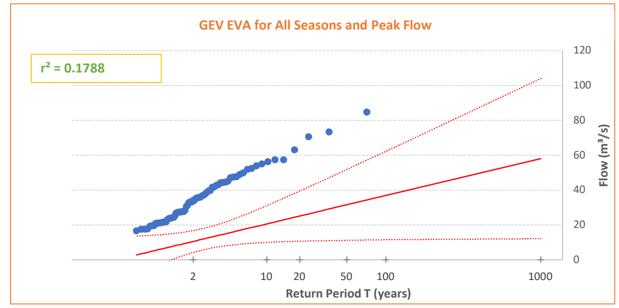
Parameter Estimates	
Gumbell	
yn	0.555
sn	1.186
GEV	
ξ	0.122
μ	8.709
σ	4.551

Regression	Gumbell	GEV
ytMean	0.555	0.082
SSxx	99.917	2.0
SSresid	167.985	46130.3
SE	1.560	25.9
r^2	0.9889	0.1788
t	1.995	1.995









EVA

Station: BEAVERBANK RIVER NEAR KINSAC + 3

Flow Type

- Max Flow
- Mean Flow
- Min Flow
- Peak Flow

Season

- Winter
- Spring
- Summer
- Fall

Re-Fit GEV Starting with Default Parameters

Re-Fit GEV Starting with Existing Parameters

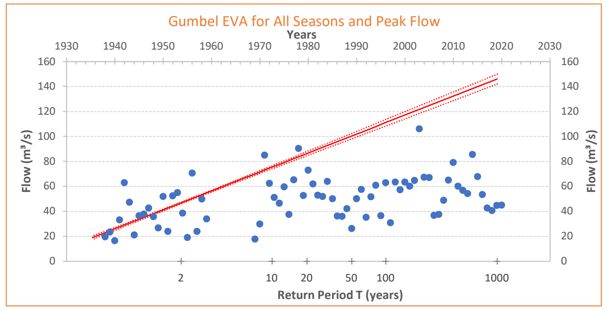
Flow Rates m³/s

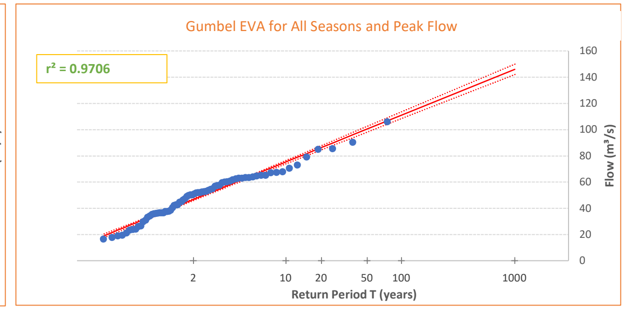
	Gumbell		GEV	
T (Years)	Flow	95 % CI +/-	Flow	95 % CI +/-
2	46.7	0.7	10.4	9.5
10	75.3	1.3	20.5	16.0
20	86.2	1.6	25.0	21.9
50	100.4	2.2	31.4	30.8
100	111.0	2.6	36.8	38.5
1000	146.0	4.0	58.0	69.7

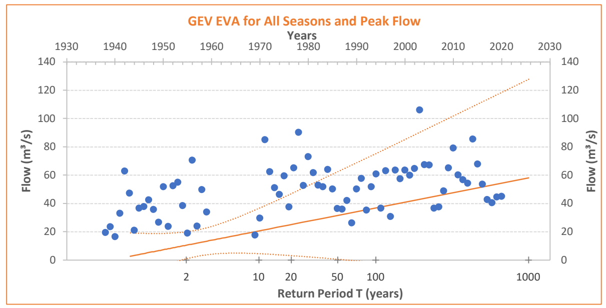
Statistics

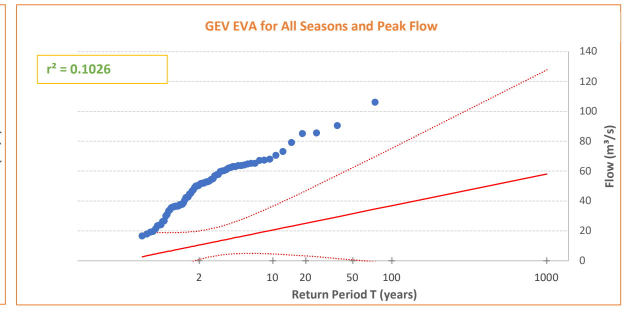
Statistic		**Parameter Estimates**	
Num Years	74.0	*Gumbell*	
Mean Yr Max	49.5	yn	0.556
StDev Yr Max	18.1	sn	1.189
Max	106.0		
Min	16.5	*GEV*	
		ξ	0.122
		μ	8.709
		σ	4.551

Regression	Gumbell	GEV
ytMean	0.556	0.082
SSxx	104.609	2.1
SSresid	710.234	116391.3
SE	3.141	40.2
r^2	0.9706	0.1026
t	1.993	1.993







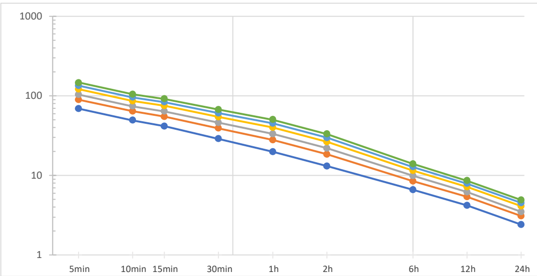


ATTACHMENT B

Calculation Report - Rainfall Analysis

IDF Data

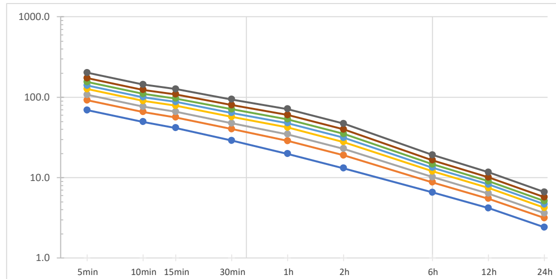
Gumbel from Database



Return Period Rates (mm/h)

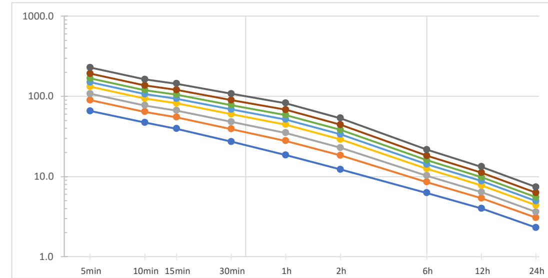
	Return Periods T (years)					
Duration	2	5	10	25	50	100
5min	68.8	89.6	103.4	120.8	133.7	146.5
10min	49.4	64.3	74.1	86.5	95.8	104.9
15min	41.5	55	63.9	75.1	83.5	91.8
30min	28.9	39.1	45.9	54.5	60.8	67.1
1h	19.8	27.9	33.3	40.1	45.2	50.2
2h	13.1	18.4	22	26.5	29.8	33.1
6h	6.6	8.5	9.9	11.5	12.8	14
12h	4.2	5.4	6.2	7.2	7.9	8.6
24h	2.4	3.1	3.5	4.1	4.5	4.9

Calculated Gumbel Distribution



	Return Periods T (years)							
Duration	2	5	10	25	50	100	250	1000
5min	69.1	92.0	107.2	126.4	140.6	154.7	173.3	201.4
10min	49.6	65.9	76.8	90.4	100.6	110.7	124.0	144.0
15min	41.7	56.5	66.3	78.7	87.9	97.0	109.0	127.1
30min	29.0	40.2	47.7	57.2	64.2	71.1	80.3	94.1
1h	19.9	28.8	34.7	42.2	47.8	53.3	60.5	71.5
2h	13.1	19.0	22.9	27.9	31.6	35.2	40.0	47.2
6h	6.6	8.8	10.2	12.0	13.4	14.7	16.5	19.2
12h	4.2	5.5	6.4	7.5	8.3	9.1	10.1	11.7
24h	2.4	3.1	3.6	4.2	4.7	5.1	5.7	6.6

Weibull Distribution



	Return Periods T (years)							
Duration	2	5	10	25	50	100	250	1000
5min	65.8	89.7	107.7	131.6	149.7	167.7	191.6	227.7
10min	47.2	64.3	77.1	94.2	107.1	120.0	137.0	162.8
15min	39.5	54.9	66.6	81.9	93.6	105.2	120.6	143.8
30min	27.3	39.2	48.1	60.0	69.0	78.0	89.8	107.8
1h	18.5	28.0	35.1	44.5	51.6	58.7	68.1	82.3
2h	12.3	18.4	23.0	29.1	33.7	38.3	44.3	53.5
6h	6.3	8.5	10.3	12.5	14.3	16.0	18.3	21.7
12h	4.0	5.4	6.4	7.8	8.8	9.8	11.2	13.3
24h	2.3	3.1	3.6	4.4	5.0	5.5	6.3	7.4

IDF Calculation

Return Period Amounts (mm)

Return Period T (years)	5min	10min	15min	30min	1h	2h	6h	12h	24h
2	5.76	8.26	10.41	14.48	19.86	26.24	39.49	50.44	57.96
5	7.67	10.99	14.12	20.12	28.81	38.06	52.60	66.08	75.50
10	8.93	12.79	16.57	23.86	34.73	45.88	61.28	76.44	87.11
25	10.53	15.07	19.67	28.58	42.22	55.77	72.24	89.52	101.77
50	11.72	16.77	21.97	32.08	47.77	63.10	80.37	99.22	112.65
100	12.89	18.45	24.25	35.55	53.28	70.38	88.45	108.86	123.45
250	14.44	20.66	27.25	40.13	60.54	79.97	99.08	121.54	137.67
1000	16.78	24.00	31.79	47.04	71.49	94.44	115.13	140.69	159.14

Return Period Amounts (mm/h)

x	y0	Duration	2	5	10	25	50	100	250	1000
0.08	1.00	5min	69.1	92.0	107.2	126.4	140.6	154.7	173.3	201.4
0.17	1.00	10min	49.6	65.9	76.8	90.4	100.6	110.7	124.0	144.0
0.25	1.00	15min	41.7	56.5	66.3	78.7	87.9	97.0	109.0	127.1
0.50	1.00	30min	29.0	40.2	47.7	57.2	64.2	71.1	80.3	94.1
1.00	1.00	1h	19.9	28.8	34.7	42.2	47.8	53.3	60.5	71.5
2.00	1.00	2h	13.1	19.0	22.9	27.9	31.6	35.2	40.0	47.2
6.00	1.00	6h	6.6	8.8	10.2	12.0	13.4	14.7	16.5	19.2
12.00	1.00	12h	4.2	5.5	6.4	7.5	8.3	9.1	10.1	11.7
24.00	1.00	24h	2.4	3.1	3.6	4.2	4.7	5.1	5.7	6.6

Confidence Intervals - Return Period Amounts (mm)

Return Period T (years)	5min	10min	15min	30min	1h	2h	6h	12h	24h
2	0.09	0.13	0.19	0.20	0.74	1.31	0.93	0.51	1.05
5	0.12	0.17	0.25	0.25	0.94	1.68	1.20	0.66	1.34
10	0.16	0.24	0.34	0.35	1.30	2.31	1.64	0.91	1.84
25	0.22	0.33	0.47	0.49	1.82	3.25	2.31	1.27	2.59
50	0.28	0.41	0.58	0.60	2.24	3.99	2.84	1.56	3.18
100	0.33	0.49	0.69	0.71	2.65	4.74	3.37	1.86	3.77
250	0.40	0.59	0.84	0.86	3.22	5.74	4.08	2.25	4.57
1000	0.50	0.74	1.06	1.09	4.07	7.27	5.17	2.85	5.79

Confidence Intervals - Return Period Amounts (mm/h)

x		Duration	2	5	10	25	50	100	250	1000
0.08		5min	1.1	1.4	1.9	2.7	3.3	3.9	4.7	6.0
0.17		10min	0.8	1.0	1.4	2.0	2.5	2.9	3.5	4.5
0.25		15min	0.8	1.0	1.4	1.9	2.3	2.8	3.4	4.2
0.50		30min	0.4	0.5	0.7	1.0	1.2	1.4	1.7	2.2
1.00		1h	0.7	0.9	1.3	1.8	2.2	2.7	3.2	4.1
2.00		2h	0.7	0.8	1.2	1.6	2.0	2.4	2.9	3.6
6.00		6h	0.2	0.2	0.3	0.4	0.5	0.6	0.7	0.9
12.00		12h	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2
24.00		24h	0.0	0.1	0.1	0.1	0.1	0.2	0.2	0.2

ATTACHMENT C

Calculation Report - Freeboard Analysis

Parameter	Factorydale Dam - Entombed Left Wall		Factorydale Dam - Vertical Concrete Dam	
	Normal (m)	Minimum (m)	Normal (m)	Minimum (m)
FSL	68.21	" "	68.21	" "
MFL	69.93	" "	69.93	" "
Dam Crest	70.54	" "	69.31	" "
Top of Core	69.81	" "	69.31	" "
Significant Wave, H_s	0.47	0.28	0.47	0.28
Set-up	0.05	0.01	0.03	0.01
Run-Up	0.98	0.55	N/A	N/A
Required Freeboard	1.03	0.56	0.50	0.29
Available Freeboard	2.33	-0.12	1.10	-0.62
Adequate (Y/N)	YES	NO	YES	NO

Project number : 10777
Client : Berwick Electric Commission
Object : Factorydale Freeboard Analysis

Calculated by: Aidan McFarland, EIT
Reviewed by : Perry Mitchelmore, P.Eng
Date : December 17, 2024

1. Purpose

To present Meco's calculations details to determine freeboard requiremens for Factorydale Dam, which has been assigned a failure consequence of Significant.

2. References

Canadian Dam Association (CDA) (2013). *Dam Safety Guidelines 2007.* 2013 Edition.

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Environment Canada (EC). St John's, Cape Race and Cape Broyle's Stations. Accessed (2020) *Historical Data.* climate.weather.gc.ca/historical_data/search_historic_data_e.html

National Building Code of Canada (NBCC) (2010) *Appendix C: Climatic and seismic information for building design in Canada.*

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World Meteorological Organization (WMO) (2010) *Guidelines for Converting Between Various Wind Averaging Perios in Tropical Cyclone Conditions.*

3. General Information

Freeboard at a dam is the vertical distance between the still pool reservoir level and the crest of the dam. The purpose of freeboard is to limit the likelihood of overtopping the dam by waves, including consideration of wind, setup and wave runup (CDA, 2007).

In addition, it should be ensured that, for normal operating conditions, steady or quasi-steady water levels (i.e. excluding waves) should not exceed the top of the impervious core. The conditions to be checked are listed as follows (CDA, 2007):
- Maximum reservoir operating water level + dam settlement due to an earthquake;
- Maximum reservoir operating water level + water level rise due to displacement caused by a landslide;
- Maximum reservoir operating water level + wind set-up;
- Maximum IDF reservoir water level + coincidental wind set-up + water level rise due to displacement caused by a concidental landslide.
- Top of core elevation under frost penetration in the area.

However, these best calculations notes are only for freeboard calculations, and another calculation tool should be used for top of core verifications.

The CDA (2007) makes the following distinction between 'normal freeboard' and 'minimum freeboard':

Normal Freeboard : Difference between the lowest elevation of the top of the dam and the normal reservoir operating water level or full supply level. Normal freeboard should be such that the structure is protected against the most critical of the following cases:

- No overtopping by 95% of the waves caused by the most critical wind with an Annual Exceendance Probability (AEP) of 1,000 years;
- Thickness of material covering the impervious core of the dyke is sufficiant to avoid freezing of the core in winter.

Minimum Freeboard : Difference between the lowest elevation at the crest of the dam and the maximum still pool reservoir water level during the inflow design flood (IDF). Minimum freeboard is such that the structure experiences no overtopping by 95% of the waves caused by the most critical wind when the reservoir is at its maximum water level during the IDF. The most critical wind it a function of the consequence classification of the dam. The suggested AEP to be used for the extreme winds are:
- 1/100 year for low consequence dams;
- 1/10 year for significant consequence dams;
- 1/2 for high, very high and extreme consequence dams.

4. Calculations

4.1 Fetch Analysis

The total fetch of the reservoir, which is used for calculation of the wind setup, is measured as the average of the 9 consecutives longest radials at 3° intervals. The total fetch measures from the control structure to the opposite land mass.

The effective fetch for wave height calculations is calculated the same way, but the length of the effective fetch is limited by islands.

Table 1 presents the effective and total fetch calculations and results. Figure 1 and Figure 2 presents the data used in the calculations.

Table 1 : Determination of average fetch length

Angle (°)	Effective Fetch (m)	Total Fetch (m)
-12	79	166
-9	83	344
-6	91	324
-3	108	355
0	121	325
3	93	282
6	94	261
9	98	166
12	99	147
Fetch	96	263

Project number :	10777	Calculated by :	Aidan McFarland, EIT
Client :	Berwick Electric Commission	Reviewed by :	Perry Mitchelmore, P.Eng
Object :	Factorydale Freeboard Analysis	Date :	December 17, 2024

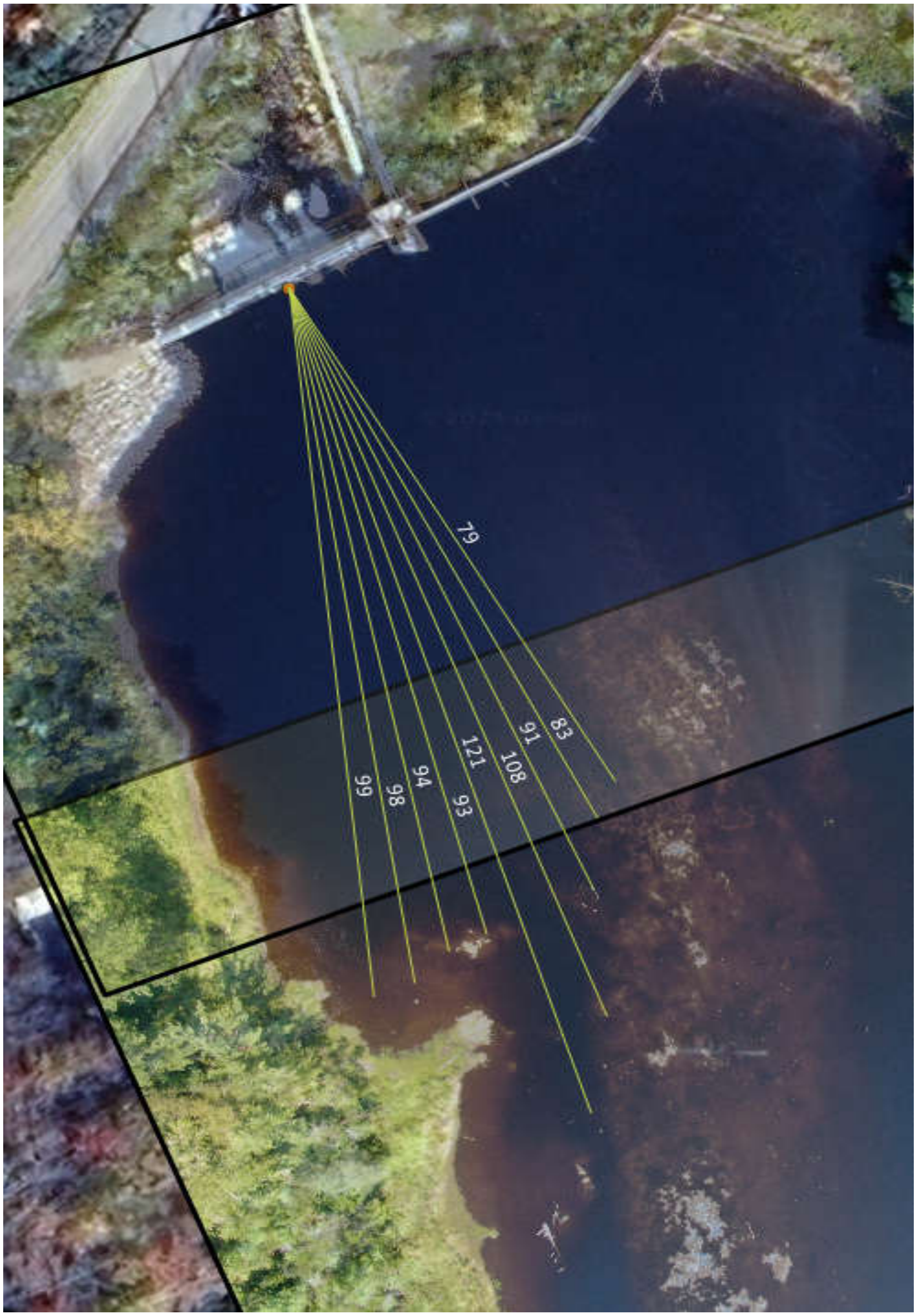

Figure 4 : Determination of effective fetch length (m)

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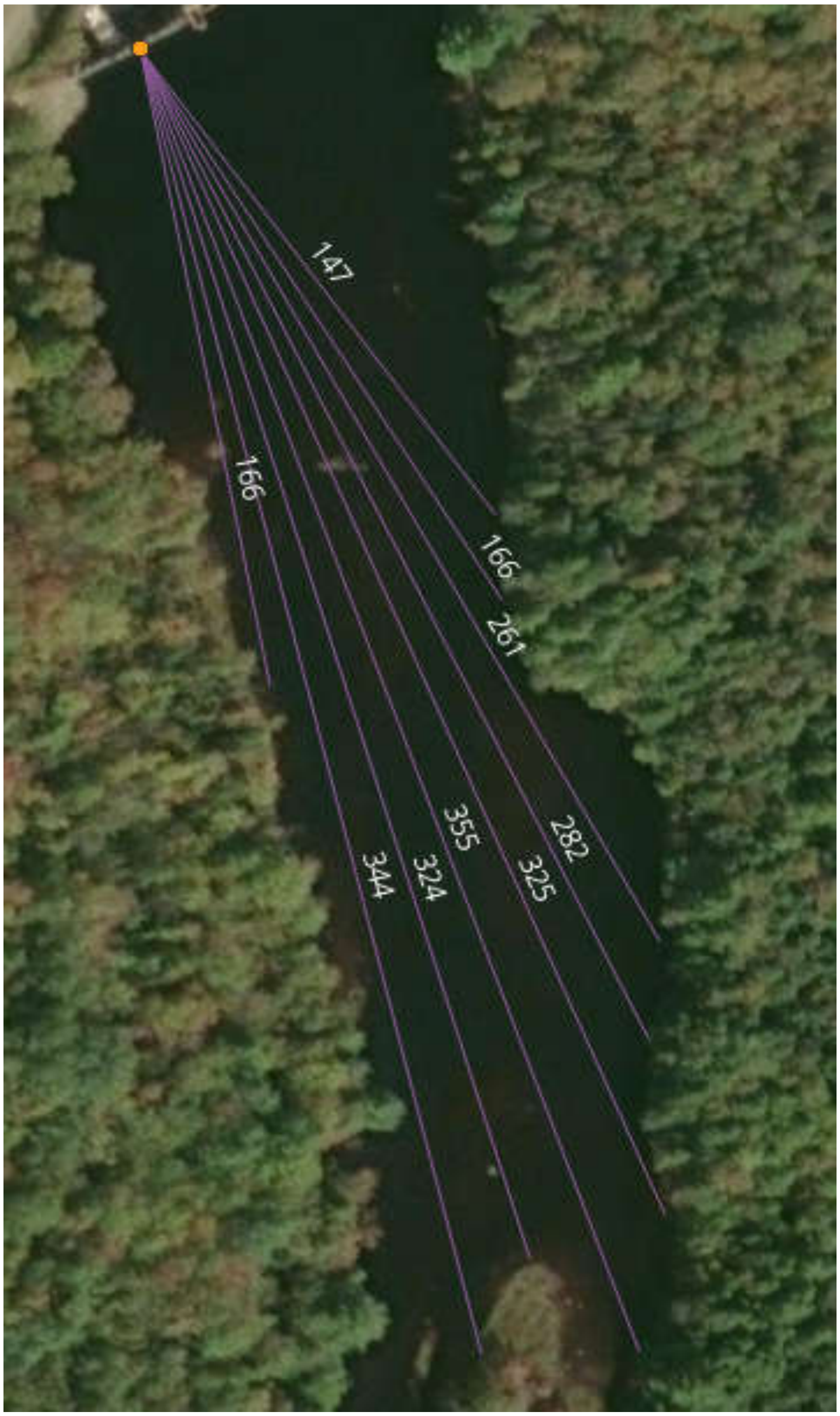

Figure 5 : Determination of total fetch length (m)

Project number :	10777	Calculated by :	Aidan McFarland, EIT
Client :	Berwick Electric Commission	Reviewed by :	Perry Mitchelmore, P.Eng
Object :	Factorydale Freeboard Analysis	Date :	December 17, 2024

4.2 Wind Speed and Wind Stress Factor

The wind speeds recorded at Greenwood Airport by EC were all reviewed and analysed but the data quality was found lacking. Wind speeds at the dam site are calculated from the wind pressures provided in the National Building Code of Canada (2020). The pressure of Greenwood (CFB) is used, with results presented in Table 2.

Table 2: AEP Wind speeds over land calculations

T (year)	Station Transfer		NBCC, 2020 : Greenwood (CFB) Wind pressures		Wind Speeds
2	20.1	m/s	---	---	21.6 m/s
10	25.2	m/s	420	Pa	25.5 m/s
50	29.7	m/s	540	Pa	28.9 m/s
100	31.6	m/s	---	---	30.3 m/s
1000	37.9	m/s	---	---	35.1 m/s

The calculated Wind Stress Factors to be used for the normal and minimum freeboard requirements are presented in Table 3.

Table 3: Wind Stress Factor Calculations - Not adjusted for critical duration

Chosen wind speed methodology	Transfer	
Dam Consequence Classification	Significant	
Freeboard	**Normal**	**Minimum**
U_{land}	37.9 m/s	25.2 m/s
U_{water}	45.5 m/s	30.2 m/s
U_A	77.7 m/s	47.0 m/s

4.3 Freeboard

Table 4 presents the relevant dam characteristics and assumpations used for freeboard calculations and Table 5 presents the Wave Characteristics Calculations.

Project number :	10777	Calculated by :	Aidan McFarland, EIT
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Object :	Factorydale Freeboard Analysis	Date :	December 17, 2024

Table 4 : Dam Characteristics

Dam Characteristics					Comments
Type of Dam	Cement				
Top of Dam Elevation	70.54	m	231.4	ft	
Top of Core Elevation	69.81	m	229.0	ft	
Upstream Slope - Z	2.0	m/m	6.56	ft/ft	**Z** H : 1 V
Upstream Slope - Θ	26.6	°	0.46	rad	
Average Reservoir Level	---	m	---	ft	
Full Supply Level	68.21	m	223.8	ft	
Peak Water Level (IDF)	69.93	m	229.4	ft	(Hec-Ras)
Toe Elevation	65.94	m	216.3	ft	
Dam Height	4.60	m	15.1	ft	

Table 5 : Wave Characteristics Calculations

Freeboard	Normal		Minimum		Comments
Depth of water - d	2.27	m	3.99	m	
Wave Period - T	1.2	seconds	1.0	seconds	
Wave Length - L	2.3	m	1.7	m	
Critical Wind Duration - t	3	minutes	3	minutes	
Ratio d/L	0.98	m	2.40	m	
Wave condition	Deep Water		Deep Water		
$1.37 * H_{shallow}$					
$T_{shallow}$					
$L_{shallow}$					
$t_{shallow}$					
Adjusted Wind Speed - U_W	53.0	m/s	34.7	m/s	Wind duration
Adjusted Wind Stress - U_A	93.7	m/s	55.8	m/s	
Significative Wave Height - H_s	0.47	m	0.28	m	
95% Waves - H_5	0.64	m	0.38	m	

Setup and Runup Calculations are presented in Table 6, along with the required normal and minimum freeboard.

Note that the wave runup equation was not developped for dams with a vertical upstream slope, even though it is steeper than a 5H:1V slope. For concrete dams with vertical slopes, the freeboard is verified on the earthfill embankment. This design choice is considered adequate because, in general, concrete dams are more forgiving for waves passing over their crest when compared to earthfill structures.

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Object :	Factorydale Freeboard Analysis	Date :	December 17, 2024

Table 6 : Runup and Setup Calculations

Freeboard	Normal	Minimum	Comments
Wind Setup - *S*	0.05 m	0.01 m	Hourly winds
Surf Similarity Factor - E_p	0.9	0.9	
Correction Factors			
A	1.6	1.6	
C	0	0	
γ_r	1	1	
γ_b	1	1	
γ_h	1	1	
Angle of Incidence - β	0 °	0 °	
γ_β	1.00	1.00	
Runup - R	3.21 ft 0.98 m	1.81 ft 0.55 m	

Table 7 presents a summary of the results.

Table 7 : Results Summary

Units	
Required Normal Freeboard	**1.03 m**
Existing Normal Freeboard	2.3 m
Required Minimum Freeboard	**0.56 m**
Existing Minimum Freeboard	-0.12 m

Factorydale Dam is not meeting the minimum freeboard requirement.

Project number : 10777
Client : Berwick Electric Commission
Object : Factorydale Freeboard Analysis
Calculated by: Aidan McFarland, EIT
Reviewed by : Perry Mitchelmore, P.Eng
Date : December 17, 2024

1. Purpose

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- 1/10 year for significant consequence dams;
- 1/2 for high, very high and extreme consequence dams.

4. Calculations

4.1 Fetch Analysis

The total fetch of the reservoir, which is used for calculation of the wind setup, is measured as the average of the 9 consecutives longest radials at 3° intervals. The total fetch measures from the control structure to the opposite land mass.

The effective fetch for wave height calculations is calculated the same way, but the length of the effective fetch is limited by islands.

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Angle (°)	Effective Fetch (m)	Total Fetch (m)
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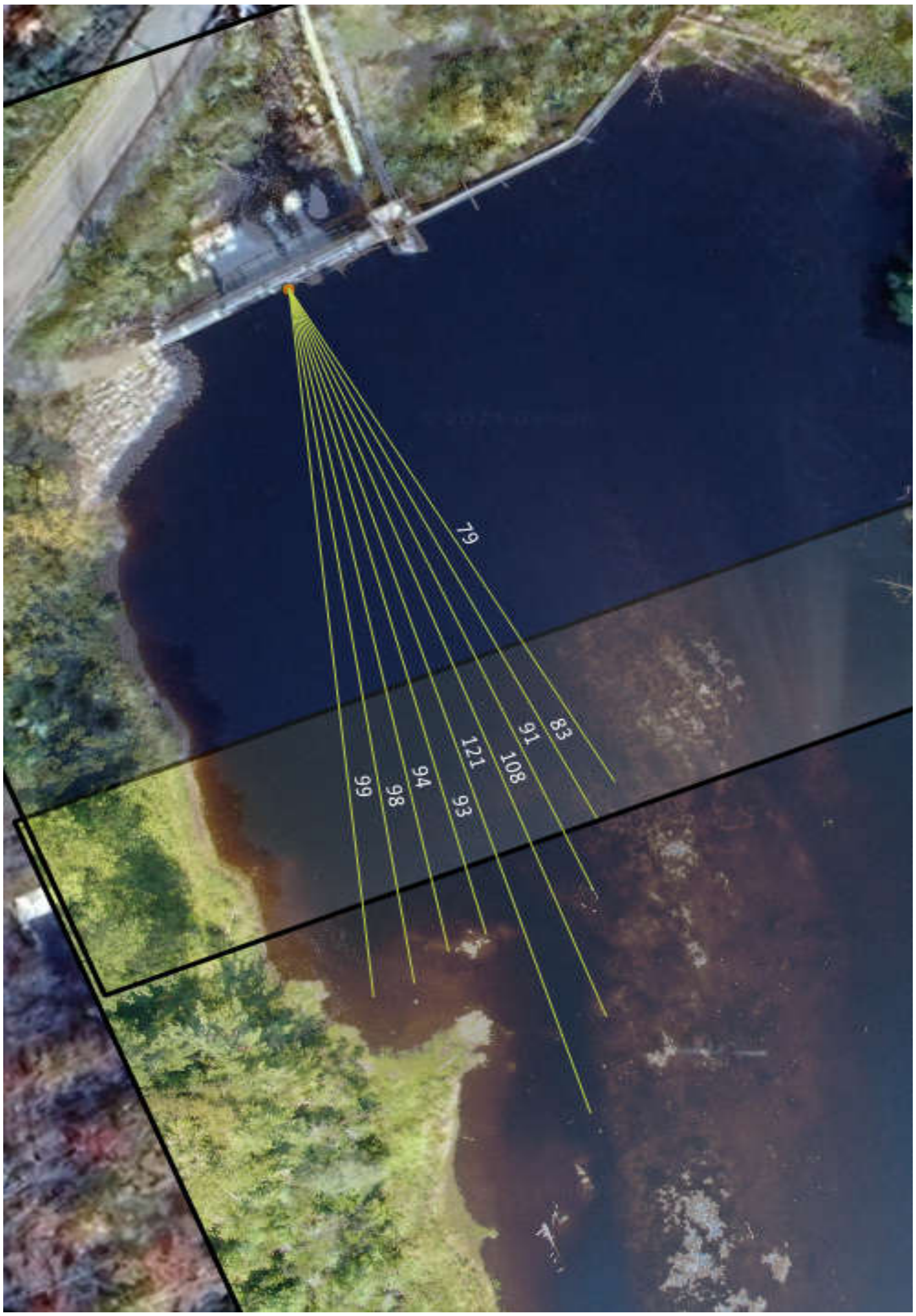

Figure 4 : Determination of effective fetch length (m)

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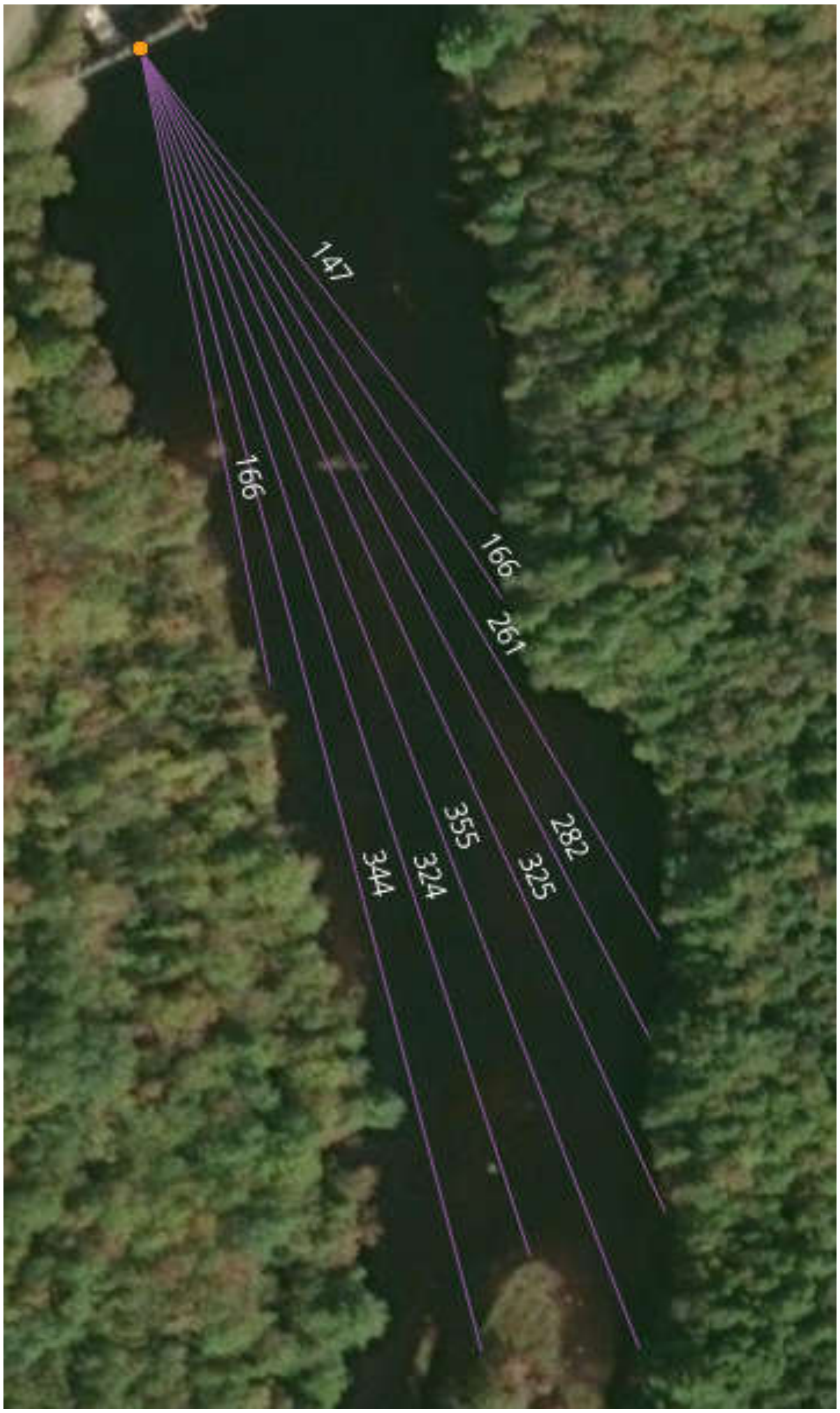

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10	25.2	m/s	420	Pa	25.5 m/s
50	29.7	m/s	540	Pa	28.9 m/s
100	31.6	m/s	---	---	30.3 m/s
1000	37.9	m/s	---	---	35.1 m/s

The calculated Wind Stress Factors to be used for the normal and minimum freeboard requirements are presented in Table 3.

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Chosen wind speed methodology	Transfer	
Dam Consequence Classification	Significant	

Freeboard	Normal	Minimum
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Dam Characteristics					Comments
Type of Dam	Cement				
Top of Dam Elevation	69.31	m	227.4	ft	
Top of Core Elevation	69.31	m	227.4	ft	
Upstream Slope - Z	0.0	m/m	0.00	ft/ft	**Z** H : 1 V
Upstream Slope - ϴ	90.0	°	1.57	rad	
Average Reservoir Level	---	m	---	ft	
Full Supply Level	68.21	m	223.8	ft	
Peak Water Level (IDF)	69.93	m	229.4	ft	(Hec-Ras)
Toe Elevation	64.21	m	210.7	ft	
Dam Height	5.10	m	16.7	ft	

Table 5 : Wave Characteristics Calculations

Freeboard	Normal		Minimum		Comments
Depth of water - *d*	4.00	m	5.72	m	
Wave Period - *T*	1.2	seconds	1.0	seconds	
Wave Length - *L*	2.3	m	1.7	m	
Critical Wind Duration - *t*	3	minutes	3	minutes	
Ratio *d/L*	1.72	m	3.44	m	
Wave condition	Deep Water		Deep Water		
$1.37*H_{shallow}$					
$T_{shallow}$					
$L_{shallow}$					
$t_{shallow}$					
Adjusted Wind Speed - U_W	53.0	m/s	34.7	m/s	Wind duration
Adjusted Wind Stress - U_A	93.7	m/s	55.8	m/s	
Significative Wave Height - H_s	0.47	m	0.28	m	
95% Waves - H_5	0.64	m	0.38	m	

Setup and Runup Calculations are presented in Table 6, along with the required normal and minimum freeboard.

Note that the wave runup equation was not developped for dams with a vertical upstream slope, even though it is steeper than a 5H:1V slope. For concrete dams with vertical slopes, the freeboard is verified on the earthfill embankment. This design choice is considered adequate because, in general, concrete dams are more forgiving for waves passing over their crest when compared to earthfill structures.

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Client :	Berwick Electric Commission	Reviewed by :	Perry Mitchelmore, P.Eng
Object :	Factorydale Freeboard Analysis	Date :	December 17, 2024

Table 6 : Runup and Setup Calculations

Freeboard	Normal	Minimum	Comments
Wind Setup - *S*	0.03 m	0.01 m	Hourly winds
Surf Similarity Factor - E_p	1.0	1.9	
Correction Factors			
A	1.6	1.6	
C	0	0	
γ_r	1	1	
γ_b	1	1	
γ_h	1	1	
Angle of Incidence - β	0 °	0 °	
γ_β	1.00	1.00	
Runup - R	3.38 ft 1.03 m	3.84 ft 1.17 m	

Table 7 presents a summary of the results.

Table 7 : Results Summary

Units	
~~**Required Normal Freeboard**~~	~~**1.06 m**~~
Existing Normal Freeboard	1.1 m
~~**Required Minimum Freeboard**~~	~~**1.18 m**~~
Existing Minimum Freeboard	-0.6 m

Vertical Concrete walls need an alternative approach for evaluation of freeboard.

Evaluate whether the average of the highest one third of the wave spectrum (H_s) does not overtop the structure.

For the Normal condition, Hs = 0.47m. S = 0.03m. 68.21m + 0.47m + 0.03m = 68.71 m < Dam Top ∴ ✓

For the Minimum condition, Hs = 0.28m. S = 0.00m. 69.93m + 0.28m + 0.01m = 70.22 m > Dam top ∴ DEFICIENT

Conclusion: For the minimum condition, the dam top or parapet would have to be raised from 69.31m to 70.22m (+0.91 m) to only be overtopped by the average of highest 33% of waves. Currenly, the MFL for the IDF is +0.62m above the Dam Top.

APPENDIX D

Concrete Stability Assessment

TECHNICAL MEMORANDUM REPORT

Project: Factorydale Dam DSR

Subject: Structure Stability Report

Author: Aidan McFarland, EIT **Reviewer:** Perry Mitchelmore, P.Eng., PMP

Date: December 17, 2024

1.0 INTRODUCTION

The purpose of this Technical Memorandum Report (TMR) is to present the methodology and results of stability analysis for rigid concrete structures at Factorydale Dam.

2.0 ANALYSIS CRITERIA

Analysis was performed using standards-based analysis (SBA) in the framework of the Canadian Dam Association (CDA) guidelines. The SBA method prescribes rigid body mechanics for overturning and sliding in two-dimensional plane strain for concrete structures. The criteria, parameters and detailed computations of the stability analysis are presented in Attachment A. Structures were evaluated for linear deformation due to sliding effects and for resistance to overturning under static load conditions. There are no indications of plastic deformation, and an internal rigidity analysis was not considered. The following loads were considered during the analysis:

- **Dead Load (D)** - Represents the force of gravity calculated based on the mass of the structure.
- **Hydrostatic Load (H)** - Horizontal component representing hydrostatic pressure of the water. For H (Normal Operating Conditions), upstream reservoir elevation at Full Supply Level (FSL) was used. For H_f (Flood Conditions), reservoir elevation at Maximum Flood Level (MFL) was utilized.
- **Ice Load (I)** – Typically, a static ice load of 150 kN/m can be assumed for concrete dams with vertical faces. In cases where there are dynamic ice flows at the structure, the dynamic load is considered. The ice force may be reduced if there is a sloped upstream face or if the ice is loaded primarily on spillway flashboards as opposed to on the concrete body below.

  Ice loading applied to the upstream slope at both structures was reduced to 73.0 kN/m (5 kips/ft) based on the following parameters:

  - January 1% temperature from the National Building Code of Canada (Greenwood Climate Station): *Mild (0 to -20°C)*
  - Headpond shoreline and characteristics: *Steeper Shore (20° to 45° slope)*

- **Uplift Load (U) -** Applied to the base of the structure assuming full uplift pressure on the upstream side and tailwater below the dam. If passive anchors are required to achieve stability, a cracked base is assumed and to result in full hydrostatic uplift at the base.
- **Earthquake Loads (Q) -** Analysis was carried out using the pseudo-static method using a Peak Ground Acceleration (PGA) based on the location of the structure and obtained from Environment Canada. The PGA value represents how hard the earth will shake during an earthquake for a given geographic area. The sum of two load events is assumed for earthquake loads: (1) hydrodynamic loading according to the Westergaard method, and (2) momentum horizontal loading of the mass.

The 2007 CDA guidelines require that the following three categories of load conditions are examined:

- **Usual Load Combinations** – Loads that are associated with normal operation of the facility and occur frequently. Normal loading during summer (No Ice) and winter (Ice) are evaluated separately.
- **Unusual Load Combinations** – Loads that occur infrequently but have a non-negligible probability of occurring within the life of a structure. Unusual loads are evaluated for the MFL during the Inflow Design Flood (IDF), as well as during significant wave action.
- **Extreme Load Combinations -** Loading conditions that are unlikely to occur during the life of the structure. Extreme loading is evaluated for seismic conditions.

For structures with rock or soil anchors, the calculated eccentricity assumes the full strength of the dowelled anchor is utilized, meaning the calculation assumes the anchor is imposing an active overturning resistance force on the structure. In reality, the anchor acts as a passive resistance force, only resisting the tensile forces incurred due to overturning effects. The anchor would resist the tensile force imposed by the overturning forces up to the tensile resistance or capacity of the anchor.

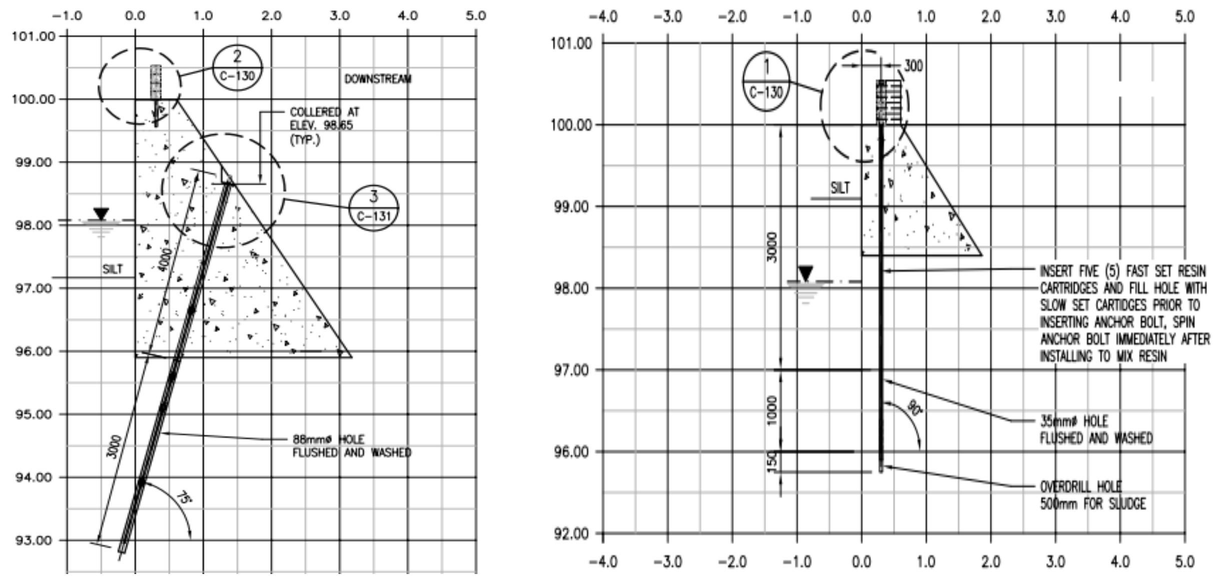
3.0 STABILITY ANALYSIS

3.1 FACTORYDALE GRAVITY DAM

Factorydale Dam is a post-tensioned, concrete gravity structure spanning a total length of 94.3 metres. The dam has a narrow crest, a vertical upstream face and a downstream slope of 1H:1.45V. There is an ogee shaped spillway located near the deepest section of the dam.

The dam was stabilized with 23 post-tensioned anchors; nine (9) Type "A" solid-bar steel resin anchors and fourteen (14) Type "B" all-thread-bar steel grout anchors. The resin anchors are installed vertically at the low-head dam on the right side of the intake, referred to as Section B. The all-thread anchors are in the higher head areas, referred to as section A. All rock anchors are post-tensioned to a minimum of 60 % of their ultimate strength. Typical cross-sections of the dam are shown in Figure 3.1.

The 2008 design indicated the full length of the left concrete gravity dam be post-tensioned to achieve sliding and overturning stability. Site observations indicate four (4) Type "A" anchors adjacent to the right abutment were not installed. The dam in this area is a low head structure but remains vulnerable during flood conditions and during periods of ice loading.



Section A: All Thread Grouted Anchors

Section B: Solid Bar Resin Anchors

Figure 3.1 Typical Cross Sections of the Dam

The geometric properties used for the analysis are listed in Table 3.1, with rock anchor properties listed in Table 3.2. Material properties are presented in Table 3.3. Geometric and rock anchor properties were taken from the *Dam and Spillway Modifications* drawing set prepared by Mitchelmore Engineering Company Ltd. (Meco), dated January 24, 2007.

Table 3.1 Geometric Properties of the Dam

Parameter		Section A	Section B
Height of Wall		5.50 m	1.60 m
Base Width		4.09 m	1.85 m
Cross-Sectional Area		12.95 m^2	1.98 m^2
Location of Centroid from the Downstream Toe	$\bar{x}$ =	2.70 m	1.18 m
	$\bar{y}$ =	2.07 m	0.67 m
EDGM (PGA) (AEP_{1000})		0.046 g[1]	0.046 g

[1] NBCC, 2015

Table 3.2 Rock Anchor Properties in Dam Sections

Parameter	Section A	Section B
Type	Type B All-Thread Bar (Grout Anchor)	Type A Solid Bar (Resin Anchor)
Diameter	25 mm	25 mm
Spacing	3,000 mm c/c	3,000 mm c/c
Length	7.00 m	4.00 m
Embedment Angle	15°	0°
Location of Anchor from the Downstream Toe	3.73 m	1.55 m
Post-Tension Load	340 kN	300 kN

Table 3.3 Material Properties of the Dam

Parameter	Value
Concrete Strength	25 MPa
Unit Weight of Concrete	23.5 kN/m^3
Unit Weight of Rock	25 kN/m^3
Unit Weight of Soil	18 kN/m^3
Unit Weight of Water	9.81 kN/m^3
Base-Rock Angle of Friction	45°

3.1.1 Results – Section A

Results of the sliding stability analysis are presented in Table 3.4. The dam complies with the standards-based criteria in the CDA guidelines for “Usual”, “Unusual”, and “Extreme” loading conditions for concrete structures.

Table 3.4 Sliding Stability Analysis Results

Loading Condition	Required Factor of Safety	Calculated Factor of Safety	Pass/Fail
Usual	1.5	1.86	Pass
Usual (No Ice)	1.5	3.22	Pass
Unusual (Flood)	1.3	1.73	Pass
Unusual (Wave)	1.3	3.09	Pass
Extreme (Earthquake)	1.1	2.71	Pass

Results of the overturning analysis are presented in Table 3.5. The dam complies with standards-based criteria for overturning stability in the CDA guidelines for “Usual”, “Unusual”, and “Extreme” loading conditions.

Table 3.5 Overturning Stability Analysis Results

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	0.68	0.67	Pass
Usual (No Ice)	0.68	-0.38	Pass
Unusual (Flood)	1.36	0.32	Pass
Unusual (Wave)	1.36	-0.38	Pass
Extreme (Earthquake)	2.05	-0.26	Pass

3.1.2 Results – Section B

Results of the sliding stability analysis are presented in Table 3.6. The section complies with the standards-based criteria in the CDA guidelines for "Usual", "Unusual", and "Extreme" loading conditions for concrete structures.

Table 3.6 Sliding Stability Analysis Results

Loading Condition	Required Factor of Safety	Calculated Factor of Safety	Pass/Fail
Usual	1.5	1.75	Pass
Usual (No Ice)	1.5	22.4	Pass
Unusual (Flood)	1.3	4.89	Pass
Unusual (Wave)	1.3	13.3	Pass
Extreme (Earthquake)	1.1	16.1	Pass

Results of the overturning analysis are presented in Table 3.7. This section complies with standards-based criteria for overturning stability in the CDA guidelines for all specified loading conditions.

Table 3.7 Overturning Stability Analysis Results

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	0.31	0.16	Pass
Usual (No Ice)	0.31	-0.24	Pass
Unusual (Flood)	0.62	-0.10	Pass
Unusual (Wave)	0.62	-0.21	Pass
Extreme (Earthquake)	0.93	-0.23	Pass

3.1.3 Sensitivity Analysis – Section A

Sensitivity analysis was performed on this section of the structure to assess the minimum allowable post-tensioning load applied to the rock anchors. The potential failure mode is overturning effects caused by the relaxation of the post-tensioning load. Analysis consisted of incrementally decreasing the post-tensioning load on the anchors until there was indication that the section did not meet the standards-based requirements for stability. The critical failure mode in this case was overturning stability with results presented in Table 3.8. The critical state was a post-tension anchor load of 331 kN per anchor, or 58 % of ultimate load.

Table 3.8 Overturning Stability Analysis Results with Reduced Post-Tensioning Load

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	0.68	0.69	Fail
Usual (No Ice)	0.68	-0.36	Pass
Unusual (Flood)	1.36	0.34	Pass
Unusual (Wave)	1.36	-0.37	Pass
Extreme (Earthquake)	2.05	-0.24	Pass

3.2 FACTORYDALE SPILLWAY

Factorydale spillway is a post-tensioned, concrete gravity overflow structure 18.7 metres long. The spillway consists of a vertical upstream slope and an ogee-shaped crest and curved downstream slope. The crest is modified with a steel catwalk is located above the structure that is supported by four (4) concrete piers and three (3) steel pier stanchions. Varying levels of silt build-up can be found on the upstream side of the structure.

The spillway includes six (6) Type “C”, all-thread-bar grout anchors. All rock anchors are post-tensioned to a magnitude of a minimum of 60 % of their ultimate strength. Typical cross-sections of the dam are shown in Figure 3.2.

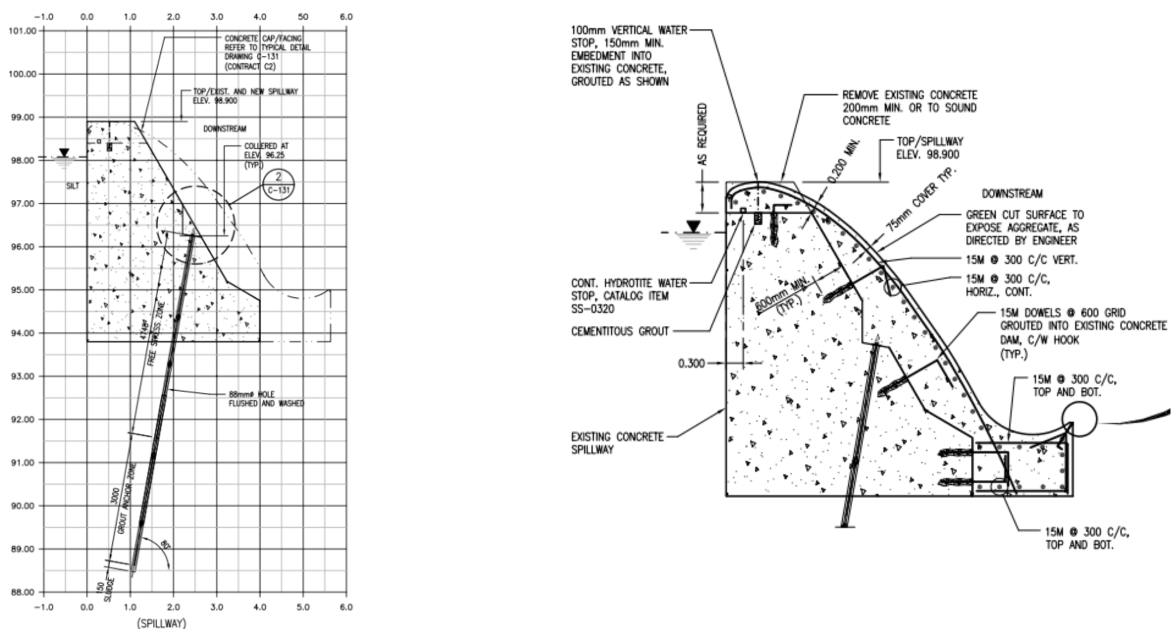


Figure 3.2 Typical Cross Section of the Spillway

The geometric properties used for the analysis are listed in Table 3.9, with rock anchor properties listed in Table 3.10. Material properties are presented in Table 3.11. Geometric and rock anchor properties were taken from DWG C-120 of the *Dam and Spillway Modifications* drawing set prepared by Mitchelmore Engineering Company Ltd. (Meco), dated January 24, 2007.

Table 3.9 Geometric Properties of the Spillway

Parameter		Value
Height of Wall		5.10 m
Base Width		5.63 m
Cross-Sectional Area		18.03 m^2
Location of Centroid from the Downstream Toe	$\bar{x}$ =	3.59 m
	$\bar{y}$ =	1.98 m
EDGM (PGA) (AEP_{1000})		0.046 g [1]

[1] NBCC, 2015

Table 3.10 Rock Anchor Properties in Spillway Section

Parameter	Section A
Type	Type C All-Thread Bar (Grout Anchor)
Diameter	32 mm
Spacing	3,000 mm c/c
Length	8.00 m
Embedment Angle	10°
Location of Anchor from the Downstream Toe	3.70 m
Post-Tension Load	500 kN

Table 3.11 Material Properties of the Spillway

Parameter	Value
Concrete Strength	25 MPa
Unit Weight of Concrete	23.5 kN/m^3
Unit Weight of Rock	25 kN/m^3
Unit Weight of Soil	18 kN/m^3
Unit Weight of Water	9.81 kN/m^3
Base-Rock Angle of Friction	45°

3.2.1 Results

Results of the sliding stability analysis are presented in Table 3.12. The spillway complies with the standards-based criteria in the CDA guidelines for “Usual”, “Unusual”, and “Extreme” loading conditions for concrete structures.

Table 3.12 Sliding Stability Analysis Results

Loading Condition	Required Factor of Safety	Calculated Factor of Safety	Pass/Fail
Usual	1.5	2.05	Pass
Usual (No Ice)	1.5	3.12	Pass
Unusual (Flood)	1.3	1.77	Pass
Unusual (Wave)	1.3	3.03	Pass
Extreme (Earthquake)	1.1	2.63	Pass

Results of the overturning analysis are presented in Table 3.13. The spillway complies with standards-based criteria for overturning stability in the CDA guidelines for “Usual”, “Unusual”, and “Extreme” loading conditions.

Table 3.13 Overturning Stability Analysis Results

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	0.94	0.83	Pass
Usual (No Ice)	0.94	-0.06	Pass
Unusual (Flood)	1.41	0.77	Pass
Unusual (Wave)	1.41	-0.07	Pass
Extreme (Earthquake)	2.82	0.06	Pass

3.2.2 Sensitivity Analysis – Bedrock Conditions

Sensitivity analysis was performed on the spillway to assess the significance of soil conditions beneath the section. The structure found on bedrock, but the condition of the underlying rock may vary. The potential failure mode was sliding effects due to fractured bedrock, reducing the friction angle between the concrete base and rock interface. The friction angle was incrementally reduced until there was indication that the structure did not meet the standards-based requirements for stability.

The critical failure mode in this case was sliding stability with results presented in Table 3.14. The critical state was a reduction in the base-rock friction angle to a magnitude of less than 36°, with the structure no longer meeting overturning criteria during “Usual” loading conditions with ice, and “Unusual” conditions during a flood.

Table 3.14 Sliding Stability Analysis Results with Varying Friction Angles

Loading Condition	Req’d FOS	Calc’d FOS (ϕ = 45°)	P/F	Calc’d FOS (ϕ = 40°)	P/F	Calc’d FOS (ϕ = 35°)	P/F
Usual	1.5	2.05	Pass	1.72	Pass	1.43	Fail
Usual (No Ice)	1.5	3.12	Pass	2.62	Pass	2.19	Pass
Unusual (Flood)	1.3	1.77	Pass	1.49	Pass	1.24	Fail
Unusual (Wave)	1.3	3.03	Pass	2.54	Pass	2.12	Pass
Extreme (Earthquake)	1.1	2.63	Pass	2.21	Pass	1.84	Pass

4.0 SUMMARY & RECOMMENDATIONS

Areas of the right gravity dam that were not post-tensioned in 2008 remain at risk of deformation. There were no observations to indicate the area is experiencing deformation to-date.

The analysis concluded that the dam and spillway comply with standards-based criteria for sliding and overturning during "Usual", "Unusual", and "Extreme" events, as described in the CDA guidelines. Sensitivity analysis determined that the structure no longer meet criteria if rock anchors in Section A experience relaxation of load by about 2.5%, or to 58% of ultimate load. There were no observations to indicate relaxation is occurring.

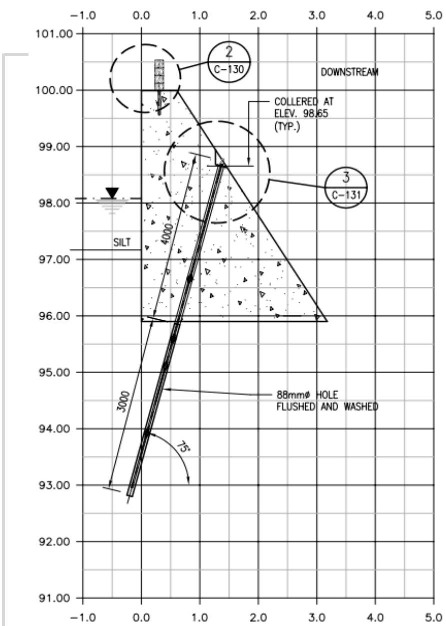
Analysis of the Factorydale Spillway determined that the structure complies with standards-based criteria for sliding and overturning during "Usual", "Unusual", and "Extreme" scenarios outlined in the CDA guidelines. Sensitivity analysis of the spillway concluded that the structure no longer complies with criteria if the condition of the concrete-bedrock interface deteriorates such that a friction angle of less than 36° develops. Foundations interface friction angles may deteriorate due to weathering, chemical seepage, overstressing. There were no observations to indicate any of these conditions are present.

Attachment A

Stability Calculations

TAP101

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam



Purpose

General stability assessment

Assumptions

- Design section on drawings represent the actual section

Cell Legend

Input
Output

Value meets requirements (OK)

Value does not meet requirements/No go (NG)

Needs additional support (NAS)

References

1. National Building Code of Canada (NBCC 2015)
2. Canadian Dam Association Technical Bulletin: Structural Considerations for Dam Safety (SCDS)
3. Williams Form Engineering Corp. - Types of Ground Anchors
4. Factorydale Pond Dam - Dam and Spillway Modifications (Drawing Set) (Meco, 2007)

Prepared by:__________________ Date:____________ Manager:______________________

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam

Vertical Loads

D	- Dead Loads due to weight	Unit Weight of Concrete =	23.52	kN/m^3		
Sv	- Vertical load caused by soil	Unit Weight of Soil =	18	kN/m^3	30	degrees
U	- Hydrostatic Uplift Pressure	Unit Weight of Rock =	25	kN/m^3	45	degrees
L	- Live loads due to activity	Unit Weight of Water =	9.81	kN/m^3		
SL	- Uniform Surcharge loads	Wave Height (m) =	0.6	m		

Horizontal Thrust

0.33 $Rankine_{Activ}$

3 $Rankine_{Pass}$

0.5 $Rankine_{AR}$

H	- Maximum Normal Headwater minus the concurrent Tailwater
Hf	- Maximum Flood Headwater minus the concurrent Tailwater
I	- Static and Dynamic force created by ice
Sh	- Horizontal Active thrust caused by soil
Q	- Earthquake Load

Types of Analysis

Load Conditions	Load Combinations	Sliding FoS	Overturning
Usual	$D+H+I+S_h+U$	1.5	one-third
Usual (No Ice)	$D+H_f+S_h+U$	1.5	one-third
Unusual (Flood)	$D+H_f+S_h+U_f$	1.3	one-half
Unusual (Temperature)	$D+H+I+S_h+U+T$	1.3	one-half
Extreme (Earthquake)	$D+H+S_h+Q+U_Q$	1.1	Full Base

Sliding Stability

$$FS = \frac{P_v \tan(\vartheta_{b-s})}{\Sigma P_h}$$

Where:
FS = Factor of safety
ϑ_{b-s} = Base-soil angle of friction
Pv = Sum of vertical forces (kN)
Ph = Sum of horizontal forces (kN)

Overturning Stability

$$x_r = \frac{\Sigma M_r - \Sigma M_o}{\Sigma P_v}$$

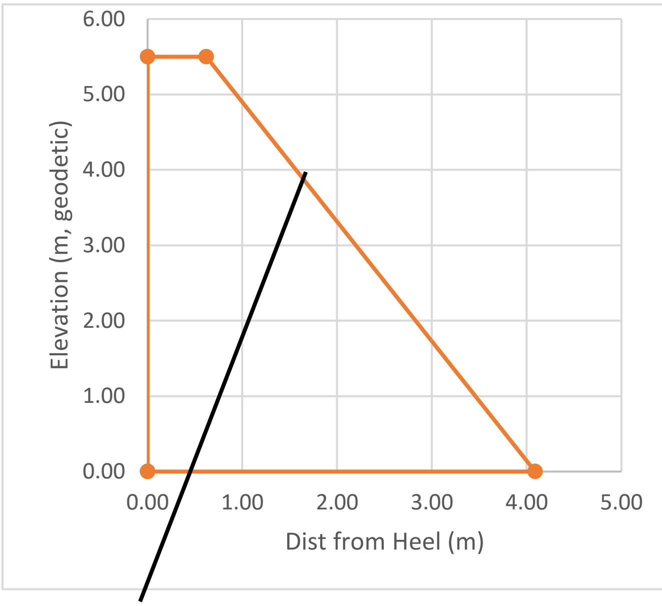
Where:
xr = Location of resultant force (m)
Mr = Resistant moment (kN·m)
Mo = Overturning moment (kN·m)
Pv = Sum of vertical forces (kN)

AND

$$e = \frac{B}{2} - x_r$$

Where:
e = Eccentricity (m)
B = Base width of dam (m)
xr = Location of resultant force (m)

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam



X (m)	Y (m)
0.00	0.00
0.00	5.50
0.62	5.50
4.09	0.00
0.00	0.00
Centre of Gravity (m) **1.39**	**2.07**

Area of Mass (m^2)	12.95
Base Width (m)	4.09

Normal Operating Level	NOL	68.71	1.63
Maximum Flood Level	MFL	69.93	2.04
Normal Tailwater Level	TWN	63.81	0.00
Flood Tailwater Level	TWF	64.75	0.31
Wave Height	WH	0.60	5.12
Soil $_{\text{Upstream}}$	S1H	66.71	0.97
Soil $_{\text{Downstream}}$	S2H	63.81	0.00

Top of Dam	CL	69.31	m
Base Level	BL	63.81	m
Height of wall	H =	5.50	m
Base width	B =	4.09	m

$\text{Uplift}_{\text{NOL}}$	2.7
$\text{Uplift}_{\text{MFL}}$	2.7

Assumptions

Simplified Section is Representative

Uplift is 100% Hydrostatic Pressure at the upstream and downstream toe

Sliding Friction Angle = 45 degrees

Earthquake Hydrostatic Uplift equals normal uplift

Ice Thickness is 0.6 metres

Compressive Strength of Concrete 25 Mpa

Threshold Shear Strength of Concrete/rock Interface is assumed to be 1/2 the full strength for Post-Earthquake and residual conditions.

PT Anchor (25mm dia. Type B) Tension	340.2
Angle (°)	15.0
Dist From Toe (m)	3.73
Spacing (m)	3
EDGM (PGA) (1/1000yr)	0.046

(1) Usual Load Condition

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-48.1	-117.8	-192.4
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-23.8	-11.5	-11.1
$Soil_{DownStream}$	At Rest	0.0	0.0	0.0
Ice	Horizontal		-73.0	-335.8
Anchor	Horizontal		29.4	0.0
Sum Normal Horiz			**-172.9**	**-539.3**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	12.95	74.5	304.6	822.4
Mass (Pier)	1.08	6.2	5.8	19.5
Anchor			109.5	408.6
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-48.1	-98.3	-268.0
Normal			**321.7**	**982.4**
FoS Sliding	1.86		ΣMo	807.3
			ΣMr	1250.4
			Resultant (m)	1.38
			Eccentricity (e)	0.67
		+/-	e_{max}	0.68

(1a) Usual Load Condition (No Ice)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-48.1	-117.8	-192.4
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-23.8	-11.5	-11.1
$Soil_{DownStream}$	At-Rest	0.0	0.0	0.0
Anchor	Horizontal		29.4	0.0
Sum Normal Horiz			**-99.9**	**-203.5**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	12.95	74.5	304.6	822.4
Mass (Pier)	1.08	6.2	5.8	19.5
Anchor			109.5	408.6
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-48.1	-98.3	-268.0
Normal			**321.7**	**982.4**
FoS Sliding	3.22		ΣMo	471.5
			ΣMr	1250.4
			Resultant (m)	2.42
			Eccentricity (e)	-0.38
		+/-	e_{max}	0.68

(2) Unusual Load Condition (IDF)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Unusual H_2O	Hydrostatic_{MFL}	-60.0	-183.5	-374.2
Unusual H_2O	Hydrostatic_{TWF}	9.2	4.3	1.3
$\text{Soil}_{Upstream}$	Active	-23.8	-11.5	-11.1
$\text{Soil}_{DownStream}$	Active	0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest	0.0	0.0	0.0
Anchor	Horizontal		29.4	0.0
Sum Unusual Horiz			**-161.4**	**-384.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	12.95	74.5	304.6	822.4
Mass (Pier)	1.08	6.2	5.8	19.5
Anchor			109.5	408.6
$\text{Uplift}_{Normal(TW)}$		-9.2	-37.6	-102.5
Uplift_{Normal}		-50.8	-103.9	-283.4
Unusual			**278.5**	**864.6**
FoS Sliding	1.73		ΣMo	771.2
			ΣMr	1251.8
			Resultant (m)	1.73
			Eccentricity (e)	0.32
		+/-	e_{max}	1.36

(3) Unusual Load Condition (Temperature / Wave)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	Hydrostatic_{NOL}	-48.1	-117.8	-192.4
Normal H_2O	Hydrostatic_{TWN}	0.00	0.0	0.0
Normal H_2O	Hydrodymanic	-7.1	-4.2	-20.8
$\text{Soil}_{Upstream}$	Active	-23.8	-11.5	11.1
$\text{Soil}_{DownStream}$	Active	0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest	0.0	0.0	0.0
Anchor	Horizontal		29.4	0.0
Sum Normal Horiz			**-104.1**	**-202.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	12.95	74.5	304.6	822.4
Mass (Pier)	1.08	6.2	5.8	19.5
Anchor			109.5	408.6
$\text{Uplift}_{Normal(TW)}$		0.0	0.0	0.0
Uplift_{Normal}		-48.1	-98.3	-268.0
Normal			**321.7**	**982.4**
FoS Sliding	3.09		ΣMo	481.2
			ΣMr	1261.5
			Resultant (m)	2.43
			Eccentricity (e)	-0.38
		+/-	e_{max}	1.36

(4) Extreme Load Condition (Seismic)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$		-48.1	-117.8	-192.4
Normal H_2O	$Hydrostatic_{TWN}$		0.0	0.0	0.0
$Soil_{Upstream}$	Active		-23.8	-11.5	-11.1
$Soil_{DownStream}$	At Rest		0.0	0.0	0.0
Anchor	Horizontal			29.4	0.0
Pseudostatic		12.95	-304.6	-14.0	-29.1
Westergaard			-1.00	-4.9	-9.6
Sum Seismic Horiz				**-118.8**	**-242.1**

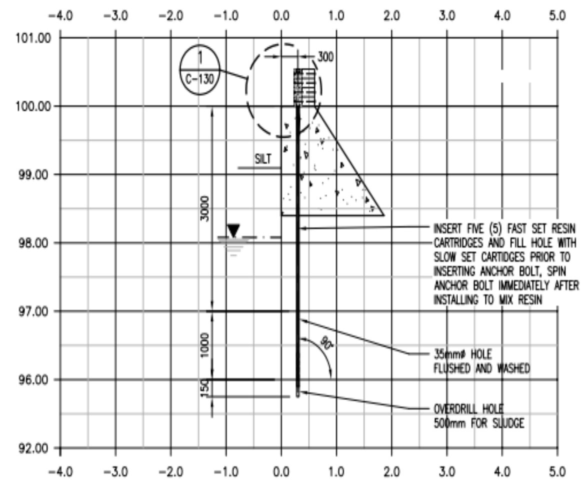
Sum Seismic Horiz

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	12.95	74.5	304.6	822.4
Mass (Pier)	1.08	6.2	5.8	19.5
Anchor			109.5	408.6
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-48.1	-98.3	-268.0
Other				
Sum Seismic Vert			**321.7**	**982.4**
FoS Sliding	2.71		ΣMo	510.2
			ΣMr	1250.4
			Resultant (m)	2.30
			Eccentricity (e)	-0.26
		+/-	e_{max}	2.05

Meco

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam

Load Condition	Horizontal		Vertical		Sliding Fos			Overturning		
	Load	Moment	Load	Moment	Req'd	Actual	Result	Req'd	Actual	Result
Usual	-172.9	-539.3	321.7	982.4	1.5	1.86	OK	0.68	0.67	OK
Usual (No Ice)	-99.9	-203.5	321.7	982.4	1.5	3.22	OK	0.68	-0.38	OK
Unusual (Flood)	-161.4	-384.0	278.5	864.6	1.3	1.73	OK	1.36	0.32	OK
Unusual (Wave)	-104.1	-202.0	321.7	982.4	1.3	3.09	OK	1.36	-0.38	OK
Extreme (Earthquake)	-118.8	-242.1	321.7	982.4	1.1	2.71	OK	2.05	-0.26	OK



Purpose

General stability assessment

Assumptions

- Design section on drawings represent the actual section

Cell Legend

Input

Output

Value meets requirements (OK)

Value does not meet requirements/No go (NG)

Needs additional support (NAS)

References

1. National Building Code of Canada (NBCC 2015)
2. Canadian Dam Association Technical Bulletin: Structural Considerations for Dam Safety (SCDS)
3. Williams Form Engineering Corp. - Types of Ground Anchors
4. Factorydale Pond Dam - Dam and Spillway Modifications (Drawing Set) (Meco, 2007)

Prepared by:____________________ Date:_____________ Manager:________________________

Vertical Loads

D	- Dead Loads due to weight	Unit Weight of Concrete =	23.52	kN/m^3		
Sv	- Vertical load caused by soil	Unit Weight of Soil =	18	kN/m^3	30	degrees
U	- Hydrostatic Uplift Pressure	Unit Weight of Rock =	25	kN/m^3	45	degrees
L	- Live loads due to activity	Unit Weight of Water =	9.81	kN/m^3		
SL	- Uniform Surcharge loads	Wave Height (m) =	0.6	m		

Horizontal Thrust

0.33 $\text{Rankine}_{\text{Activ}}$

H	- Maximum Normal Headwater minus the concurrent Tailwater	3 $\text{Rankine}_{\text{Pass}}$
Hf	- Maximum Flood Headwater minus the concurrent Tailwater	0.5 $\text{Rankine}_{\text{AR}}$
I	- Static and Dynamic force created by ice	
Sh	- Horizontal Active thrust caused by soil	
Q	- Earthquake Load	

Types of Analysis

Load Conditions	Load Combinations	Sliding FoS	Overturning
Usual	$D+H+I+S_h+U$	1.5	one-third
Usual (No Ice)	$D+H_f+S_h+U$	1.5	one-third
Unusual (Flood)	$D+H_f+S_h+U_f$	1.3	one-half
Unusual (Temperature)	$D+H+I+S_h+U+T$	1.3	one-half
Extreme (Earthquake)	$D+H+S_h+Q+U_Q$	1.1	Full Base

Sliding Stability

$$FS = \frac{P_v \tan(\vartheta_{b-s})}{\Sigma P_h}$$

Where:
FS = Factor of safety
ϑ_{b-s} = Base-soil angle of friction
Pv = Sum of vertical forces (kN)
Ph = Sum of horizontal forces (kN)

Overturning Stability

$$x_r = \frac{\Sigma M_r - \Sigma M_o}{\Sigma P_v}$$

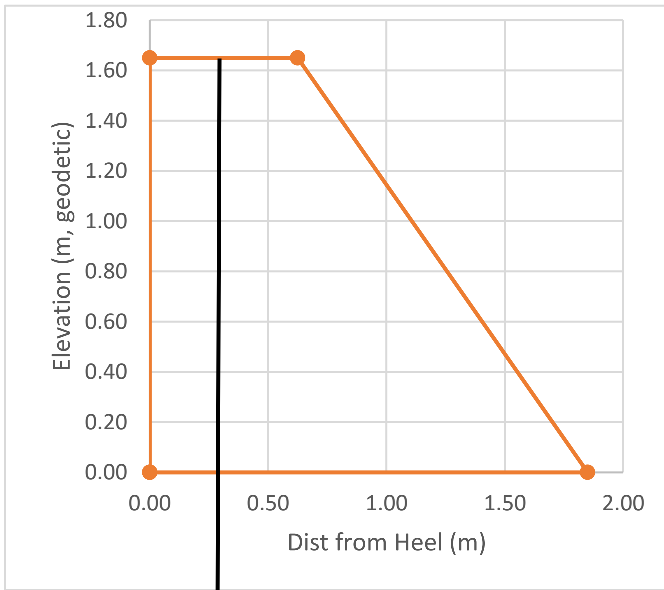
Where:
xr = Location of resultant force (m)
Mr = Resistant moment (kN·m)
Mo = Overturning moment (kN·m)
Pv = Sum of vertical forces (kN)

AND

$$e = \frac{B}{2} - x_r$$

Where:
e = Eccentricity (m)
B = Base width of dam (m)
xr = Location of resultant force (m)

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam



X (m)	Y (m)
0.00	0.00
0.00	1.65
0.63	1.65
1.85	0.00
0.00	0.00

Centre of Gravity (m)	**0.67**	**0.69**
Area of Mass (m^2)	2.04	
Base Width (m)	1.85	

Normal Operating Level	NOL	68.71	0.35
Maximum Flood Level	MFL	69.93	0.76
Normal Tailwater Level	TWN	67.66	0.00
Flood Tailwater Level	TWF	67.71	0.02
Wave Height	WH	0.60	1.28
Soil $_{\text{Upstream}}$	S1H	68.41	0.25
Soil $_{\text{Downstream}}$	S2H	67.66	0.00

Top of Dam	CL	69.31	m
Base Level	BL	67.66	m
Height of wall	H =	1.65	m
Base width	B =	1.85	m

$\text{Uplift}_{\text{NOL}}$	1.2
$\text{Uplift}_{\text{MFL}}$	1.2

Assumptions

Simplified Section is Representative

Uplift is 100% Hydrostatic Pressure at the upstream and downstream toe

Sliding Friction Angle = 45 degrees

Earthquake Hydrostatic Uplift equals normal uplift

Ice Thickness is 0.6 metres

Compressive Strength of Concrete 25 Mpa

Threshold Shear Strength of Concrete/rock Interface is assumed to be 1/2 the full strength for Post-Earthquake and residual conditions.

PT Anchor (25mm dia. Type B) Tension	300
Angle (°)	0.0
Dist From Toe (m)	1.55
Spacing (m)	3
EDGM (PGA) (1/1000yr)	0.046

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam

(1) Usual Load Condition

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-10.3	-5.4	-1.9
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-6.1	-0.8	-0.2
$Soil_{DownStream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At Rest	0.0	0.0	0.0
Ice	Horizontal		-73.0	-54.7
Sum Normal Horiz			**-79.2**	**-56.8**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	2.04	26.0	48.0	56.7
Anchor			100.0	118.1
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-10.3	-9.5	-11.8
Normal			**138.5**	**163.0**
FoS Sliding	1.75		ΣMo	68.6
			ΣMr	174.8
			Resultant (m)	0.77
			Eccentricity (e)	0.16
		+/-	e_{max}	0.31

(1a) Usual Load Condition (No Ice)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-10.3	-5.4	-1.9
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-6.1	-0.8	-0.2
$Soil_{DownStream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At-Rest	0.0	0.0	0.0
Ice	Horizontal		0.0	0.0
Sum Normal Horiz			**-6.2**	**-2.1**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	2.04	26.0	48.0	56.7
Anchor	0		100.0	118.1
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-10.3	-9.5	-11.8
Normal			**138.5**	**163.0**
FoS Sliding	22.43		ΣMo	13.8
			ΣMr	174.8
			Resultant (m)	1.16
			Eccentricity (e)	-0.24
		+/-	e_{max}	0.31

(2) Unusual Load Condition (IDF)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Unusual H_2O	$Hydrostatic_{MFL}$	-22.2	-25.2	-19.0
Unusual H_2O	$Hydrostatic_{TWF}$	0.5	0.0	0.0
$Soil_{Upstream}$	Active	-6.1	-0.8	-0.2
$Soil_{DownStream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At Rest	0.0	0.0	
Other				
Sum Unusual Horiz			**-26.0**	**-19.2**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	2.04	26.0	48.0	56.7
Anchor			100.0	118.1
$Uplift_{Normal(TW)}$		-0.5	-0.9	-1.1
$Uplift_{Normal}$		-21.7	-20.1	-24.8
Other				
Unusual			**127.0**	**148.8**
FoS Sliding	4.89		ΣMo	45.2
			ΣMr	174.8
			Resultant (m)	1.02
			Eccentricity (e)	-0.10
		+/-	e_{max}	0.62

(3) Unusual Load Condition (Temperature / Wave)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-10.3	-5.4	-1.9
Normal H_2O	$Hydrostatic_{TWN}$	0.00	0.0	0.0
Normal H_2O	Hydrodymanic	-7.1	-4.2	-4.4
$Soil_{Upstream}$	Active	-6.1	-0.8	0.2
$Soil_{DownStream}$	Active	0.0	0.0	0.0
	At Rest	0.0	0.0	
Ice	Horizontal		0.0	0.0
Other				
Sum Normal Horiz			**-10.4**	**-6.2**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	2.04	26.0	48.0	56.7
Anchor			100.0	118.1
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal}$		-10.3	-9.5	-11.8
Other				
Normal			**138.5**	**163.0**
FoS Sliding	13.30		ΣMo	18.1
			ΣMr	175.0
			Resultant (m)	1.13
			Eccentricity (e)	-0.21
		+/-	e_{max}	0.62

(4) Extreme Load Condition (Seismic)

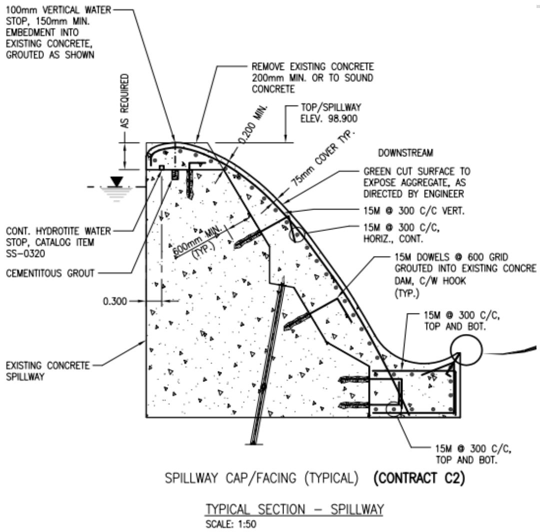
Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	Hydrostatic_{NOL}		-10.3	-5.4	-1.9
Normal H_2O	Hydrostatic_{TWN}		0.0	0.0	0.0
$\text{Soil}_{Upstream}$	Active		-6.1	-0.8	-0.2
$\text{Soil}_{DownStream}$	Active		0.0		
	At Rest		0.0	0.0	0.0
	Passive		0.0		
Pseudostatic		2.04	-48.0	-2.2	-1.5
Westergaard			-0.21	-0.2	-0.1
Sum Seismic Horiz				**-8.6**	**-3.7**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		2.04	26.0	48.0	56.7
Anchor				100.0	118.1
$\text{Uplift}_{Normal(TW)}$			0.0	0.0	0.0
Uplift_{Normal}			-10.3	-9.5	-11.8
Other					
Sum Seismic Vert				**138.5**	**163.0**
FoS Sliding		16.09		ΣMo	15.5
				ΣMr	174.8
				Resultant (m)	1.15
				Eccentricity (e)	-0.23
			+/-	e_{max}	0.93

Technical Analysis Procedure
GRAVITY DAM STABILITY
Factorydale Dam

Load Condition	Horizontal		Vertical		Sliding Fos			Overturning		
	Load	Moment	Load	Moment	Req'd	Actual	Result	Req'd	Actual	Result
Usual	-79.2	-56.8	138.5	163.0	1.5	1.75	OK	0.31	0.16	OK
Usual (No Ice)	-6.2	-2.1	138.5	163.0	1.5	22.43	OK	0.31	-0.24	OK
Unusual (Flood)	-26.0	-19.2	127.0	148.8	1.3	4.89	OK	0.62	-0.10	OK
Unusual (Wave)	-10.4	-6.2	138.5	163.0	1.3	13.30	OK	0.62	-0.21	OK
Extreme (Earthquake)	-8.6	-3.7	138.5	163.0	1.1	16.09	OK	0.93	-0.23	OK

Technical Analysis Procedure
OGEE SPILLWAY STABILITY
Factorydale Dam Spillway



Purpose

General stability assessment

Assumptions

- Design section on drawings represent the actual section

Cell Legend

Input
Output

Value meets requirements (OK)
Value does not meet requirements/No go (NG)
Needs additional support (NAS)

References

1. National Building Code of Canada (NBCC)
2. Canadian Dam Association Technical Bulletin: Structural Considerations for Dam Safety (SCDS)
3. Williams Form Engineering Corp. - Types of Ground Anchors
4. Factorydale Pond Dam - Dam and Spillway Modifications (Drawing Set) (Meco, 2007)

Prepared by:________________ Date:__________ Manager:__________________

Technical Analysis Procedure
OGEE SPILLWAY STABILITY
Factorydale Spillway

Vertical Loads

D	- Dead Loads due to weight	Unit Weight of Concrete =	23.52	kN/m^3		
Sv	- Vertical load caused by soil	Unit Weight of Soil =	18	kN/m^3	30	degrees
U	- Hydrostatic Uplift Pressure	Unit Weight of Rock =	25	kN/m^3	45	degrees
L	- Live loads due to activity	Unit Weight of Water =	9.81	kN/m^3		
SL	- Uniform Surcharge loads	Wave Height (m) =	0.6	m		

Horizontal Thrust

0.33 $\text{Rankine}_{\text{Activ}}$

3 $\text{Rankine}_{\text{Pass}}$

0.5 $\text{Rankine}_{\text{AR}}$

H	- Maximum Normal Headwater minus the concurrent Tailwater
Hf	- Maximum Flood Headwater minus the concurrent Tailwater
I	- Static and Dynamic force created by ice
Sh	- Horizontal Active thrust caused by soil
Q	- Earthquake Load

Types of Analysis

Load Conditions	Load Combinations	Sliding FoS	Overturning
Usual	$D+H+I+S_h+U$	1.5	one-third
Usual (No Ice)	$D+H_f+S_h+U$	1.5	one-third
Unusual (Flood)	$D+H_f+S_h+U_f$	1.3	one-half
Unusual (Temperature)	$D+H+I+S_h+U+T$	1.3	one-half
Extreme (Earthquake)	$D+H+S_h+Q+U_Q$	1.1	Full Base

Sliding Stability

$$FS = \frac{P_v \tan(\vartheta_{b-s})}{\Sigma P_h}$$

Where:
FS = Factor of safety
ϑ_{b-s} = Base-soil angle of friction
Pv = Sum of vertical forces (kN)
Ph = Sum of horizontal forces (kN)

Overturning Stability

$$x_r = \frac{\Sigma M_r - \Sigma M_o}{\Sigma P_v}$$

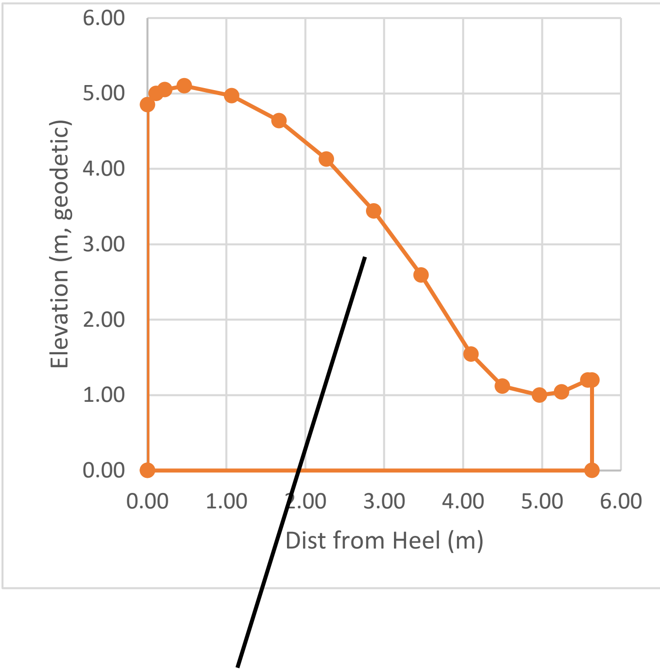
Where:
xr = Location of resultant force (m)
Mr = Resistant moment (kN·m)
Mo = Overturning moment (kN·m)
Pv = Sum of vertical forces (kN)

AND

$$e = \frac{B}{2} - x_r$$

Where:
e = Eccentricity (m)
B = Base width of dam (m)
xr = Location of resultant force (m)

Technical Analysis Procedure
OGEE SPILLWAY STABILITY
Factorydale Spillway



X (m)	Y (m)				
0.00	0.00	Normal Operating Level	NOL	68.71	1.87
0.00	4.85	Maximum Flood Level	MFL	69.93	2.27
0.11	5.00	Normal Tailwater Level	TWN	63.11	0.00
0.22	5.05	Flood Tailwater Level	TWF	64.80	0.56
0.47	5.10	Wave Height	WH	0.60	5.82
1.07	4.97	Soil $_{Upstream}$	S1H	66.31	1.07
1.67	4.64	Soil $_{Downstream}$	S2H	63.11	0.00
2.27	4.13				
2.87	3.44	Top of Dam	CL	68.21	m
3.47	2.59	Base Level	BL	63.11	m
4.10	1.54	Height of wall	H =	5.10	m
4.50	1.12	Base width	B =	5.63	m
4.97	1.00		$Uplift_{NOL}$		3.8
5.25	1.04		$Uplift_{MFL}$		3.8
5.58	1.20				
5.63	1.20				
5.63	0.00				
0.00	0.00				

Centre of Gravity (m)	**2.04**	**1.98**
Area of Mass (m^2)	18.03	
Base Width (m)	5.63	

Assumptions
Simplified Section is Representative
Uplift is 100% Hydrostatic Pressure at the upstream and downstream toe
Sliding Friction Angle = 45 degrees
Earthquake Hydrostatic Uplift equals normal uplift
Ice Thickness is 0.6 metres
Compressive Strength of Concrete 25 Mpa
Threshold Shear Strength of Concrete/rock Interface is assumed to be 1/2 the full strength for Post-Earthquake and residual conditions.

PT Anchor (32mm dia. Type C) Tension	500
Angle (°)	10.0
Dist From Toe (m)	3.7
Spacing (m)	3
EDGM (PGA) (1/1000yr)	0.046

(1) Usual Load Condition

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-54.9	-153.8	-287.1
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-26.2	-14.0	-14.9
$Soil_{DownStream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At Rest	0.0	0.0	0.0
Ice	Horizontal		-73.0	-386.9
Anchor	Horizontal		28.9	0.0
Sum Normal Horiz			**-211.9**	**-688.9**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	18.03	75.3	424.0	1521.6
Anchor			164.1	607.3
$Uplift_{Normal(TW)}$		0.0	0.2	0.6
$Uplift_{Normal}$		-55.0	-154.8	-581.4
Ice	Vertical		0.0	0.0
Normal			**433.5**	**1548.1**
FoS Sliding	2.05		ΣMo	1270.3
			ΣMr	2129.5
			Resultant (m)	1.98
			Eccentricity (e)	0.83
		+/-	e_{max}	0.94

(1a) Usual Load Condition (No Ice)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-54.9	-153.8	-287.1
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	-26.2	-14.0	-14.9
$Soil_{DownStream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At-Rest	0.0	0.0	0.0
Anchor	Horizontal		28.9	0.0
Sum Normal Horiz			**-138.9**	**-302.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	18.03	75.3	424.0	1521.6
Anchor	0		164.1	607.3
$Uplift_{Normal(TW)}$		0.0	0.2	0.6
$Uplift_{Normal}$		-55.0	-154.8	-581.4
Normal			**433.5**	**1548.1**
FoS Sliding	3.12		ΣMo	883.4
			ΣMr	2129.5
			Resultant (m)	2.87
			Eccentricity (e)	-0.06
		+/-	e_{max}	0.94

(2) Unusual Load Condition (IDF)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Unusual H_2O	$Hydrostatic_{MFL}$		-66.9	-227.9	-518.0
Unusual H_2O	$Hydrostatic_{TWF}$		16.5	13.9	7.8
$Soil_{Upstream}$	Active		-26.2	-14.0	-14.9
$Soil_{DownStream}$	Active		0.0	0.0	0.0
$Soil_{DownStream}$	At Rest		0.0	0.0	
Anchor	Horizontal			28.9	0.0
Sum Unusual Horiz				**-199.1**	**-525.1**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		18.03	75.3	424.0	1521.6
Anchor				164.1	607.3
$Uplift_{Normal(TW)}$			-16.5	-93.0	-349.3
$Uplift_{Normal}$			-50.4	-141.9	-532.7
Other					
Unusual				**353.3**	**1246.9**
FoS Sliding		1.77		ΣMo	1414.8
				ΣMr	2136.7
				Resultant (m)	2.04
				Eccentricity (*e*)	0.77
			+/-	e_{max}	1.41

(3) Unusual Load Condition (Temperature / Wave)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$		-54.9	-153.8	-287.1
Normal H_2O	$Hydrostatic_{TWN}$		-0.03	0.0	0.0
Normal H_2O	Hydrodymanic		-7.1	-4.2	-23.7
$Soil_{Upstream}$	Active		-26.2	-14.0	14.9
$Soil_{DownStream}$	Active		0.0	0.0	0.0
	At Rest		0.0	0.0	
Anchor	Horizontal			28.9	0.0
Sum Normal Horiz				**-143.1**	**-296.0**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		18.03	75.3	424.0	1521.6
Anchor				164.1	607.3
$Uplift_{Normal(TW)}$			0.0	0.2	0.6
$Uplift_{Normal}$			-55.0	-154.8	-581.4
Other					
Normal				**433.5**	**1548.1**
FoS Sliding		3.03		ΣMo	892.2
				ΣMr	2144.4
				Resultant (m)	2.89
				Eccentricity (*e*)	-0.07
			+/-	e_{max}	1.88

(4) Extreme Load Condition (Seismic)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$		-54.9	-153.8	-287.1
Normal H_2O	$Hydrostatic_{TWN}$		0.0	0.0	0.0
$Soil_{Upstream}$	Active		-26.2	-14.0	-14.9
$Soil_{DownStream}$	Active		0.0		
	At Rest		0.0	0.0	0.0
	Passive		0.0		
Anchor	Horizontal			28.9	0.0
Pseudostatic		18.03	-424.0	-19.5	-38.6
Westergaard			-1.14	-6.4	-14.3
Sum Seismic Horiz				**-164.8**	**-355.0**

Sum Seismic Horiz

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		18.03	75.3	424.0	1521.6
Anchor				164.1	607.3
$Uplift_{Normal(TW)}$			0.0	0.2	0.6
$Uplift_{Normal}$			-55.0	-154.8	-581.4
Other					
Sum Seismic Vert				**433.5**	**1548.1**
FoS Sliding		2.63		ΣMo	936.3
				ΣMr	2129.5
				Resultant (m)	2.75
				Eccentricity (e)	0.06
			+/-	e_{max}	2.82

Load Condition	Horizontal		Vertical		Sliding Fos			Overturning		
	Load	Moment	Load	Moment	Req'd	Actual	Result	Req'd	Actual	Result
Usual	-211.9	-688.9	433.5	1548.1	1.5	2.05	OK	0.94	0.83	OK
Usual (No Ice)	-138.9	-302.0	433.5	1548.1	1.5	3.12	OK	0.94	-0.06	OK
Unusual (Flood)	-199.1	-525.1	353.3	1246.9	1.3	1.77	OK	1.41	0.77	OK
Unusual (Wave)	-143.1	-296.0	433.5	1548.1	1.3	3.03	OK	1.41	-0.07	OK
Extreme (Earthquake)	-164.8	-355.0	433.5	1548.1	1.1	2.63	OK	2.82	0.06	OK

APPENDIX E

Glossary of Terms

GLOSSARY AND DEFINITIONS

Acceptable risk - The level of risk (the combination of the probability and the consequence of a specified hazardous event) which the public are prepared to accept without further management. Acceptability of risk may be reflected in government regulations.

Annual Exceedance Probability (AEP) - Probability that an event of specified magnitude will be equalled or exceeded in any year.

Abutment - That part of the valley side or other supporting structure against which the dam is constructed.

Appurtenances - Structures and equipment on a project site, other than the dam itself. They include, but are not limited to, such facilities as intake towers, powerhouse structures, tunnels, canals, penstocks, low-level outlets, surge tanks and towers, gate hoist mechanisms and their supporting structures, and all critical water control and release facilities. Also included are mechanical and electrical control and standby power supply equipment located in the powerhouse or in remote control centres.

Base of dam - General foundation area of the lowest portion of the main body of a dam.

Catchment - Surface area which drains to a specific point, such as a reservoir; also known as the watershed or watershed area.

Consequences of dam failure - Impacts in the downstream as well as upstream areas of a dam resulting from failure of the dam or its appurtenances.

Consequence Category - Scale of adverse incremental consequences that would be caused by failure of a dam.

Dam - Barrier which is constructed for the purpose of enabling the storage or diversion of water, water containing any other substance, fluid waste or fluid tailings, providing that such barrier could impound 30,000 m3 or more and is 2.5 m or more in height. The height is measured vertically to the top of the barrier, as follows:

(i) from the natural bed of the stream or watercourse at the downstream toe of the barrier, in the case of a barrier across a stream or watercourse

(ii) from the lowest elevation at the outside limit of the barrier, in the case of a barrier that is not across a stream or watercourse

"Dam" is herein defined to include works (appurtenances) incidental to, necessary for, or in connection with, the barrier.

For purposes of these guidelines, this definition may be expanded to include "dams" under 2.5 m in height or which can impound less than 30,000 m3, if the consequences of failure would be unacceptable to the public, such as:

- Dams with erodible foundations where a breach could lower the reservoir more than 2.5 m.
- Dams retaining contaminated substances.

Dam safety inspection - An inspection of the dam to observe its condition. Inspections are carried out much more frequently than Dam Safety Reviews.

Dam Safety Review - Comprehensive formal review carried out at regular time intervals to determine whether an existing dam is safe, and if it is not safe, to determine required safety improvements.

Decommissioned dam – Dam that has reached the stage in its life cycle when both dam construction and the intended use of the dam have been permanently terminated in accordance with a decommissioning plan.

Emergency - In terms of dam operation, any condition which develops naturally or unexpectedly, endangers the integrity of the dam, upstream or downstream property or life, and requires immediate action.

Emergency Preparedness Plan (EPP) - A document that contains procedures for dealing with emergencies at the dam or its appurtenances; and includes communication directories and inundation maps showing upstream and downstream water levels and arrival times of floods.

Extreme event - Event which has a very low annual exceedance probability (AEP).

Extreme loads - The rare loadings imposed by extreme events such as large earthquakes, floods and landslides.

Failure of dam - In terms of structural integrity, the uncontrolled release of the contents of a reservoir through collapse of the dam or some part of it.

Full Supply Level (FSL) - Maximum normal operating water surface level of a reservoir.

Flowsliding - Flowsliding is the displacement of large masses of soil for several metres, due to the significant strength reductions associated with liquefaction.

Foundation - Rock and/or soil mass that forms a base for the structure, including its abutments.

Freeboard - Vertical distance between the water surface elevation and the lowest elevation of the top of the containment structure.

Hazard - Threat; condition, which may result from an external cause (e.g. earthquake or flood), with the potential for creating adverse consequences.

Incremental consequences of failure -
Incremental losses or damage which dam failure might inflict on upstream areas, downstream areas, or at the dam, over and above any losses which might have occurred for the same natural event or conditions, had the dam not failed.

Industrial dam - Dam, including foundations, water control structures and base of the basin, which is constructed to retain wastes from industrial operations.

Inflow Design Flood (IDF) -
Most severe inflow flood (volume, peak, shape, duration, timing) which a dam and its spillway must handle safely.

Inspection - See "Dam safety inspection"

Maximum Credible Earthquake (MCE) -
Largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or interpreted tectonic framework.

Maximum Design Earthquake (MDE) -
The earthquake that would result in the most severe ground motion which a dam structure must be able to endure without the uncontrolled release of water from the reservoir.

Normal Operating Level (NOL) –
Mean operating water level of a resevoir normally applies to a reservoir with the level minly controlled by an overflow spillway.

OMS Manual - Operation, Maintenance and Surveillance Manual, which documents procedures for safe operation, maintenance and surveillance of a dam.

Outlet works - Combination of intake structure, conduits, tunnels, flow controls and energy dissipation devices to allow the release of water from a dam.

Owner - Person or legal entity, including a company, organization, government unit, public utility, corporation or other entity which is responsible for the safety of the dam. The person or legal entity may either hold a government license to operate a dam or retain the legal property title on the dam site, dam and/or reservoir.

Probable Maximum Flood (PMF) -
Estimate of hypothetical flood (peak flow, volume and hydrograph shape) that is considered to be the most severe "reasonably possible" at a particular location and time of year, based on relatively comprehensive hydrometeorological analysis of critical runoff-producing precipitation (snowmelt if pertinent) and hydrologic factors favourable for maximum flood runoff.

Probable Maximum Precipitation (PMP) -
Greatest depth of precipitation for a given duration meteorologically possible for a given size storm area at a particular location at a particular time of year, with no

allowance made for long-term climatic trends. The PMP is an estimate of an upper physical bound to the precipitation that the atmosphere can produce.

Regulatory agency - Usually a government ministry, department, office or other unit of the national or provincial government entrusted by law or administrative act with the responsibility for the general supervision of the safe design, construction and operation of dams and reservoirs, as well as any entity to which all or part of the executive or operational tasks and functions have been delegated by legal power.

Reservoir - Body of water, fluid waste or fluid tailings which is impounded by one or more dams, inclusive of its shores and banks and of any facility or installation necessary for its operation.

Reservoir capacity - Total or gross storage capacity of the reservoir at full supply level.

Return period - Reciprocal of the annual exceedance probability (AEP).

Risk - Measure of the probability and severity of an adverse effect to health, property, or the environment. Risk is estimated by the mathematical expectation of the consequences of an adverse event occurring (i.e., the product of the probability of occurrence and the consequence"). See also "acceptable risk".

Safe dam - Dam which does not impose an unacceptable risk to people or property, and which meets safety criteria that are acceptable to the government, the engineering profession and the public.

Spillway - Weir, channel, conduit, tunnel, chute, gate or other structure designed to permit discharges from the reservoir.

Spillway crest - Uppermost portion of the spillway overflow section.

spillway design flow (SGF) –
The flow obtained when the inflow design flood (IDF) is routed through the reservoir and its spillway. For small reservoirs where the routing effect is negligable the SDF = IDF.

Tailings dam - Dam, including foundations, water control structures and base of the basin, which is constructed to retain tailings or other waste materials from mining operations.

Tailwater level - Level of water in the discharge channel immediately downstream of a dam.

Temporary suspension -
Suspension, either expected or unexpected, of advanced exploration, mining or mine production in accordance with a plan, during which time the site is monitored on a continuous basis by the owner and protective measures are in place.

Toe of dam - Junction of the downstream (or upstream) face of dam with the ground surface (foundation). Sometimes "heel" is used to define the upstream toe of a concrete gravity dam.

Top of dam - Minimum elevation of the uppermost surface of a dam proper, not taking into account any camber allowed for settlement, curbs, parapets, guard rails or other structures that are not a part of the main water-retaining structure. This elevation may be a roadway, walkway or the non-overflow section of a dam.

IR-29 APPENDIX 1

RETScreen Financial Analysis Report

Financial parameters

General

Fuel cost escalation rate		2%
Inflation rate	%	2%
Discount rate	%	9%
Reinvestment rate	%	9%
Project life	yr	50

Finance

Incentives and grants	$	0
Debt ratio	%	50%
Debt	$	1,236,250
Equity	$	1,236,250
Debt interest rate	%	4.5%
Debt term	yr	20
Debt payments	$/yr	95,038

Income tax analysis ☐

Annual savings and revenue

Electricity export revenue

Electricity exported to grid	MWh	1,551
Electricity export rate	$/MWh	131
Electricity export revenue	$	203,324
Electricity export escalation rate	%	2%

GHG reduction savings

Gross GHG reduction	tCO_2e/yr	1,039
Gross GHG reduction - 50 yrs	tCO_2e	51,956
GHG reduction savings	$	0

Other revenue (cost) ☐

Clean Energy (CE) production revenue ☐

Costs | Savings | Revenue

Initial costs

Initial cost	100%	$	2,472,500
Total initial costs	**100%**	**$**	**2,472,500**

Yearly cash flows - Year 1

Annual costs and debt payments

O&M costs (savings)	$	75,356
Debt payments - 20 yrs	$	95,038
Total annual costs	**$**	**170,394**

Annual savings and revenue

Electricity export revenue	$	203,324
GHG reduction savings	$	0
Total annual savings and revenue	**$**	**203,324**
Net yearly cash flow - Year 1	**$**	**32,930**

Financial viability

Pre-tax IRR - equity	%	7.5%
Pre-tax MIRR - equity	%	8.4%
Pre-tax IRR - assets	%	4.4%
Pre-tax MIRR - assets	%	6.9%
Simple payback	yr	19.3
Equity payback	yr	19.6
Net Present Value (NPV)	$	-306,627
Annual life cycle savings	$/yr	-27,973
Benefit-Cost (B-C) ratio		0.75
Debt service coverage		1.4
GHG reduction cost	$\$/tCO_2e$	59.29
Energy production cost	$/kWh	0.186

Yearly cash flows

Year	Pre-tax	Cumulative
#	$	$
0	-1,236,250	-1,236,250
1	35,490	-1,200,760
2	38,100	-1,162,660
3	40,763	-1,121,898
4	43,479	-1,078,419
5	46,249	-1,032,170
6	49,075	-983,095
7	51,957	-931,137
8	54,897	-876,240
9	57,896	-818,345
10	60,954	-757,390
11	64,074	-693,316
12	67,257	-626,059
13	70,502	-555,557
14	73,813	-481,743
15	77,190	-404,553
16	80,635	-323,918
17	84,148	-239,770
18	87,732	-152,038
19	91,387	-60,650
20	95,116	34,466
21	193,957	228,423
22	197,836	426,259
23	201,793	628,052
24	205,829	833,881
25	209,946	1,043,827
26	214,144	1,257,971
27	218,427	1,476,399
28	222,796	1,699,195
29	227,252	1,926,446
30	231,797	2,158,243
31	236,433	2,394,676
32	241,161	2,635,837
33	245,985	2,881,822
34	250,904	3,132,726

Yearly cash flows