Report on

2024 Dam Safety Review

Lake Geraldine Dam

Iqaluit, NU

Final | Revision 1 | February 17, 2025

Meco Project Number: 10612

2024 DAM SAFETY REVIEW

LAKE GERALDINE DAM

Prepared for

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EXECUTIVE SUMMARY

Lake Geraldine is the water supply for the City of Iqaluit. Lake Geraldne has a catchment area of 3.9 km^2 and relies on storage provided by Lake Geraldine Dam to sustain the water supply. The dam was originally built by the Department of Defence in the 1950's and has been raised four time since to the current size and location. The dam consists of a rockfill extension with a vertical concrete core wall and a concrete gravity spillway and short abutment section. The City of Iqaluit operates the reservoir and dam based on an approval from the Nunavut Water Board which requires the City maintain the dam such that is complies with the Canadain Dam Association (CDA) Dam Safety Guidelines (2013).

Lake Geraldine Dam is assessed as an Extreme Consequence dam based on the classification system in the CDA Guidelines. A failure of the dam is a loss of the water supply and hardship and life safety risk to the residents on Iqaluit. The dam is designed to safely discharge a probable maximum flood (PMF) and survive a 1 in 10,000 year earthquake. The dam will not overtop during the PMF but is susceptible to splash overtopping from a design wind event that could undermine the toe of the rockfill dam. The concrete dam is unlikely to be susceptible to undermining from wave splash. **Modifications to the crest of the Rockfill dam are recommended to mitigate the risk of wave ovetopping.**

The concrete gravity spillway is constructed on permanently frozed bedrock. A geotechnical investigation in 2019 observed the bedrock as fractured and broken. Stability analysis of the spillway using historical construction records to develop foundation engineering parameters indicate the spillway complies with standards-based criteria for sliding and overturning. Unknown risks related to loss of permafrost from climate change, and potential for degradation of the foundation interface was identified for the spillway. **Additional monitoring is recommended for pore pressure, temperature and displacement at the spillway.**

The physical condition of the rockfill dam and concrete spillway is Good. There are minor deficiencies that can be adressed through regular maintenance. Surveillance is adequate, but can be improved with staff training and more frequent informal inspections. Instrumentation for monitoring ground temperatures and porewater pressures are not being monitored. **A recommendation is provided to establish performance monitoring of existing instrumentation, including parsing data and design of a reporting dashboard.**

There is a dam safety management safety system (DSMS) in place for Lake Geraldine Dam but implementation to date has been limited. An external dam safety consultant was engaged from 2021 to 2023 to perform tasks and activities associated with the DSMS. An attempt to extend the program in 2024 was unsuccessful. There are outstanding tasks related to design and surveillance that can be enhanced with more accountability with the City administration, coupled with independent professional advisory support. **A recommendation is provided to integrate the DSMS with an existing compliance-based management system at the City of Iqaluit. The City should extend efforts to retain a dam asfety professional with technical aspsects of the DSMS, review of instrumentation data, and general engineering, inclusive of training.**

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1 INTRODUCTION

Acting on the authorization of the City of Iqaluit (Iqaluit), Mitchelmore Consulting International Ltd., (Meco) has performed the 2024 Dam Safety Review (DSR) of Lake Geraldine Dam, located near Iqaluit, NU. The work was completed in conformance with Service Contract SC001430 and Meco proposal 10612 submitted September 6, 2024.

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. The Canadian Dam Association (CDA) *Dam Safety Guidelines* recommend a DSR cover all aspects required to demonstrate that;

- The dam is safe, operated safely, and maintained in a safe condition, and
- Surveillance is adequate to detect any developing safety problems.

1.1 DISCLAIMER

This report has been prepared for City of Iqaluit by Mitchelmore Consulting International Ltd. (Meco) as per Meco Proposal 10612 on September 6, 2024, subject to the following limitations, qualifications and disclaimers:

- The report is intended for the exclusive use of City of Iqaluit and may be shared with the Nunavut Water 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 Lake Geraldine 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.
- Information supplied by City of Iqaluit or third parties for use in this report has not been verified by Meco unless stated otherwise.
- Meco disclaims all liability for the use and interpretation of this report by any third party.

2 System Description And Background

Lake Geraldine is the water supply for the City of Iqaluit. The lake retained by Lake Geraldine Dam and forms the headwater of a small watershed. The reservoir and dam are located above the City of Iqaluit as shown in Figure 2.1.

Figure 2.1 Location of Dam

The current layout for the dam along with naming conventions in the various reports is shown in Figure 2.2. The dam is a hybrid structure with a concrete gravity spillway and dam section coupled with a concrete core rockfill wing dam and concrete core rockfill saddle dam identified as South Berm. The foundation is likely permafrost bedrock for the gravity dam and spillway, and the concrete core wall except for a small area of the North Berm. Details of dam geometry are summarized in Table 1.1 and Table 1.2.

Table 1.1 Embankment Dam Details

Dam Embankment			
Description	North Berm	Central Berm	South Berm
Foundation	Corewall on BR, 4m section on permafrost Rockfill shell on permafrost	Corewall on BR, Rockfill shell on permafrost	Corewall on BR, Rockfill shell on permafrost
Length of Crest	55.5 m	78.0 m	69 m
Minimum Crest Elevation	112.5 m	112.5 m	112.5 m
Maximum Height	4.3 m	4.5 m	1.0 m
Upstream Slope	2.0H:1V	2.0H:1V	2.0H:1V
Downstream Slope	2.0H:1V	2.0H:1V	2.0H:1V

Table 1.2 Concrete Dam Details

Main Spillway			
Type of Structure	Concrete gravity overflow		
Foundation	Bedrock		
Sill Elevation	111.33 m		
Crest Length	15.3 m	Crest Elevation	112.28 m
Right Abutment	Concrete Dam with rock fill 39 m long		
Left Abutment	Concrete Dam with rock fill 13.3 m long		
Total Discharge Capacity of Spillway Structures			
Total Discharge Capacity ofSpillway Structures [1]	24.5 m^3/s		
Inlet Structure to Transmission Main			
Type of Structure	Concrete gravity		
Screen	Steel bars imbedded in concrete		
Outlet structure	Twin 450 mm diameter mains		

[1] – Flow limit after which the concrete gravity section overtops.

Figure 2.2 Dam Components

2.1 History of Lake Geraldine Dam

2.1.1 Original Construction

Original dam constructed by the Department of Defense. Documentation consists of design drawings.

The dam included a spillway with a concrete ogee crest, abutting concrete gravity dam to the left and right, as well as a concrete core rockfill wing dam to the right with a reinforced concrete training wall at the interface. Relevant elevation data were spillway sill elevation 106.38 m and top of concrete El. 107.88 m.

2.1.2 Spillway Modified – 1979

Documentation in the Permanent Record file (PRF) indicates that the dam was raised 0.3 metres. However, record drawings of this modification were not available and may not exist. It is not clear whether the Central Berm was raised at this time, or if only the gravity dams and/or spillway were raised. Relevant elevation data was a spillway sill elevation 106.68 m.

2.1.3 Dam Raise - 1985

Consulting firm Oliver-Mangione-McCalla & Associated limited completed design improvements to the water supply dam. The improvements included

- Rockfill Berm was widened and raised by approximately 0.90 m to 108.78 m. The concrete core wall was extended following the existing batter and fastened to the original core wall with passive dowels at 500 mm spacing, embedded 150 mm into the original concrete.
- The North Berm was constructed to Elevation 108.78 m, including a sandbag core (no concrete core or cut-off) on native soil to Elevation 108.33 m.
- The spillway was raised by 1.150 metres 107.83 m as a broad-crested weir fastened to the existing spillway with a passive pig-tail anchor attached to a mechanical rock anchor.
- The concrete dam abutting the spillway was raised 0.90 metres to El. 108.78 m with a concrete block 900 mm high and 1,630 mm wide, fastened to the original dam with passive 15 M dowels embedded 300 mm into the original dam.

2.1.4 Dam Raise with Post-Tensioning – 1995

Consulting firm Oliver-Mangione-McCalla & Associated limited completed a second design improvements to the water supply dam in 1995. Post-tensioned anchor tendons are used for the first time to strengthen the concrete dam and spillway. The improvements included:

- The North and Central Rockfill Dam were merged over the previous rock knoll, and the concrete core wall at the North Dam is extended to bedrock El. 104.55 m.
- The core wall at the North Dam replaces the jute sandbags installed to bedrock. There is a small section where the core wall is not extended to bedrock, where two layers of rigid insulation are installed. The core wall is extended to El. 110.33 m.
- Central Rockfill Dam, the concrete core wall was extended following the existing batter and fastened to the original core wall with dowels at 500 mm spacing, embedded 300 mm into the original concrete. A 300 mm wide filter sand is placed adjacent to the concrete core wall upstream and downstream.
- The rockfill slope is steepened to 1.5:1 on the upstream and with a 300 mm thick riprap, 100 – 200 mm diameter, for erosion protection,
- The spillway was raised by 1.500 metres to El. 109.33 m as a thin-crested weir fastened to the existing spillway with 15 M dowels to match the vertical rebar in the extension, embedded 40 mm.
- The previous spillway section (1985 construction) was anchored into the foundation with a specified load of 226 kN/m of length. Anchor size and spacing are not provided. Rock anchor installation as-builts were not available. Notes on drawing 94-10047-1-3 and 94-10047-1-4 are confusing as to whether the anchors are post-tensioned. The notes require a calculation document from the installation contractor, which are not part of the permanent record file.

- The concrete dam abutting the spillway was to El. 110.28 m with a concrete block 1,500 mm high and 1,630 mm wide, fastened to the original dam with passive 15 M dowels embedded 300 mm into the original dam. Previous gravity section (1985 construction) is anchored into the foundation with a specified load of 226 kN/m of dam length, same as at the spillway

There are no construction records to verify the tendon anchors installed in 1995 were post-tensioned, although it seems unlikely that the type of construction specified would have been installed otherwise. Notes on the drawings recommend relieving the post-tensioning and grouting the tendon without tension.

2.1.5 Dam Raise with Post-Tensioning – 2006

Consulting firm Trow Associates Inc. completed a design crest raise to the water supply dam in 2005 and 2006. Similar to the 1995 modifications, Trow also used post-tensioned anchor tendons to strengthen the concrete dam and spillway. Unfortunately, Trow used the same notes on the drawings as in 1995 and did not provide an anchor installation report. The improvements included:

- In 2005, the previous gravity dam and spillway sections (1995 construction) is anchored with 46 mm diameter tendon type anchors into the foundation with a specified load of 403 kN/m of dam length, at an anchor spacing of 2,800 mm. The drawing notes are the same as from 1995 and are confusing as to whether the anchors are post-tensioned. The notes require a calculation document from the installation contractor, which is not part of the permanent record.
- In 2006, the crest of the dam and the spillway was raised, which includes a new South Berm. All rockfill berms are shown with a crest El. 112.50 m and a top of concrete core wall 112.30 m. The new core wall extension is fastened to the 1995 section with vertical 15 M dowels embedded 450 mm into the old concrete and the sand filter on the upstream and downstream is extended. Two strips of waterstop are added for leakage management. The upstream and downstream slopes are shown as 2:1 with a 2.4 m wide crest.
- The spillway was raised by 2.0 metres to El. 111.33 m as a broad-crested weir and a thin concrete shell downstream of the existing spillway, fastened to the old concrete with 15 M dowels at 1,500 vertical and 1,000 mm horizontal spacing embedded 600 mm into the older concrete surface.
- The concrete dam abutting the spillway was to El. 112.28 m with a concrete block 2,000 mm high and 2,630 mm wide, fastened to the original dam with passive 15 M dowels embedded 600 mm into the original dam.

2.1.6 Instrumentation and Monitoring - 2019

The goal in 2019 was to establish performance-based surveillance at Lake Geraldine that would identify deviations in performance conditions and corrective or risk mitigation measures be implemented before adverse consequences result. While not an exhaustive list, there are five (5) generally recognized purposes of instrumentation.

1. Analytical assessment – verification of design parameters
2. Observing performance of known anomalies
3. Predicting future performance
4. Establishing baseline data
5. Refining future designs

The City of Iqaluit completed an insitu geotechnical investigation at Lake Geraldine Dam in 2019 (CanaDrill, 2020) with 11 boreholes to investigate and monitor soil and bedrock conditions. Borehole locations are identified in Figure 2.1. Instrumentation was installed in all boreholes for performance monitoring; five (5) with open standpipe piezometers and seven (7) with thermistors.

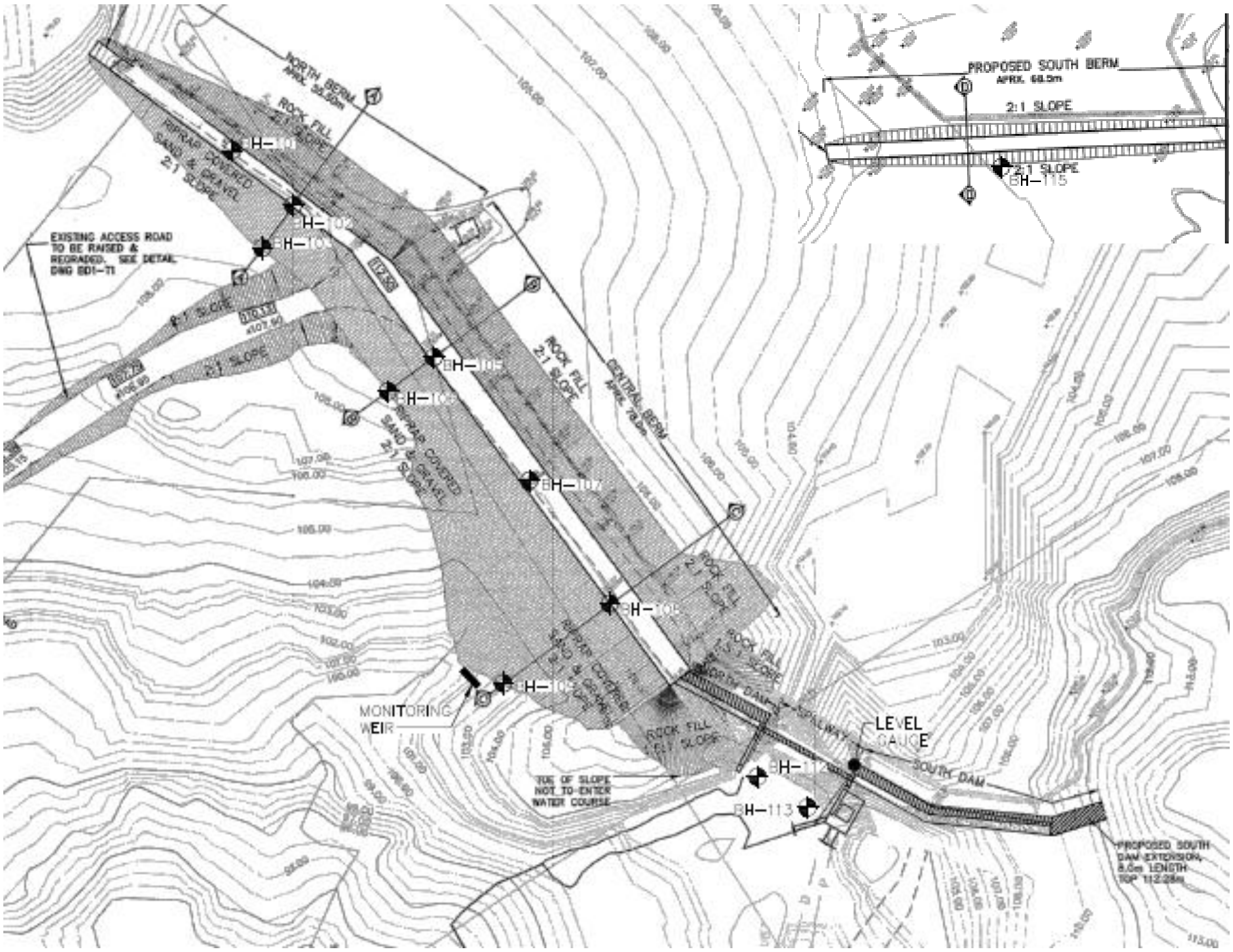

Figure 2.3 Borehole/Instrumentation Locations

Instrumentation installation details summarized in Table 2.1. Instrumentation in the North Berm include one (1) open standpipe piezometer and one 2 - array system of thermistors with an upstream thermistor in the dam and a downstream thermistor at the toe. The goal is;

- Piezometer in BH101 will monitor water level downstream of the core wall and benchmark with the reservoir level to monitor changes in porewater pressure during different seasons, and
- Thermistors in BH102 and BH103 will monitor for changes in depth of the active zone for different seasons

Table 2.1 Instrumentation Installation Details

Location	Instrumentation Type	Elevation (m)		Comment
		Top	Bottom	
BH101	Piezometer P101-1	106.3	105.9	400 mm Monitoring Zone
BH102	Thermistor T102-1	112.4	97.1	14.0 m string, 29 bulbs
BH103	Thermistor T103-1	108.5	100.6	7.0 m string, 15 bulbs
BH105	Thermistor T105-1	112.5	97.3	14.0 m string, 29 bulbs
BH106	Thermistor T106-1	108.1	100.2	7.0 m string, 15 bulbs
BH107	Piezometer P107-1	103.7	103.2	500 mm Monitoring Zone
BH108	Thermistor T108-1	112.5	97.2	14.0 m string, 29 bulbs
BH109	Thermistor T109-1	103.0	95.1	7.0 m string, 15 bulbs
BH112	Piezometer P112-1	99.3	98.8	500 mm Monitoring Zone
BH113	Piezometer P113-1	99.4	98.9	500 mm Monitoring Zone
	Piezometer P113-2	98.2	97.7	500 mm Monitoring Zone
BH115	Thermistor T115-1	112.1	104.1	7.0 m string, 15 bulbs

Instrumentation in the Central Berm includes one (1) open standpipe piezometer and a two 2 - array system of thermistors with an upstream thermistor in the dam and a downstream thermistor at the toe. The goal is;

- Piezometer in BH106 will monitor water level downstream of the core wall and benchmark with the reservoir level to monitor changes in porewater pressure during different seasons,
- Thermistors in BH104 and BH105 will monitor changes in depth of the active zone for different seasons, and
- Thermistors in BH107 and BH108 will monitor for changes in depth of the active zone for different seasons. This is an area of observed leakage and differences in the dam and toe active zone depth may identify uncontrolled seepage source from the reservoir.

Instrumentation in the spillway three (3) open standpipe piezometers. The goal is;

- Piezometer in BH112 will monitor water level downstream of the spillway, right side, and benchmark with the reservoir level to monitor for changes in porewater pressure during different seasons, and
- Piezometer in BH113 will monitor two water levels downstream of the spillway, left side, and benchmark with the reservoir level to monitor for changes in porewater pressure during different seasons.

Instrumentation in the Suth Berm include one (1) open standpipe piezometer. The goal is;

- Piezometer in BH115 will monitor water level downstream of the core wall and benchmark with the reservoir level to monitor changes in porewater pressure during different seasons.

2.2 Lake Geraldine Dam Rating Curves

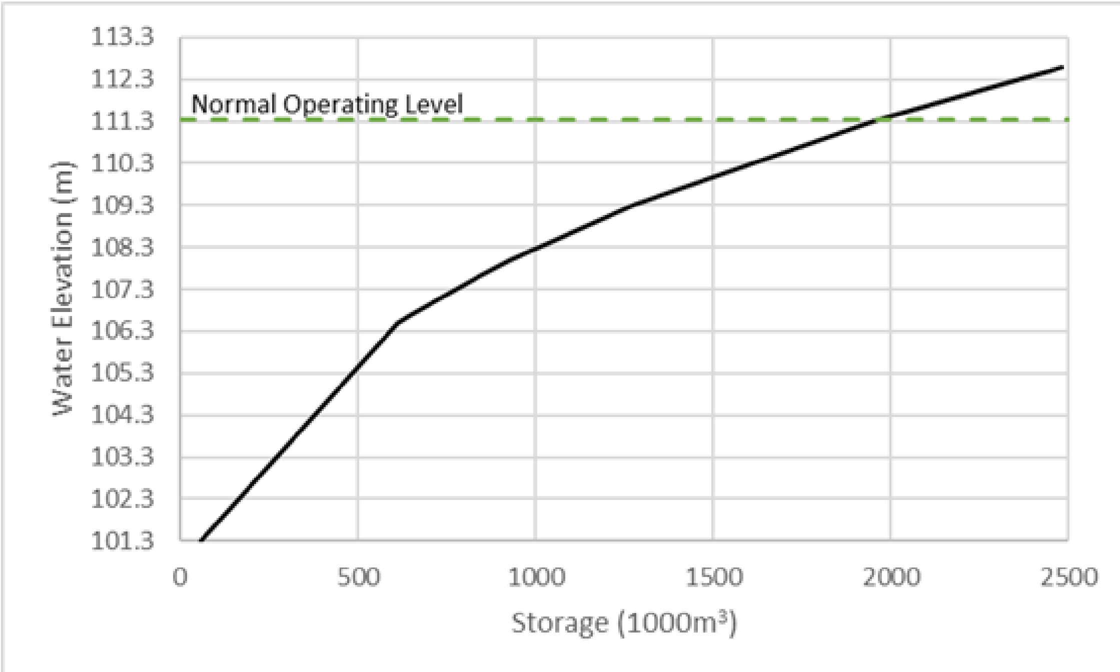

Figure 2.4 Lake Geraldine Stage-Storage Curve

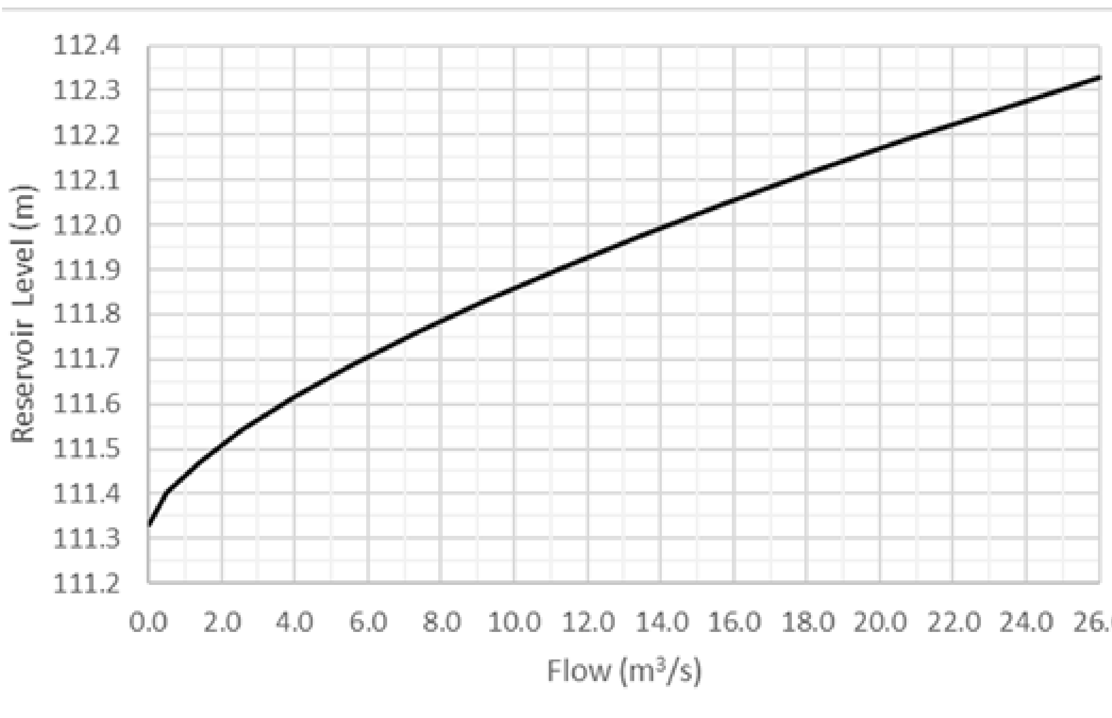

Figure 2.5 Lake Geraldine Spillway Stage-Discharge Curve

3 Dam Safety Review

The DSR was completed in general accordance with the CDA Dam Safety Guidelines (CDA, 2013). A summary of the CDA Guidelines is described in Appendix A.

3.1 Dam Classification

The CDA Guidelines recommend dams are assigned a consequence classification based on an assessment of consequences of a failure using dam failure models. 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.

The consequences of failure at Lake Geraldine Dam and Spillway were assessed as Extreme (AMEC, 2018). The inundation maps used to determine the consequence classification were developed in 2012 by Amec and have not been updated. The City of Iqaluit has grown slightly since 2012 but there have not been any significant changes to the downstream environment compared with inundation maps from 2012. The consequences appear to be comparable to the period when dam failure models were prepared, and the recommended classification is Extreme.

The recommended IDF is a probable maximum flood (PMF). The recommended earthquake design ground motion (EDGM) is a seismic event with an annual exceedance probability of 10,000 years.

Table 3.1 Recommended Classification for Lake Geraldine Dam

Structure Name	Previous Classification	Recommended Classification	Recommended IDF	Recommended EDGM
Lake Geraldine Dam	Extreme	Extreme	Q_{PMF} (21.9 m^3/s)	AEP 10,000 (PGA 0.116g)

3.2 Condition Assessment

A site dam safety inspection (DSI) was performed September 18, 2024. Weather conditions were favorable and there were no missed opportunities for inspections due to weather. The dam was observed to be in Good physical condition with minor deficiencies noted. The dam was not observed as under duress or at threat of imminent failure. A detailed physical condition report is included in Appendix B.

There are 11 deficiencies identified, with priority rating from Low to Very High, as summarized in Table 3.2. Very High priority items should be addressed in 2025. Other deficiencies can be addressed in a timely manner following the guidelines provided in Appendix B.

Table 3.2 Lake Geraldine Dam Physical Condition Deficiency List

Location	Priority	Defect	Recommended Mitigation
Concrete Spillway	Very High	Grout Injection Collars project above the surface	Zip cut all protrusions and repair as necessary
Instrumentation	Very High	Instrumentation installed in 2019 is not being monitored	Initiate a plan to immediately start collecting data from the installed instrumentation devices
Intake Valve	Very High	Intake valve is not regularly operated and may not work on demand	Current plans to relocate the intake valve chamber downstream should be completed
Spillway	Very High	Planned CCTV instrumentation at the spillway is not installed	Complete plans to install a remote observation post using CCTV camera
North and Centre Rockfill Berm Upstream Rockfill	High	The rockfill forming the upstream slope is small and likely does not provide suitable erosion protection	Install a riprap protection zone on the upstream slope
Concrete Joint Sealant	High	Sealant material is missing	Replace sealant material, as necessary, and install a solid cover to prevent future wildlife damage
All Structures	High	Currently there is no public safety around dams (PSAD) plan	Complete a PSAD risk assessment and develop a public safety plan (PSP) for Lake Geraldine Dam
Instrumentation Collars	Medium	Crushed stone placed at the base on new collars are eroding	Replace crushed stone with a geotextile fabric below the stone to prevent drop-out
South Berm	Medium	Contaminated soil located at the base of the hydro pole	Removed the contaminated soil and the remnant pole anchor
Spillway Channel	Low	Missing rock at the spillway toe may cause spillway instability	Rock stones in the spillway channel to be redistributed flush with the spillway apron
Concrete Spillway	Low	Hollow area downstream of South Concrete Gravity Dam	If spall occurs, repair area in accordance with ACI standard repair methodology
South Berm	Low	Displaced rockfill results in a gap in the rockfill parapet	Replace rockfill parapet in area of displacement

3.3 DESIGN ADEQUACY

The design adequacy review is completed to a preliminary level. Detailed analysis of embankments dams, concrete structures and hydraulic design are included in the appendices. Design Criteria for the structures are summarized in Table 3.3.

Table 3.3 Design Criteria and Operating Levels

Structure Name	FSL (m)	IDF (m^3/s)	MFL (m)	Crest (m)	EDGM
Lake Geraldine Dam & Spillway	111.33	21.9	111.90	112.50	PGA 0.116g

Risk of poor performance at Lake Geraldine was evaluated in a hazard and failure mode matrix (HFMM) in 2019, that identified 14 potential risk pairings for the dam and spillway. Each pairing was evaluated against standards-based criteria in the CDA Guidelines, where applicable. If the pairing event complies with the prescribed standard, the risk is considered tolerable. The CDA Guidelines require engineering judgement for many failure modes where standards-based criteria are not provided, e.g., internal erosion. Engineering judgement adopts descriptive probabilities to qualitatively assign a risk, using terms like "more likely than not to occur" as intolerable, and "less likely than not to occur" as tolerable.

Three risk pairings were assessed as high risk and identified for remedial measures. The following activities are recommended to reduce risk.

- Increase the height of the rockfill core wall 300 mm.
- Increase the crest height 1.0 metre, which can be achieved with a crest raise and a rockfill parapet.
- Route the warm water return line in the reservoir to discharge at the spillway sill.
- Actively monitor for foundation seepage at the North Berm and the Central Berm

3.3.1 HYDROTECHNICAL REVIEW

A detailed hydrotechnical review was completed as part of the recent risk assessment (Meco, 2019). Lake Geraldine watershed is small with an estimated catchment area of only 3.9 km^2 with local relief of between 100 meters and 200 meters above sea level, with Lake Geraldine's reservoir representing approximately 10 percent of the area.

The inflow design flood (IDF) for Lake Geraldine Dam is a probable maximum flood (PMF), a theoretical event that combines precipitation in the form of rain and snow, coupled with a temperature sequence. The precipitation model uses a probable maximum precipitation (PMP) determined using the statistical estimate methodology developed by Hershfield. Snowmelt was calculated in the HEC-HMS package using the temperature index methodology, which is an extension of the degree-day approach to modeling snowpack, with Environment Canada recorded data from 2015.

The Canadian Dam Association (CDA) Guidelines define three (3) scenarios in determining the PMF:

- Spring PMP with a 100-year snowpack melting;
- 100-year Spring rain event with the PMSA melting; and
- Fall PMP.

The governing combination of events that produced the maximum PMF effect at the Lake Geraldine reservoir was a Spring PMP combined with a 100-year snowpack. The resulting peak inflow for the reservoir is 21.9 m^3/s. A routing analysis estimated a 24-hour PMF event surcharges the reservoir 0.57 metres to 111.90 m with a corresponding outflow of 10.6 m^3/s.

The hydrotechnical analysis is comprehensive and still relevant for Lake Geraldine watershed. There have been no significant floods since the assessment that would result in a material change to the assessment. A review of climate normal for Iqaluit in Table 3.4 and Table 3.5 demonstrate a warming environment and reduced precipitation. For the City of Iqaluit, this suggests a potential decrease in clean water availability, as water production in Iqaluit relies heavily on snow accumulation during winter. This is unlikely to increase the risk to dam safety but will have an impact on operations.

Table 3.4 Climate Normals for Iqaluit (1981-2010)

Jan	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Daily Temp (C)	-26.9	-27.5	-23.2	-14.2	-4.4	3.6	8.2	7.1	2.6	-3.7	-12.0	-21.3	-9.3
Rainfall (mm)	0.0	0.0	0.0	0.2	3.1	23.8	51.9	68.6	42.2	6.8	0.6	0.0	197.2
Snowfall (cm)	21.7	21.0	21.6	31.5	27.6	9.3	0.0	0.9	13.2	29.4	29.7	23.4	229.3
Precip (mm)	19.7	18.7	18.7	27.5	29.2	33.0	51.9	69.5	55.2	33.3	27.2	19.9	403.7
Snow Depth (cm)	22	25	26	31	22	3	0	0	0	5	15	19	14

Table 3.5 Climate Normals for Iqaluit (1991-2020)

Jan	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Daily Temp (C)	-26.0	-27.0	-22.4	-13.5	-3.2	3.9	8.1	7.5	2.9	-3.2	-11.1	-18.9	-8.6
Rainfall (mm)	0.4	0.1	0.0	0.0	3.3	46.1	44.4	65.5	43.9	12.3	0.7	0.0	216.6
Snowfall (cm)	19.4	15.1	20.6	23.8	23.0	3.8	0.0	0.1	8.5	21.1	25.9	28.8	190.0
Precip (mm)	16.3	14.0	21.4	22.7	21.0	48.7	39.8	61.7	50.8	30.2	18.5	16.2	361.2
Snow Depth (cm)	19	21	23	26	19	1	0	0	0	3	10	16	11

3.3.2 Freeboard Analysis

A detailed Freeboard assessment was completed as part of the recent risk assessment (Meco, 2019). At the time, freeboard requirements for Normal operating and for Minimum operating environments were non-compliant. For an Extreme consequence dam, the CDA Guidelines recommend a wind speed with an annual exceedance probability of two (2) years. Under the circumstances, Lake Geraldine Dam is non-compliant for crest height and susceptible to wave action splashing over the dam during floods and high wind speed events.

3.3.3 Concrete Structures

Both the spillway and abutting concrete dam at Lake Geraldine were originally constructed as gravity stabilized structures. Following the modifications in 1995, stability was reliant on an array of steel tendon anchors. During two construction periods, steel tendons are installed to stabilize the concrete structure, 1995 and 2006, but in both cases, records indicate they were load tested in the bedrock, but reference drawings indicate they were not post-tensioned in the concrete mass. This is not a standard construction method, but the review of records indicates that the steel tendons are installed as passive anchors rather than post-tensioned anchors.

A passive anchoring system relies on displacement to mobilize full strength. Displacement can have negative impacts on concrete structures that may develop a cracked base and adverse uplift pressures.

Passive anchor performance cannot be verified so a conservative approach with respect to uplift pressures was used to evaluate stability of the concrete structures. A suite of uplift pressures was considered, including uplift for over two-thirds of the base, i.e., 100 percent uplift to the second anchor location. For this extreme condition, both the spillway and abutting dam were compliant with CDA criteria for stability with less than half the anchor capacity mobilized.

Other non-structural failure modes may result in premature failure of anchors, including unexpected load transfer from thawing permafrost, debonding of grout with the bedrock from temperature changes, or breakdown on the fractured bedrock and pullout of the anchors. A monitoring program to evaluate horizontal and vertical displacement, as well as porewater pressures in the base of the concrete structures is suggested as a means to monitor performance.

3.3.4 Rockfill Dam

The rockfill dam with concrete core wall was assessed as adequately stable with respect to external erosion for the current slope inclination and material properties. The 2019 risk analysis considered the key risk driver for the rockfill dam was uncertainty with respect to permafrost loss and the potential impact on seepage under the core wall, and deformation of the rockfill shell on the downstream. There has not been any observation of deformation. Seepage observed downstream of the Central Dam indicates potential for seepage under the core wall.

There are two (2) areas of concern with respect to foundation performance, (1) the North Berm and (2) the central berm.

- At the North Berm, the thermal model indicated the active layer extends to a depth of about 7.5 m which is below the measured depth of 5.0 m. Seepage at the North Berm is modelled to be greater than at the Central Berm, based primarily on the assumption that the corewall is not founded entirely on bedrock.
- At the Central Berm, peak seepage occurs in the summer when the downstream slope is completely thawed, and flow is concentrated along the bedrock surface. This was also verified during the geotechnical investigation. During re-freezing in the autumn, seepage is gradually reduced and eventually ceases when the thawed zone becomes discontinuous. This observation is to be verified through surveillance.

Observations of leakage at the Central Dam were noted in the geotechnical investigation during drilling at BH107. The observed depth to bedrock is nearly 9 m lower than at BH105, located to the right, and there was caving reported at elevation of ~103 metres, approximately 6.5 metres above bedrock. Water was encountered at the bedrock / rockfill interface in BH107, but a piezometer at the interface could not be installed due to caving. Pooled water was observed downstream BH107 during the drilling.

The thermal effect of the warm water in the reservoir results in thawing of the ground downstream of the core wall during summer. At the central berm, the thermal model indicated the active layer extends to a depth of about 7.9 m (Elevation 104.6 m), which is below the assumed bedrock surface. This was verified in initial results from the geotechnical investigation.

3.3.5 INSTRUMENTATION

Instrumentation to monitor temperature and porewater pressures were installed as part of the 2019 geotechnical investigation. The City of Iqaluit modified the instruments in 2023 by including more protections and all-weather stations for continuous data recording. There is no record of the work completed in 2023 and there were no calibration records for the dataloggers. These records should be sourced from the contract firm that completed the work and included in the Permanent Record File.

Other than readings recovered as part of the investigation (CanaDrill, 2020), there was no monitoring of instruments until 2023, when all-weather dataloggers were installed for the thermistors. A comparison of data recovered from the data loggers indicates there may be a discrepancy in numbering in 2023 and 2019, highlighted in Table 3.6. Staff at the City of Iqaluit recovered data from five (5) of the seven (7) all-weather stations thermistor monitoring sites from the installation date, i.e., mid-2023, to current in December 2024.

Further, analysis of the data recovered indicates there is a need for further processing to organize the data to allow for informed decisions on dam safety. There are no records of porewater pressures from the installed standpipes. There is a high likelihood that data from the different instruments can be recovered and processed for review.

Table 3.6 Instrumentation Installation Details

ID	Instrumentation	CanaDrill	2024 Data Recovered
BH102	Thermistor T102-1	14.0 m string, 29 bulbs	15 Analog Channels
BH103	Thermistor T103-1	7.0 m string, 15 bulbs	23 Analog Channels
BH105	Thermistor T105-1	14.0 m string, 29 bulbs	15 Analog Channels
BH106	Thermistor T106-1	14.0 m string, 29 bulbs	23 Analog Channels
BH108	Thermistor T108-1	7.0 m string, 15 bulbs	15 Analog Channels
BH109	Thermistor T109-1	7.0 m string, 15 bulbs	No Data
BH115	Thermistor T115-1	14.0 m string, 29 bulbs	No Data
BH101	Piezometer P101-1	400 mm Monitoring Zone	No Data
		400 mm Monitoring Zone	No Data
BH107	Piezometer P107-1	500 mm Monitoring Zone	No Data
BH112	Piezometer P112-1	500 mm Monitoring Zone	No Data
BH113	Piezometer P113-1	500 mm Monitoring Zone	No Data
	Piezometer P113-2	500 mm Monitoring Zone	No Data

4 Dam Safety Management

Management of dam safety has the goal of reducing risk of failure and inadequate functionality while also providing a means to respond to events that are unplanned. A dam safety management system (DSMS) describes the overall management framework for dam safety activities, decisions and supporting processes. Components of a DSMS recommended for the City of Iqaluit in 2019 are outlined in Figure 4.1.

Dam Safety Policy

Reporting

- Timely Reporting to Decision Makers
- Effective Communication

Planning

- Work Program Components
- Execution Responsibilities
- Standards and Procedures
- Resources
- Schedules

Corrective Actions

- Follow-Up Verification
- Incident Investigations
- Deficiencies and non-conformances Action List

Implementation

- Operation
- Maintenance
- Emergency Preparedness
- Public Safety

Checking and Reviewing

- Surveillance and DSR's
- Peer Review or Review Boards
- Audits
- Incident Investigation
- Testing of EPP/ERP
- Equipment Tests

Supporting Processes

- Staff Training and Qualifications
- Program Communication
- Record Keeping and Management

Figure 4.1 Dam Safety Management System (CDA Guidelines, 2013)

4.1 Dam Safety Management Gaps

The 2019 dam safety assessment identified gaps in current practices compared with recommended practice included in the CDA Guidelines for governance, facility management and administration. The assessment included nine (9) recommended mitigation activities to address the gap between policy and procedure and compliance, summarized in Table 4.1. Two (2) of nine (9) were achieved as of 2024.

Table 4.1 Dam Safety Management Compliance Gap

Location	Priority	*Description*	*Recommended Mitigation*	Achieved (Y/N)
Governance	High	The City of Iqaluit does not have a policy statement on dam safety	A sample policy statement on dam safety is proposed. Review, edit and adopt as appropriate.	N
Governance	High	When planning, a list of all deficiencies is needed to fully appreciate scope of need	Identify and catalogue current planning activities that are related to dam safety	N
Governance	Medium	There is no current means to monitor conformance to long term planning and policy regarding dam safety	Develop an internal audit program to monitor progress towards conformance with the DSMP	N
Governance	Medium	Performance of activities as they relate to dam safety are not measured against expectations	Develop an Improvement Management Program (IMP) for dam safety with annual benchmarking.	N
Governance	Medium	There is currently no means to assess performance against goals and expectations	The Director of Public Works will report to senior management on system performance annually	N
Facility Management	High	Operations staff are not trained in dam safety inspection techniques	Provide on the job training with dam safety professional engineers to operations staff	N
Facility Management	High	Instrumentation installed in 2019 requires equipment and software for monitoring	Sub-contract monitoring until such time as resources can be trained in-house to complete tasks	Y
Facility Management	Medium	There is a need to conduct a risk assessment for the public accessing the dam	Perform a risk assessment as prescribed by the Canadian Dam Association (CDA)	N
Facility Management	Medium	The permanent record file (PRF) is currently maintained by an independent consultant	The PRF should be managed in-house under the direction of the Director of Engineering and Public Works	Y

None of the suggested improvements in governance were achieved. Interviews with current management indicate staff turnover rates disrupt continuity in initiatives, but also the specialty nature of dam safety as potential causes of the lack of progress rather than desire. Senior management interviews identify Lake Geraldine Dam performance as the joint responsibility of the Department of Public Works and the Department of Engineering & Capital Planning, organized as per Figure 4.2.

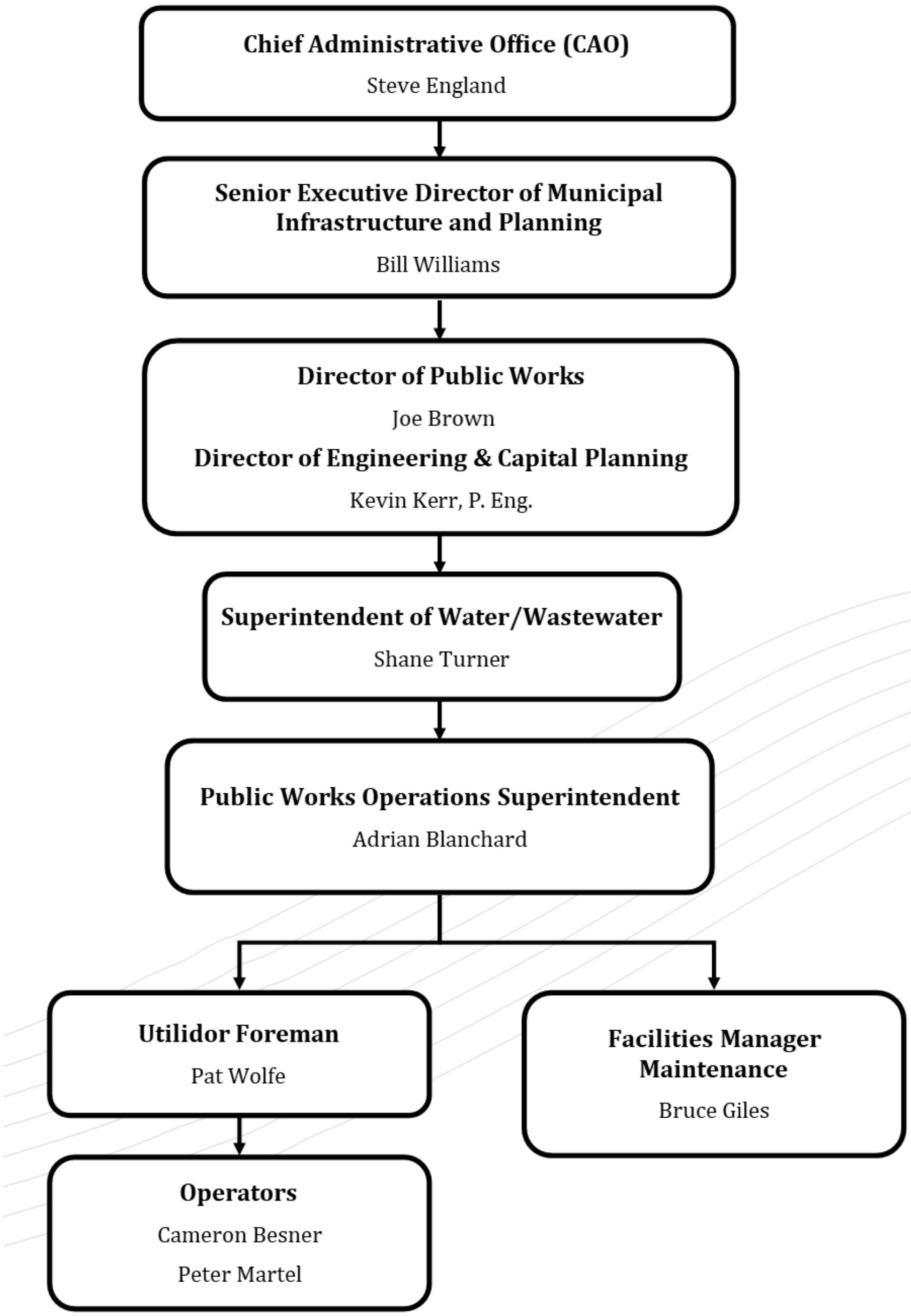

Figure 4.2 City of Iqaluit Organization Chart

For City staff, Lake Geraldine Dam is a small part of the overall asset portfolio, and a standalone DSMS may cause resource challenges. Integrating asset management plans with existing compliance based management systems that achieve compliance with the CDA Guidelines but are familiar with current management policies, as outlined in Figure 4.3 provides a possible model to manage dam safety for City Staff. Other recommendations included in the 2019 assessment will address challenges with continuity of actions and accountability of senior leadership for the city when implemented. Other tasks not addressed from the risk assessment include

- Establish a master list of deficiencies and prioritize actions.
- Regularly test emergency response and preparedness plans.
- Centralize the PRF under the City's engineering department.
- Conduct risk assessments for public access areas around the dam.
- No system currently tracks long-term policy adherence or prioritizes actions based on urgency.

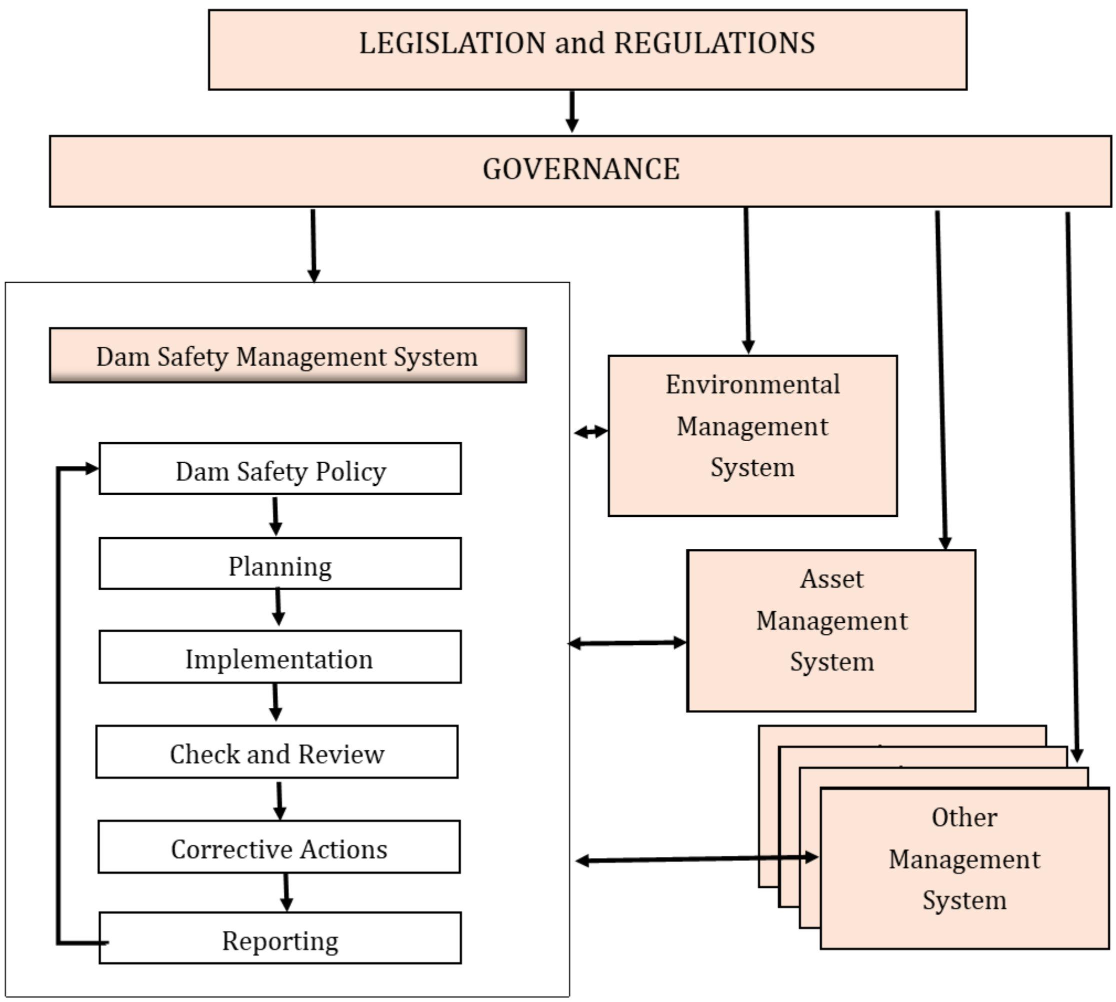

Figure 4.3 Dam Safety Management Model

The City of Iqaluit achieved two (2) of the four recommendations regarding facility management. City staff developed a relationship with Concentric Engineering as an outside resource for dam safety management from 2021 to 2023, based on an RFP for Dam Safety Improvements that included the following tasks;

- Three Year Inspection & Monitoring Program from 2021, 2022 and 2023
- Dam Safety Review (DSR) in 2021
- Automated Monitoring System to record reservoir level, intake flow rate and a CCTV at the dam.
- Dam improvements including replacement of riprap, addition of fill and repair of expansion joints
- Miscellaneous Maintenance Items
- Public Safety Risk Assessment and use the results to prepare a public safety plan (PSP).

The documentation provided for the DSR indicate consistent dam safety inspections and some upgrade work. All tasks were not completed in the period. The City of Iqaluit issued a subsequent RFP in 2024 but did not receive any proposals, and the task is currently outstanding again. The documentation provided for the DSR did not include verification of on-the-job training or job shadowing and identified poor record keeping of upgrades to the instrumentation at the dam.

4.2 Normal and Emergency Management

The City of Iqaluit has four (4) documents in the permanent record file (PRF) governing operations at Lake Geraldine Dam from Normal Operation to Emergency Operations.

- An **Operations, Maintenance, and Surveillance (OMS) Manual at Lake Geraldine Dam**, which describes how day-to-day tasks related to dam safety at LGD are performed, and by whom. The OMS manual contains procedures for normal, flood, and emergency operations;
- An **Emergency Preparedness Plan (EPP)**, which provides inundation maps and other information to assist external response agencies in preparation of their local emergency plans to deal specifically with a dam failure at LGD;
- An **Emergency Response Plan (ERP)**, which provides general information related to the City of Iqaluit's response to a dam safety emergency; and
- A **Dam Safety Management Plan (DSMP)**, a framework for dam safety activities, decisions, and supporting processes which encompass all elements of dam safety for the City of Iqaluit.

All documents are structured around a target action response plan (TARP) that uses common operating levels to govern transitions from normal to emergency operations, to an evacuation if a dam failure is imminent. The agreed upon TARP level for Lake Geraldine are in Table 4.2. The purpose of the OM&S, EPP and ERP is to;

- Establish a framework for managing the dam's safety and operational reliability.
- Identify roles and responsibilities for dam safety personnel.
- Utilizes the Incident Command System (ICS) to coordinate emergency operations.
- Detail procedures for operations, maintenance, and surveillance.
- Document performance analysis and record-keeping

Table 4.2 TARP Levels – Lake Geraldine Dam

Activation Level	Condition	
	Hydrologic Event	**Other Events**
NORMAL CONDITION (Green)	Reservoir is at or below Elev. 111.33 m. No weather events forecast.	Not Applicable
FLOOD SITUATION (Blue) Out-of-bank water levels	a) Reservoir level at Elev 111.33 m with heavy snowpack. Rain forecast for 15 mm or more in 24 hours. b) Reservoir level at or above Elev 111.40 m.	Not applicable.
DAM ALERT (Yellow) Abnormal condition poses a threat	a) Reservoir Level at or above Elev 111.60 m due to snowmelt only. Rain forecast for 15 mm or more in 24 hours. b) Annual snowfall 2.0x more than normal. Reservoir at (111.33 m).	Abnormal condition that may affect dam performance has been identified, e.g.: • Beaching erosion at the crest, either upstream or downstream; • New leakage, or recurring leakage, observed to be increased and/or silt laden; • Minor structural deformation or deterioration.
DAM EMERGENCY (Orange) Potential dam failure is developing	a) Reservoir Level at or above Elev 111.90 m. Rain and/or snowmelt may result in additional flow. b) At elevated reservoir levels but is not eroding the slope and level is expected to recede.	Leakage downstream of the rockfill dam is escalating and is brown and silt laden. Rockfill dam has suddenly deformed and sinkholes and depressions are observed, associated with leakage. Minor displacement of the concrete dam/spillway and associated leakage. Abnormal conditions create threat to dam safety, requiring immediate attention. If implemented, remediation is expected to be effective.
DAM FAILURE (Red) Dam failure is imminent or has occurred	a) Reservoir level is expected to overtop concrete at the gravity dam, i.e., Elev 112.33 m. b) Rockfill Dam overtopping is occurring or imminent.	Upstream water level is decreasing rapidly, indicating an internal dam failure. Concrete dam has actively displaced and leakage is overwhelming the spillway channel. Rockfill dam is deforming, and leakage is excessive and escalating. Failure of dam is occurring or imminent.

Lake Geraldine Dam requires minimal operating decisions. The DSMS documents provide clear guidance for decision making in emergencies and for routine maintenance. The documents should be reviewed annually and updated to promote awareness and familiarity with the documents.

4.2.1 Dam Safety Inspections

Concentric Engineering, based in Ottawa, Ontario, was awarded a three year contract for dam safety inspections, among other tasks. Between 2021 to 2023, Concentric Engineering completed eleven (11) dam safety inspection (DSI) reports for Lake Geraldine Dam and Spillway; generally, between April and November. The inspection reports contain a mix of observed conditions (deficiencies) and suggestions for management (compliance). Through the documents, repeatedly observed deficiencies include:

- Ongoing mapping and repairs to cracking, scaling, and spalling on the concrete surfaces.
- A need for riprap/armor stone maintenance to prevent erosion.
- Vegetation management to ensure vegetation around the dam and berms does not compromise structural integrity.

The documents also repeated observed non-compliances include:

- City operators were encouraged to use weekly routine inspections by trained staff for early detection of issues. There were no records of training provided.
- Potential concerns about reliance on rock anchors for structural stability, although there was no visual evidence presented to suggest overstressing. The reports regularly expressed concern about the fractured bedrock under the spillway and recommended further geotechnical investigation work.
- A need to maintain and regularly update the emergency action plan.
- There is a need for regular monitoring to ensure the reservoir operates within safe levels.
- A need for periodic geotechnical and structural evaluations.

There are no records in the PRF to substantiate routine inspections by City staff or training by Concentric Engineering. Regular informal routing inspections will enhance the quarterly formal inspections and, for a relatively unchanged condition at Lake Geraldine Dam, likely could reduce the number of formal inspections to one annually.

4.2.2 Automated Monitoring System

In 2023, a contractor was retained to automate the instrumentation installed in the 2019 geotechnical investigation. There is no documentation of what was done, the purpose or results in the Permanent Record File for Lake Geraldine Dam provided for the DSR. Since the start of the DSR, the City of Iqaluit recovered data from some of the instruments (6 of 8 thermistors, none of the piezometers) but have not been able to recover a report of what was done and by whom. These records should be sourced from the contract firm that completed the work and included in the Permanent Record File.

5 CONCULSIONS AND RECOMMENDATIONS

Acting on the authorization of City of Iqaluit, Mitchelmore Consulting International Ltd., (Meco) has performed the 2024 Dam Safety Review (DSR) of the Lake Geraldine Dam. A DSR is 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. The Canadian Dam Association (CDA) *Dam Safety Guidelines* recommend a DSR cover all aspects required to demonstrate that;

- The dam is safe, operated safely, and maintained in a safe condition, and
- Surveillance is adequate to detect any developing safety problems.

Lake Geraldine is the water supply for the City of Iqaluit. Based on current digital terrain maps, Lake Geraldine Dam controls a catchment area of approximately 3.9 square kilometers. The Dam was originally constructed as a gravity concrete structure, more likely than not on permanently frozen bedrock. A rockfill embankment with thin concrete core was constructed as an abutment. The concrete dam and spillway have been modified four (4) times to increase storage. During each modification the rockfill embankment distance was increased and the concrete spillway and dam was raised. Currently the concrete dam and spillway relies on supplemental anchors for stability. The rockfill embankment was modified in 1995 to ensure the centre concrete core wall was constructed on bedrock for all but a small section near the right abutment outcrop.

Lake Geraldine Dam and Spillway was evaluated based on a visual physical condition assessment September 18, 2024. Weather conditions were favorable, and all exterior parts of the dam were observed. Overall, Lake Geraldine Dam and Spillway was observed to be in Good condition with only minor deterioration or structural deficiencies. These should be addressed based on the timelines provided.

A review of the consequences of dam failure included a review of inundation mapping from 2012. The review determined the maps are a reasonable representation of the current downstream environment. Based on the review, the recommended consequence classification for Lake Geraldine Dam is verified as Extreme. The associated design criteria are as follows:

Dam	Previous Classification	Recommended Classification	Recommended IDF	Recommended EDGM
Lake Geraldine Dam and Spillway	Extreme	Extreme	Q_{PMF} (21.9 m^3/s)	$AEP_{10,000}$ (PGA 0.114g)

A design adequacy assessment was completed to a preliminary level. A preliminary design adequacy assessment is completed using geometry provided by the client, material properties are estimated using engineering judgement. The following is a summary of the review:

- Flood hydrology prepared in 2019 was reviewed and considered relevant. The inflow design flood (IDF) is a probable maximum flood (PMF) associated with the Spring freshet and is the sum of a probable maximum precipitation (PMP) combined with a 100-year snowmelt. Hydraulic routing of the inflow hydrology estimated the maximum flood level (MFL) during the IDF is 111.90 metres.

- Freeboard, the distance between the reservoir level and the crest, is inadequate during normal operation and during a design flood. The review noted that Iqaluit typically experiences the strongest winds in September, i.e., normal operation, while the reservoir is more likely to be elevated during Spring, i.e., design flood. The critical component of the dam with respect to freeboard is the rockfill dam since the concrete dam can likely sustain splash overtopping without undermining. The freeboard analysis indicates the rockfill embankment crest should be raised by one metre to comply with freeboard criteria. A risk-informed analysis considering joint probabilities may result in a more economical design.
- Stability analysis of the concrete dam and spillway determined the structures comply with standards-based criteria in the CDA Guidelines, provided anchors behave as per design. Anchors are evaluated as passive tendons, grouted in the bedrock. There are two tiers of anchors, installed in 1995 and 2006. Factors that may result in reduced stability include de-bonding of the anchor grout with permafrost loss, degradation of the fractured bedrock foundation, unanticipated ice load, etc. Because of the uncertainty related to anchor installation and performance, additional monitoring is recommended. A monitoring plan should be developed separately, but should include monitoring porewater pressures at the concrete/rock interface, temperature, and lateral movement.
- Stability analysis of the rockfill embankment determined the analysis completed in 2019 was comprehensive and conditions have not changed in the interim. The key risk driver for the rockfill embankment is the development of internal erosion in the foundation. Two areas are identified as high risk, (1) the North Dam where the concrete core wall was not installed to bedrock for the full length, and (2) the Central Dam where caving was observed at BH107 and where ponding of water has been observed annually.

The City of Iqaluit has a dam safety management system (DSMS) that included procedures for operations, maintenance and surveillance (OMS), an emergency management plan (EMP), both organized around target action response plan (TARP) levels at Lake Geraldine. The DSMS does not include a public safety around dams (PSAD) risk analysis and associated public safety and security plan (PSSP). The DSMS includes instrumentation for monitoring performance with respect to permafrost losses and porewater pressures. To date the DSMS is not being implemented effectively.

The goal of dam safety management should be compliance within a familiar framework. The City of Iqaluit may improve success in implementing the DSMS by integrating dam safety management into existing compliance-based management structure while documenting operating procedures that achieve compliance with the requirements of the CDA Guidelines. The City of Iqaluit can improve knowledge transfer and continuity by requiring record documents for activities at Lake Geraldine, for example;

- In 2023, a contractor was retained to automate the instrumentation installed in the 2019 geotechnical investigation. There is no documentation of what was done, instrument type and calibration, the purpose or results. Without this information, monitoring of dam performance is impaired and potentially lost. These records should be sourced from the contract firm that completed the work and included in the Permanent Record File.
- Anchoring in 1995 and 2006 required an installation report by the contractor. Without this report, the anchors are considered passive anchors, although they were likely post-tensioned.

Repeated Risks and Challenges

- Aging Infrastructure. The dam, originally built in 1958 and raised multiple times, shows evidence of aging. The structural capacity of older extensions needs ongoing reassessment.

- Understanding and accounting for changes to climate from increasing ground temperatures.
- Environmental Factors: Arctic conditions contribute to freeze-thaw cycles, exacerbating concrete deterioration and requiring frequent repairs.
- Reliance on Rock Anchors: Concerns about relying on passive rock anchors for stability remain a significant risk. Current anchors are considered marginally compliant. There are repeated recommendations for further geotechnical investigations to evaluate the risk of anchor relaxation.

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APPENDIX A

CDA Guidelines Summary

TECHNICAL MEMORANDUM– CDA GUIDELINES

Project: 2024 Dam Safety Review
Lake Geraldine Dam & Spillway

Subject: CDA Dam Safety Review Synopsis

Author: Perry Mitchelmore, P.Eng.

Date: December 23, 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. NSTIR 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 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 City of Iqaluit 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 2015 National Building Code of Canada (median) values for a Class C soil foundation. Mean spectral parameters for 100-year to 2,500-year events were obtained by request from the Geological Survey of Canada. Spectral parameters for the 5,000-year and 10,000-year 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	Annual Exceedance Probability (AEP)			
	100-year	500-year	1,000-year	2,500-year
PSA 0.2 sec	0.0170	0.0400	0.0560	0.0860
PSA 0.5 sec	0.0140	0.0330	0.0450	0.0640
PSA 1.0 sec	0.0080	0.0210	0.0300	0.0420
PSA 2.0 sec	0.0040	0.0110	0.0160	0.0230
PGA	0.0080	0.0210	0.0310	0.0500

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

Physical Condition Assessment

TECHNICAL MEMORANDUM REPORT

Project: Lake Geraldine Dam

Subject Dam Safety Inspection (DSI)

Author: Perry Mitchelmore, P.Eng. **Reviewer** Mark Rustan

Date: December 31, 2024

The current Technical Memorandum Report (TMR) is for a dam safety inspection (DSI) of the *Lake Geraldine Dam* which was completed September 18, 2024 for City of Iqaluit.

1 INTRODUCTION

1.1 System Description

Lake Geraldine Dam is a composite structure that includes a concrete gravity section with an integral concrete spillway, and three rockfill berms: the north, center, and south berm section, each with a central concrete core. Properties of the dam are described in Table 1.1. A physical inspection of the dam is attached along with site photographs.

Table 1.1 Lake Geraldine Dam Properties

Dam/Berm Segment	Length (m)	Crest Elevation	Base Elevation	Bedrock Elevation	Height of Dam/Berm (m)
North Berm	55.5	112.5	108.3	105.0	4.3
Center Berm	78.0	112.5	108.3	97.5	4.5
North Dam	13.3	112.3	102.6	97.5	11.0
Spillway	15.3	111.3	101.6	96	10.0
South Dam	39.1	112.3	102.6	97.5	11.0
South Berm	68.5	112.5	111.5	110.0	1.0

The 15.3 m wide spillway has a sill elevation of 111.3 m (representing the Full Supply Level (FSL) of the reservoir), while the north and south concrete dam sections on either side of the spillway have a crest elevation of 112.3 m. At FSL, the concrete dam has approximately 1.0 m of freeboard. The South Concrete gravity section extends approximately 39.1 m to a rock abutment. The North Concrete gravity section extends 13.3 m where it mates with the Centre Rockfill Berm concrete core wall. The interface includes a vertical PVC waterstop.

The Center Rockfill Berm is approximately 75 m long to a rocky knoll where it meets the access ramp and extends approximately 60m to the north rock abutment. The South Rockfill Berm is a separate structure located in a valley to the south of the main concrete dam. The south berm is approximately 68.5 m long and about a metre high. The north, center and southern berms incorporate a concrete cutoff wall which is reportedly founded in rock at the base of the berms. The rockfill slopes slope approximately 2H:1V and appear regular and planar. There does not appear to be a separate size riprap on the upstream slope. The crest is approximately 4 metres wide and levelled with crushed stone. There is a slight rockfill parapet on the upstream slope.

Instrumentation to monitor pore pressures and ground temperature was installed in the dam in 2019. Pore pressure instrumentation includes five (3) standpipe type piezometers; two (2) in the rockfill Berm and three (3) in the spillway apron. Ground temperature instrumentation includes seven (7) 29-bulb thermistors in the crest and two of the rockfill dam. All instrumentation was modified late 2023. There are no records of the modifications and there is no recorded data available for any of the instruments.

1.2 Deficiencies Identified in Previous Inspections

The City of Iqaluit retained a third-party Dam Safety Engineer between 2021 and 2023 to perform quarterly dam safety inspections required in the operations, maintenance and surveillance (OMS) manual for the dam. Concentric Limited performed the quarterly inspections of Lake Geraldine Dam with the last inspection performed October 2023. An interpreted list of deficiencies from the various reports are summarized in Table 1.2.

There are four (4) Very High priority deficiencies that are either incomplete or ongoing. From a dam safety lens, these four deficiencies are operational priorities and unlikely to result in a dam failure. The depressions at the Centre and North Rockfill Berms were not observed and are likely addressed or of little consequence. The valve chamber is due to be relocated as part of ongoing capital work at the dam.

There were 11 High priority deficiencies that are either incomplete, complete or ongoing. Damage to the sealant materials at the concrete dam is an ongoing issue and mitigation is incomplete. Other incomplete items include a test excavation at the centre berm downstream shell replacement of riprap. Of the 11 deficiencies, five (5) were observed as complete and six (6) as incomplete. Two (2) of the incomplete deficiencies should be removed from the list. The last dive inspection of the concrete dam was completed in 2019 and a follow up in 2024 seems unnecessary without an incident, e.g., leakage, to justify the inspection. A 10-year cycle for diving inspection is adequate in the absence of an incident. Excavation of the rockfill toe at the centre dam to investigate the source of leakage should only be initiated if the downstream population is evacuated. Such a high risk activity requires detailed engineering objectives, an observation and response plan, as well as public safety assurances. There are likely better alternatives to an intrusive investigation.

There are no Moderate priority items but three (3) Low priority deficiencies that are incomplete. All three (3) are achievable and should be completed.

Remove two (2) Very High and two (2) High priority deficiencies as no longer relevant. Updates on all other deficiencies are included as part of the 2024 DSI.

Table 1.2 Pre-Inspection Deficiency List – Lake Geraldine Dam

Priority	Deficiency	Status
Very High	The contaminated soil around the base of the hydro pole adjacent to the south berm should be removed.	Incomplete
~~Very High~~	~~Outstanding deficiencies remaining to be corrected by Nunavut Excavation should be completed no later than the 2nd week of October 2023.~~	~~Unknown~~
~~Very High~~	~~The depressions at the base of the upstream face of the center and north berms should be repaired in the spring of 2024 when the water level is low.~~	~~No depressions observed~~
Very High	Exercising and testing of the valves within the valve chamber	Ongoing
High	Damaged and deteriorated sealant material should be replaced in 2024. It may be necessary to install a (removable) sheet metal panel over the horizontal expansion joint to protect it against damage from ravens.	Incomplete
High	One sink hole was observed within the south access road that leads to the southern berm. The sink hole should be excavated area should be refilled and compacted in 12” lifts. We also recommend the Concentric be present to identify the possible cause and record the depth, size, and potential impact on the berm.	No depressions observed
~~High~~	~~Undertake an underwater survey of the concrete dam and spillway in 2024.~~	~~Incomplete~~
High	The protective galvanized metal enclosure installed over the pipeline from the dam to the water treatment plant should be re-instated.	Complete
High	The metal posts / markers that were installed along the north side of the south access road should be re-instated.	Complete
High	The north and south access roads should be regraded and eroded material replaced in the spring of 2024.	Complete
~~High~~	~~Undertake a test opening on the downstream side of the concrete dam and center berm in late January – early February 2024 to ascertain the source of the water that forms large ice sheets within the valley in the winter months.~~	~~Incomplete~~
High	The rip-rap material in the spillway should be re-distributed to provide cover at the base of the spillway.	Incomplete
High	The displaced rip-rap material at the south berm should be re- instated.	Incomplete
High	Repair cracks within the concrete dam, this work is tentatively scheduled for summer 2024.	Complete
High	Repair of spalled concrete within the concrete dam, this work is tentatively scheduled for summer 2024.	Complete
Low	The installation of video surveillance should be considered with scheduled implementation in the next 5 years.	Incomplete
Low	Updating of the permanent record file and its storage in a central location with an index that documents the date and contents of all records. The permanent record file needs to include: a) As-built drawings and specifications for work undertaken at the dam. b) Weekly/monthly inspections completed by City staff. c) Dam Safety Inspections and Dam Safety Reviews generated by third parties on behalf of the City of Iqaluit. d) All maintenance records. e) Correspondence with regulatory agencies. f) Dam operation, maintenance, and surveillance documents. g) Reports and documentation generated by third parties on behalf of the City of Iqaluit.	Incomplete
Low	Implement a public awareness program to educate and inform the public that: a) The dam and earthen berms are no trespass area. b) Dog walkers should not allow their pets to travel atop and across the earthen berms due to the risk of (dog) fecal matter contamination of the potable water supply. c) ATV and skidoos should not travel on berms and across Lake Geraldine.	Incomplete

1.3 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 1.3 defines the priority rankings and the recommended timeline to rectify the deficiency.

Table 1.3 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

1.4 Condition Rating

Observations and issues identified during the 2024 inspection are presented in the inspection forms included in Appendix A. Site photos are included with the inspection forms in Appendix A. Condition status divisions, under the condition heading in the inspection forms are defined in Table 1.4.

Table 1.4 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.

1.5 Site Inspection Overview

The inspection was carried out September 18, 2024. Weather conditions were favorable and there were no missed opportunities for inspections due to weather. The City of Iqaluit engineering leadership team was briefed on results before the Meco team departed Iqaluit. The condition of each dam structure varied from Fair to Good with deficiencies noted. Deficiencies are described and photographed. The dam was not observed as under duress or at threat of imminent failure. The inspection team members included:

Perry Mitchelmore, P.Eng. (Geotechnical / Dam Safety)
Mark Rustan, P.Eng. (Structural)

2 Condition Assessment

2.1 Rockfill Berm

The North, Centre and South Rockfill dams were observed to be in Good Condition. Observations were consistent with previous inspections and the structures are largely unchanged since the last dam safety inspection in October 2023. The following deficiencies were observed.

The rockfill forming the upstream slope has unravelled in the past and was recently repaired. The rockfill sizes are inadequate to resist wave and ice erosion and will likely continue to unravel in the form of sloughing and beaching (Photo 4, 5 and 6). An engineered riprap is recommended for the upstream slope of the rockfill dams to mitigate erosion.

The instrumentation installed in 2019 was modified in 2023 by installing steel collars. Crushed gravel installed at the base of the new collars is eroding and potentially undermining the all-weather boxes installed for instruments (Photo 14, 15 and 16). Future repairs should include a geotextile fabric between the crushed stone and the rockfill to mitigate potential for drop-out of the finer crushed rock.

2.2 Concrete Dam and Spillway

The north and south concrete gravity dams and the concrete spillway were observed to be in Good Condition. Observations were consistent with previous inspections and the structure is largely unchanged since the last dam safety inspection in October 2023. The following deficiencies were observed.

Sealants of exposed joints, believed to have been installed in either 2023 or 2024, have deteriorated and will no longer prevent water and ice ingress. The deficiency was previously identified and installation of sheet metal covers was the recommended mitigation measure. The deficiency is attributed to birds eating the material and the mitigation was intended to hide the sealant. There was no indication of displacement to explain the loss of materials. Providing cover will likely mitigate the problem. Prior to installing covers, all joints should be resealed and the cover immediately installed.

Recent injection grouting at cracks in the concrete was completed but the caps were not removed. The remnant caps project above the conveyance surface and may cause cavitation and shear stresses in the concrete that may cause premature deterioration. All caps should be cut flush to the concrete surface.

Tap testing of the concrete surface identified one hollow area that may be experiencing delamination downstream of South Concrete Gravity Dam, (Photo 19). The area is approximately 300mm square and should be monitored in future inspections for potential spalling.

2.3 Instrumentation & Monitoring

Instrumentation for monitoring dam performance based on ground temperatures and location of the phreatic surface was installed in 2019. There was a suite of data collected at the time of installation but there are no records of monitoring at the instruments since 2019. In 2023, the instrument collars were modified, and all-weather boxes were installed for monitoring. Until recently, there have been no monitoring records from that period and no records of the work that was performed.

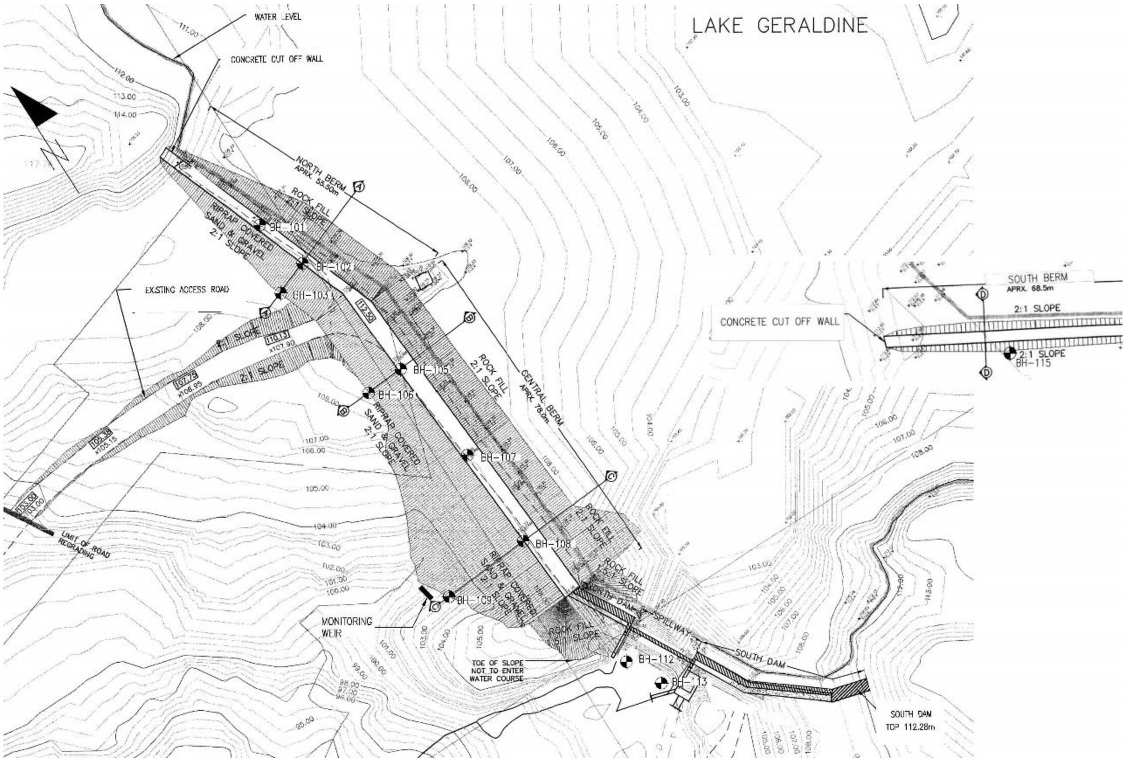

Figure 2.1 Location of Instrumentation Monitoring Devices

December 2024, City staff were able to provide records from a data recorder installed in 2023 for the thermistors installed in 2019. City staff continue to seek reports on what work was performed, instrument calibration records and validation of the 2023 update work compared with the 2019 installation logs. Preliminary plots of the data recovered from the datalogger indicate records date back to the summer of 2023, but that there are potential anomalies in the data.

There is currently no monitoring occurring at Lake Geraldine Dam and there are no records on how to monitor the instruments installed in 2019 and modified in 2023. The three (3) piezometers located at the spillway apron have the collar protection removed and are likely compromised. The datalogger report does not include piezometric data.

2.4 Deficiencies and Recommendations

Deficiencies identified during the 2024 Dam Safety Inspection and their corresponding recommended mitigation for Lake Geraldine Dam are summarized in Table 2.1.

Table 2.1 Lake Geraldine Dam Deficiency List

Index #	Location	Priority	Defect	Recommended Mitigation
2024-1	North and Centre Rockfill Berm Upstream Rockfill	High	The rockfill forming the upstream slope is small and does not provide suitable erosion protection	Install a riprap protection zone on the upstream slope

Index #	Location	Priority	Defect	Recommended Mitigation
2024-2	Instrumentation Collars	Medium	Crushed stone placed at the base on new collars are eroding	Replace crushed stone with a geotextile fabric below the stone to prevent drop-out
2024-3	Concrete Spillway	Very High	Grout Injection Collars project above the surface	Zip cut all protrusions and repair as necessary
2024-4	Concrete Joint Sealant	High	Sealant material is missing	Replace sealant material, as necessary, and install a solid cover to prevent future wildlife damage
2024-5	Instrumentation	Very High	Instrumentation installed in 2019 is not being monitored	Initiate a plan to immediately start collecting data from the installed instrumentation devices
		Previous Deficiencies Still Relevant		
	South Berm	Medium	Contaminated soil located at the base of the hydro pole	Removed the contaminated soil and the remnant pole anchor
	Spillway Channel	Low	Missing rock at the spillway toe may cause spillway instability	Rock stones in the spillway channel to be redistributed flush with the spillway apron
	South Berm	Low	Displaced rockfill results in a gap in the rockfill parapet	Replace rockfill parapet in area of displacement
	Intake Valve	Very High	Intake valve is not regularly operated and may not work on demand	Current plans to relocate the intake valve chamber downstream should be completed
	Spillway	Very High	Planned CCTV instrumentation at the spillway is not installed	Complete plans to install remote observation post using CCTV camera
	All Structures	High	Currently there is no public safety around dams (PSAD) plan	Complete a PSAD risk assessment and develop a public safety plan (PSP) for Lake Geraldine Dam

3 Conclusion

The overall physical condition rating for Lake Geraldine Dam is Good. There are five (5) new deficiencies identified and an additional four (4) carried over from previous inspections. In general, the dam was not observed to be under duress or at risk of imminent failure. The principal concern is related to the dam safety management failure to actively monitor instrumentation installed in 2019.

ATTACHMENT A

Physical Inspection Report & Photolog

Lake Geraldine Dam Routine Inspection Checklist

Name of Inspector:
Perry Mitchelmore
Mark Rustan

Date of Inspection: September 18, 2024
Weather: Clear Day, 4 degrees C

<u>Instruction to Inspector:</u>
Inspector to walk the crest and the base on the concrete dam, north, centre and south berms to visually inspect and record findings below. Where changes are observed, record observations and recover a photograph. Report and photograph conditions highlighting anything that seems new or unusual.

Description	Yes	No	Comment
General Observations			
Presence of snow/ ice on ground		X	Not applicable.
Presence of snow/ ice on Lake		X	Not applicable.
Vandalism/garbage		X	None observed.
Gates locked	X		
Concrete Dam			
New cracking / spalling visible along the top of the concrete dam		X	There were two chips that may have been impact damage, minor deficiencies, no obvious scaling or spalling.
New cracking / spalling visible along the downstream side of the concrete dam	X		There was one delamination area identified that was not previously identified. Recent remedial works involving grout injection is incomplete. Grout injection tubes should be cut flush with concrete.
Presence of active leaks through concrete			No active leakage observed through the dam. A small seep at the left abutment of the spillway previously observed was unchanged.
Deteriorated/leaking sealant			Sealant and backing rod in contraction joints recently replaced but already deteriorated.
Presence of active leaks beneath concrete dam			Foundation leak at the left abutment still active. Rate of leakage is minor. Continue to observe.
Misalignment of sections of the concrete wall and spillway			No misalignment observed.

Lake Geraldine Dam Routine Inspection Checklist

Centre Berm			
Upstream Slope- Slough, slides or bulges			None observed.
Upstream Slope- Gravel washout			None observed.
Upstream Slope- Displaced Riprap			Recent riprap is small size compared with older materials. Some beaching erosion observed at the waterline.
Downstream Slope- Slough, slides or bulges			None observed.
Downstream Slope- Gravel washout			Gravel washout around and adjacent to instrumentation. May be a construction defect. Washouts do not reduce freeboard at the crest in current condition.
Downstream Slope- Displaced Riprap			None observed.
Downstream Slope- Wet area of seepage at base of slope			No wet areas observed.
Top of Berm- Sinkhole			None observed.
Top of Berm- Gravel washout			Only as noted above. Crest was intact and level during the inspection.
North Berm			
Upstream Slope- Slough, slides or bulges			None observed.
Upstream Slope- Gravel washout			None observed.
Upstream Slope- Displaced Riprap			Recent riprap is small size compared with older materials. Some beaching erosion observed at the waterline.
Downstream Slope- Slough, slides or bulges			None observed.
Downstream Slope- Gravel washout			Gravel washout around and adjacent to instrumentation. May be a construction defect. Washouts do not reduce freeboard at the crest in current condition.
Downstream Slope- Displaced Riprap			None observed.
Downstream Slope- Wet area of seepage at base of slope			No wet areas observed.
Top of Berm- Sinkhole			None observed.
Top of Berm- Gravel washout			Only as noted above. Crest was intact and level during the inspection.

Lake Geraldine Dam Routine Inspection Checklist

South Berm			
Upstream Slope- Slough, slides or bulges			None observed.
Upstream Slope- Gravel washout			None observed.
Upstream Slope- Displaced Riprap			Previously noted missing riprap is still missing.
Downstream Slope- Slough, slides or bulges			None observed.
Downstream Slope- Gravel washout			None observed.
Downstream Slope- Displaced Riprap			None observed. Utility pole removed, foundation anchor still in place.
Downstream Slope- Wet area of seepage at base of slope			No wet spots observed.
Top of Berm- Sinkhole			None observed.
Top of Berm- Gravel washout			None observed.

Table 1 Piezometer Data [No Data Available]

Date Inspector

Piezometer ID	Water Depth (m)	Comment
BH107		No instrumentation available to measure water level. Recent changes not documented.
BH101		No instrumentation available to measure water level. Recent changes not documented.
BH112		No instrumentation available to measure water level. Cap missing from Borehole collar.
BH113 (s)		No instrumentation available to measure water level. Cap missing from Borehole collar.
BH113 (d)		No instrumentation available to measure water level. Cap missing from Borehole collar.

Lake Geraldine Dam Routine Inspection Checklist

Table 2 **Thermistor Data [No Data Available]**

Date Inspector: Reviewer Date

Thermistor Bulb	Thermistor ID						
	T1	T2	T3	T4	T5	T6	T7
Air							
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
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28							
29							

Lake Geraldine Dam Routine Inspection Checklist

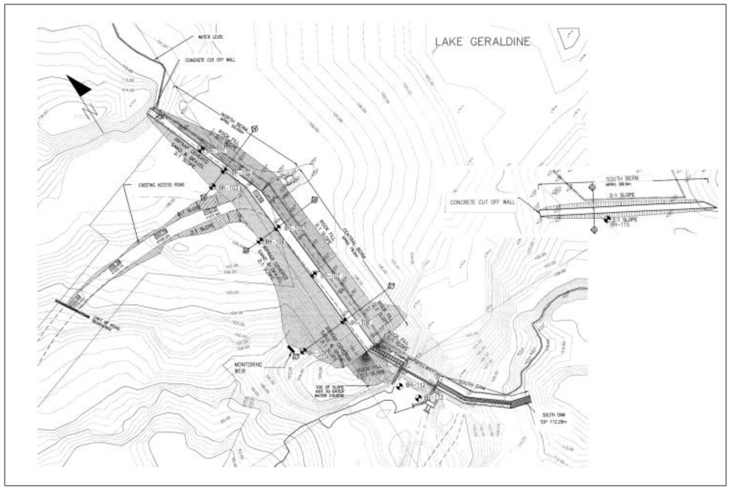


Figure 1 Instrumentation Layout

Figure 2 North Berm Abutment

Figure 3 Material Stockpile Below North Berm

Figure 4 Upstream Riprap at North Berm

Lake Geraldine Dam Routine Inspection Checklist

Figure 5 Upstream Riprap at North Berm Abutment

Figure 6 Upstream Riprap at Centre Berm

Figure 7 Upstream Limited Riprap Parapet

Figure 8 Downstream Rockfill Slope

Lake Geraldine Dam Routine Inspection Checklist

Figure 9 Upstream Rockfill – South Berm

Figure 11 Utility Pole Anchor at Toe of South Berm

Figure 10 Crest of South Berm

Figure 12 Downstream Pooled Water – Source Unknown

Figure 13 North Berm Crest BH101 and BH102

Figure 14 BH102 and BH 103 Thermistor

Figure 15 BH102 Thermistor

Figure 16 Central Berm Crest, BH 105, Rockfill Parapet

Figure 17 BH105 and Dam Access Road

Figure 18 BH105 and BH106 Thermistor

Figure 19 Spillway Spall Location

Figure 20 BH105 and BH106 Thermistor

Figure 21 BH109 Thermistor, Rock Slope

Figure 22 BH107 with Piezometer

Figure 23 South Berm, BH111 Thermistor

Figure 24 BH112, Piezometer Exposed

Figure 25 Spillway Apron, BH113 Piezometer Exposed

Figure 26 Left Abutment of South Concrete Dam

Figure 27 South Concrete Dam Tie-In to Bedrock

Figure 28 South Concrete Dam, Upstream Freeboard

Figure 29 South Concrete Dam, Downstream Face

Figure 30 South Concrete Dam

Figure 31 South Concrete Dam Intake

Figure 32 Spillway Conveyance and Training Wall

Figure 33 Intake Valve Chamber

Figure 34 Exposed Waterline to Treatment Plant

Figure 35 Discharge Channel Training Wall

Figure 36 Public Safety Interaction

Figure 37 Porosity in Spillway Conveyance Surface

Figure 38 Left Training Wall Sealant Intact

Figure 39 Protruding Caps to be Removed

Figure 40 North Concrete Dam Training Wall

Figure 41 North Concrete Dam Abutment

Figure 42 View across Crest. Unnknown Sensor

Figure 43 Waterstop at Center Berm Corewall – North Concrete Dam Interface

Figure 44 South Concrete Dam Seeping at Foundation

Figure 45 Minor Seepage at Concrete / Bedrock Fundation

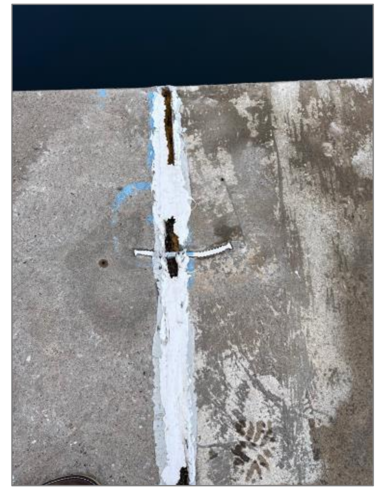


Figure 47 Expansion Joinst Sealer – Rodent Activity

Figure 46 Crack Repair – Injectors Not Cut Flush

Figure 48 Expansion Joint Sealer - Removed by Rodents

APPENDIX C

Structural Design Adequacy

TECHNICAL MEMORANDUM REPORT

Project: Lake Geraldine Dam DSR

Subject: Structure Stability Report

Author: Aidan McFarland, EIT

Reviewers: Perry Mitchelmore, P.Eng., PMP
Mark Rustan, P.Eng

Date: January 3, 2025

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 Lake Geraldine Dam and Spillway.

2.0 ANALYSIS CRITERIA

Analysis was performed using standards-based analysis (SBA) in the framework of the Canadian Dam Association (CDA) Dam Safety Guidelines. The SBA method prescribes rigid body mechanics for overturning and sliding in two-dimensional plane strain for concrete structures. 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.
- **Uplift Load (U)** - Applied to the base of the structure assuming full uplift pressure on the upstream side and tailwater below the dam. When passive anchors are required to achieve stability, a cracked base is idealized to result in full hydrostatic uplift at the base. For Lake

Geraldine Dam and Spillway, full uplift was computed from the heel of the structure up to the location of the rock anchor closest to the toe.

- **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 sum of two load events is used for earthquake loads: (1) hydrodynamic loading according to the Westergaard method, and (2) momentum horizontal loading of the mass.

The CDA Guidelines are not explicit in guidance on methods for anchored dams. Anchoring generally is either post-tensioned or passive. Post-tensioned anchors are pre-loaded and, when testing is documented, may have the full load applied in stability analysis. Passive anchors have greater uncertainty. Industry practice for evaluating passive anchors is to specify the factored strength of the dowelled anchor, generally 60 percent of the ultimate tendon yield. As a reacting load, passive anchors only resisting the tensile forces incurred due to the imposed load. In order to develop this strength, the anchor must yield and displace which can have negative consequences with respect to uplift and cracking for a dam.

3.0 Stability Analysis

3.1 Lake Geraldine Dam

A typical cross-section of the dam is shown in Figure 3.1. The first time the dam and spillway were stabilized with supplemental anchors was in 1995. The anchors, believed to be 46 mm diameter vertical rock anchor tendons were installed on the upstream side at a 2.7 metre spacing. A subsequent dam raise in 2006, the anchoring on the upstream was split and a new layer of steel tendons were installed downstream sides of the structure. Upstream anchors are referenced as Anchor 1 and downstream anchors are referenced as Anchor 2. Anchors installed in 2006 are also believed to be 46 mm diameter 1030 steel tendons without corrosion protection.

The geometric properties used for the analysis are listed in Table 3.1, with rock anchor properties listed in Table 3.2. Geometric properties were taken from DWG S3 of the *Lake Geraldine Dam – Earth & Concrete Work* drawing set prepared by Trow Associates Inc., dated February 2006. Rock anchor properties were taken from DWG RA1, RA2, and RA3 of the *Lake Geraldine Dam – Rock Anchors* drawing set prepared by Trow Associates Inc., dated May 2005.

Table 3.1 Geometric Properties of the Dam

Parameter		Value
Height of Wall		12.3 m
Base Width		7.93 m
Cross-Sectional Area		49.9 m^2
Location of Centroid from the Downstream Toe	$\bar{x}$ =	5.53 m
	$\bar{y}$ =	4.75 m
EDGM (PGA) ($AEP_{10,000}$)		0.116 g [1]

[1] NBCC, 2015, extrapolated value.

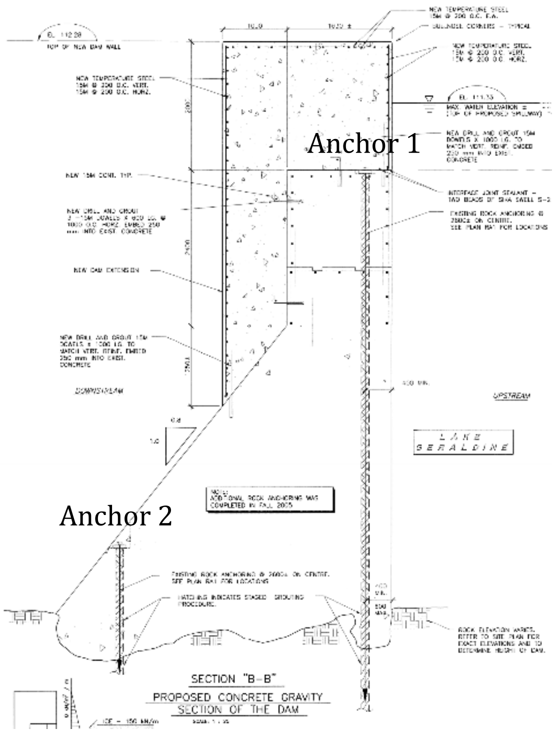


Figure 3.1 Typical Cross Section of the Dam

Table 3.2 Rock Anchor Properties of the Dam

Parameter	Anchor 1	Anchor 2
Type	Grade 835/1030 MPa	Grade 835/1030 MPa
Diameter	46 mm	46 mm
Spacing	1,300 mm c/c	2,600 mm c/c
Embedment Angle	0°	0°
Location of Anchor from the Downstream Toe	7.33 m	1.00 m
Tensile Load [1]	1,041 kN/anchor	1,041 kN/anchor

[1] Anchor loads set at 60% of ultimate strength.

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	36° [1], 46° [2]

[1] Fractured Bedrock Condition (Meco, 2020)
[2] Concrete-Rock Condition (Meco, 2020)

3.1.1 Results

Results of the sliding stability analysis are presented in Table 3.4. The dam is compliant with CDA criteria for stability provided the passive anchors are not stressed more than 60 percent. Results of the overturning analysis are presented in Table 3.5. The dam is compliant with CDA criteria for overturning using the same limitation as for sliding.

Table 3.4 Sliding Stability Analysis Results

Loading Condition	Required Factor of Safety	Fractured Bedrock ($\phi = 36°$)		Concrete-Rock ($\phi = 46°$)	
		Calculated FOS	P/F	Calculated FOS	P/F
Usual	1.5	1.86	Pass	2.00	Pass
Usual (No Ice)	1.5	2.31	Pass	2.48	Pass
Unusual (Flood)	1.3	2.07	Pass	2.20	Pass
Unusual (Wave)	1.3	2.29	Pass	2.46	Pass
Extreme (Earthquake)	1.1	1.75	Pass	1.88	Pass

Table 3.5 Overturning Stability Analysis Results

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	1.32	0.57	Pass
Usual (No Ice)	1.32	-0.49	Pass
Unusual (Flood)	1.98	-0.27	Pass
Unusual (Wave)	1.98	-0.46	Pass
Extreme (Earthquake)	3.97	0.12	Pass

3.2 Lake Geraldine Spillway

A typical cross-section of the dam is shown in Figure 3.2. The spillway was initially stabilized with 46 mm diameter angled rock anchors in 1995 on the upstream side, referred to as Anchor 1. Additional angled rock anchors were installed in 2006 on the upstream side of the structure and placed in between anchors installed in 1995. Downstream vertical anchors were also installed in 2006 and placed in with anchors installed in 1995, referred to as Anchor 2.

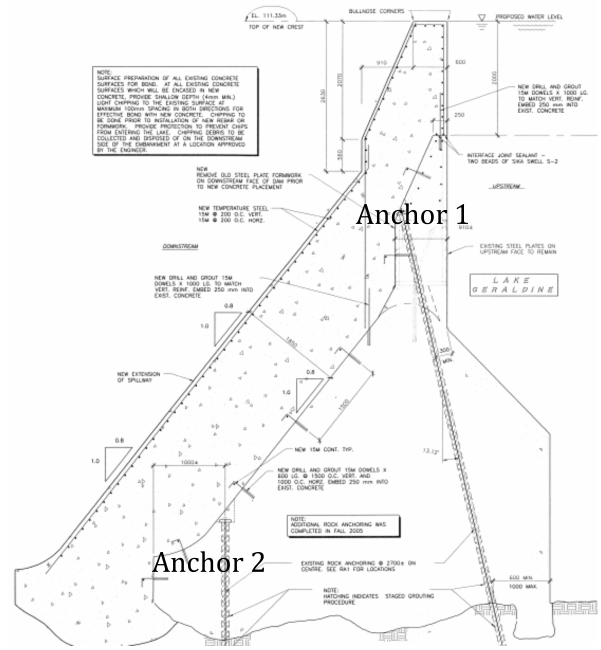


Figure 3.2 Typical Cross Section of the Spillway

The geometric properties used for the analysis are listed in Table 3.6, with passive anchor properties listed in Table 3.2. Material properties are presented in Table 3.3. Geometric properties were taken from DWG S2 of the *Lake Geraldine Dam – Earth & Concrete Work* drawing set prepared by Trow Associates Inc., dated February 2006. Rock anchor properties were taken from DWG RA1, RA2, and RA3 of the *Lake Geraldine Dam – Rock Anchors* drawing set prepared by Trow Associates Inc., dated May 2005.

Table 3.6 Geometric Properties of the Spillway

Parameter		**Value**
Height of Wall		11.3 m
Base Width		10.2 m
Cross-Sectional Area		55.8 m^2
Location of Centroid from the Downstream Toe	$\bar{x}$ =	6.23 m
	$\bar{y}$ =	3.55 m
EDGM (PGA) (AEP$_{10,000}$)		0.116 g [1]

[1] NBCC, 2015, extrapolated value.

Table 3.7 Rock Anchor Properties of the Spillway

Parameter	**Anchor 1**	**Anchor 2**
Type	Grade 835/1030 MPa	Grade 835/1030 MPa
Diameter	46 mm	46 mm
Spacing	1,350 mm c/c	2,700 mm c/c

Embedment Angle	13.1°	0°
Location of Anchor from the Downstream Toe	9.24 m	4.49 m
Tensile Load [1]	1,041 kN/anchor	1,041 kN/anchor

[1] Anchor loads set at 60% of ultimate strength.

Table 3.8 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	36° [1], 46° [2]

[1] Fractured Bedrock Condition (Meco, 2020)
[2] Concrete-Rock Condition (Meco, 2020)

3.2.1 Results

Results of the sliding stability analysis are presented in Table 3.9. The dam is compliant with CDA criteria for stability provided the passive anchors are not stressed more than 60 percent.

Table 3.9 Sliding Stability Analysis Results

Loading Condition	Required Factor of Safety	Fractured Bedrock (ϕ = 36°)		Concrete-Rock (ϕ = 46°)	
		Calculated FOS	P/F	Calculated FOS	P/F
Usual	1.5	2.39	Pass	2.60	Pass
Usual (No Ice)	1.5	3.18	Pass	3.46	Pass
Unusual (Flood)	1.3	2.72	Pass	2.93	Pass
Unusual (Wave)	1.3	3.15	Pass	3.43	Pass
Extreme (Earthquake)	1.1	2.15	Pass	2.34	Pass

Results of the overturning analysis are presented in Table 3.10. The dam is compliant with CDA criteria for stability provided the passive anchors are not stressed more than 60 percent.

Table 3.10 Overturning Stability Analysis Results

Loading Condition	Maximum Allowable Eccentricity (m)	Calculated Eccentricity (m)	Pass/Fail
Usual	1.71	0.40	Pass
Usual (No Ice)	1.71	-0.66	Pass
Unusual (Flood)	2.56	-0.48	Pass
Unusual (Wave)	2.56	-0.63	Pass
Extreme (Earthquake)	5.12	-0.12	Pass

3.3 Sensitivity Analysis – Rock Anchor Service Load

Performance of passive anchors over time cannot be guaranteed or verified. Sensitivity analysis was performed to determine conditions that might cause the dam to be at greater risk.

- The ultimate tendon capacity of a 46 mm 1030 steel tendon is 1,735 kN. A sensitivity analysis of stability indicates 560 kN of resistance, or 32 percent, is adequate to achieve the standards based criteria for overturning, which is the governing condition, at the design anchor spacing. There is sufficient redundancy on the tendon size.
- The spacing of anchors on the upstream is +/- 1.3 metres on-centre. A sensitivity analysis of stability indicates that spacing 2.5 metres is adequate to achieve the standards based criteria for overturning for the 46 mm anchor. There is sufficient redundancy on the number of tendons.

There are several potential non-structural failure modes that may impact the performance of the anchoring system at the dam and spillway. The nature of the foundation is a significant unknown as permafrost losses occur in northern environments. The mechanics of permafrost loss and load transfer in the foundation cannot be verified directly.

4.0 Summary & Recommendations

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, when anchors are activated to 60% of their ultimate tensile strength. Sensitivity analysis determined that the dam and spillway no longer meet criteria if rock anchors are simply activated up to their specified service loads.

Since the structure complies with stability requirements only if rock anchor service loads are exceeded, there is increased susceptibility for the structure to fail due to grout bonding or bedrock failure resulting in anchor pull-out. This scenario will cause the structure to uplift and introduce additional sliding effects. Therefore, concrete cracking and mass movement of the structure should be inspected regularly to determine if there is a potential risk for bonding failure or rock pull-out.

Attachment A

Stability Calculations

Technical Analysis Procedure
Concrete Spillway Stability
Lake Geraldine Spillway

Purpose

General stability assessment

Assumptions

- Design section on drawings represent the actual section
- Cracked base from heel up to location of Anchor 2
- Base level set at average bedrock elevation determined by Canadrill
- MFL and TWF retrieved from previous hydrotechnical study conducted by Meco
- Shear capacity of rock anchors taken as 60 % ultimate load

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. Lake Geraldine Spillway - Dam and Spillway Modifications (Drawing Set) (Trow, 2006)
4. Technical Analysis & Risk Assessment (Meco, 2020)
5. Williams Form Engineering Corp - Grout Bonded Anchors

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	36	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

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

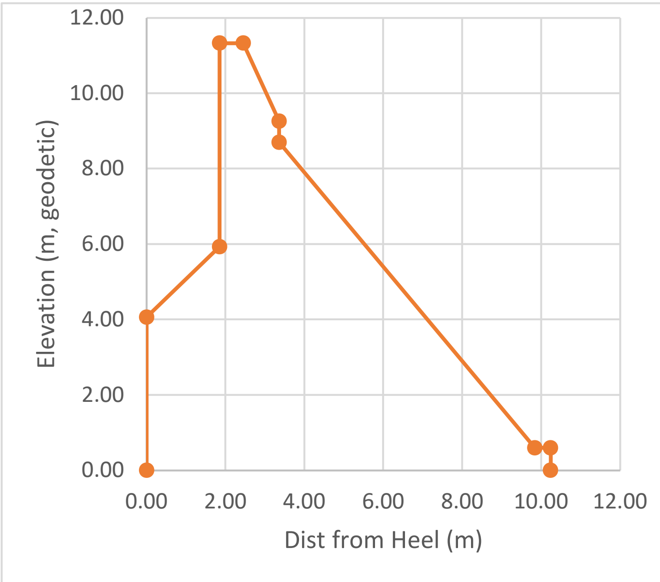
Where:
xr = Location of resultant force (m)
Mr = Resistant moment (kN·m)
Mo = Overturning moment (kN·m)
Pv = Sum of vertical forces (kN)

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
Concrete Spillway Stability
Lake Geraldine Spillway



X (m)	Y (m)					
0.00	0.00	Normal Operating Level	NOL	111.33		3.78
0.00	4.07	Maximum Flood Level	MFL	111.90		3.97
1.85	5.93	Normal Tailwater Level	TWN	100.00		0.00
1.85	11.33	Flood Tailwater Level	TWF	101.33		0.44
2.45	11.33	Wave Height	WH	0.60		11.56
3.36	9.26	Soil $_{\text{Upstream}}$	S1H	100.00		0.00
3.36	8.70	Soil $_{\text{Downstream}}$	S2H	100.00		0.00
9.84	0.60					
10.24	0.60	Top of Dam	CL	111.33	m	
10.24	0.00	Base Level	BL	100.00	m	
		Height of wall	H =	11.33	m	
		Base width	B =	10.24	m	
		$\text{Uplift}_{\text{NOL}}$				6.8
		$\text{Uplift}_{\text{MFL}}$				6.8

Centre of Gravity (m)	**4.01**	**3.55**
Area of Mass (m^2)	55.78	
Base Width (m)	10.24	

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		Anchor 1	Anchor 2
	Tension	1041	1041
	Angle (°)	13.1	0.0
	Dist From Toe (m)	9.24	4.49
	Spacing (m)	1.35	2.7

EDGM (PGA) (1/10000yr) 0.116

(1) Usual Load Condition

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-111.1	-629.6	-2378.0
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At Rest	0.0	0.0	0.0
Ice	Horizontal		-150.0	-1654.5
Anchor 1 Tension	Horizontal		175.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Normal Horiz			**-604.6**	**-4032.5**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	55.78	128.1	1312.0	8173.6
Anchor 1 Tension	Vertical		751.0	6939.1
Anchor 2 Tension	Vertical		385.6	1731.1
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal(Full)}$		-111.1	-639.1	-4706.9
$Uplift_{Normal(Remaining)}$		-111.1	-249.5	-746.9
Normal			**1560.0**	**11390.0**
FoS Sliding	2.39		ΣMo	9486.3
			ΣMr	16843.8
			Resultant (m)	4.72
			Eccentricity (*e*)	0.40
		+/-	e_{max}	1.71

(1a) Usual Load Condition (No Ice)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-111.1	-629.6	-2378.0
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At-Rest	0.0	0.0	0.0
Anchor 1 Tension	Horizontal		175.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Normal Horiz			**-454.6**	**-2378.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	55.78	128.1	1312.0	8173.6
Anchor 1 Tension	Vertical		751.0	6939.1
Anchor 2 Tension	Vertical		385.6	1731.1
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal(Full)}$		-111.1	-639.1	-4706.9
$Uplift_{Normal(Remaining)}$		-111.1	-249.5	-746.9
Normal			**1560.0**	**11390.0**
FoS Sliding	3.18		ΣMo	7831.8
			ΣMr	16843.8
			Resultant (m)	5.78
			Eccentricity (*e*)	-0.66
		+/-	e_{max}	1.71

(2) Unusual Load Condition (IDF)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Unusual H_2O	Hydrostatic_{MFL}		-116.7	-694.6	-2755.2
Unusual H_2O	Hydrostatic_{TWF}		13.0	8.7	3.8
$\text{Soil}_{Upstream}$	Active		0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest		0.0	0.0	0.0
Anchor 1 Tension	Horizontal			175.0	0.0
Anchor 2 Tension	Horizontal			0.0	0.0
Sum Unusual Horiz				**-510.9**	**-2751.4**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		55.78	128.1	1312.0	8173.6
Anchor 1 Tension	Vertical			751.0	6939.1
Anchor 2 Tension	Vertical			385.6	1731.1
$\text{Uplift}_{Flood(TW)}$			-13.0	-133.6	-684.1
Uplift_{Flood}			-103.7	-596.2	-4391.2
$\text{Uplift}_{Flood(Remaining)}$			-103.7	-232.8	-696.8
Unusual				**1486.0**	**11071.7**
FoS Sliding		2.72		ΣMo	8527.3
				ΣMr	16847.7
				Resultant (m)	5.60
				Eccentricity (*e*)	-0.48
			+/-	e_{max}	2.56

(3) Unusual Load Condition (Temperature / Wave)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	Hydrostatic_{NOL}		-111.1	-629.6	-2378.0
Normal H_2O	Hydrostatic_{TWN}		0.00	0.0	0.0
Normal H_2O	Hydrodymanic		-7.1	-4.2	-48.0
$\text{Soil}_{Upstream}$	Active		0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest		0.0	0.0	0.0
Anchor 1 Tension	Horizontal			175.0	0.0
Anchor 2 Tension	Horizontal			0.0	0.0
Sum Normal Horiz				**-458.9**	**-2426.0**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		55.78	128.1	1312.0	8173.6
Anchor 1 Tension	Vertical			751.0	6939.1
Anchor 2 Tension	Vertical			385.6	1731.1
$\text{Uplift}_{Normal(TW)}$			0.0	0.0	0.0
$\text{Uplift}_{Normal(Full)}$			-111.1	-639.1	-4706.9
$\text{Uplift}_{Normal(Remaining)}$			-111.1	-249.5	-746.9
Normal				**1560.0**	**11390.0**
FoS Sliding		3.15		ΣMo	7879.9
				ΣMr	16843.8
				Resultant (m)	5.75
				Eccentricity (*e*)	-0.63
			+/-	e_{max}	3.41

(4) Extreme Load Condition (Seismic)

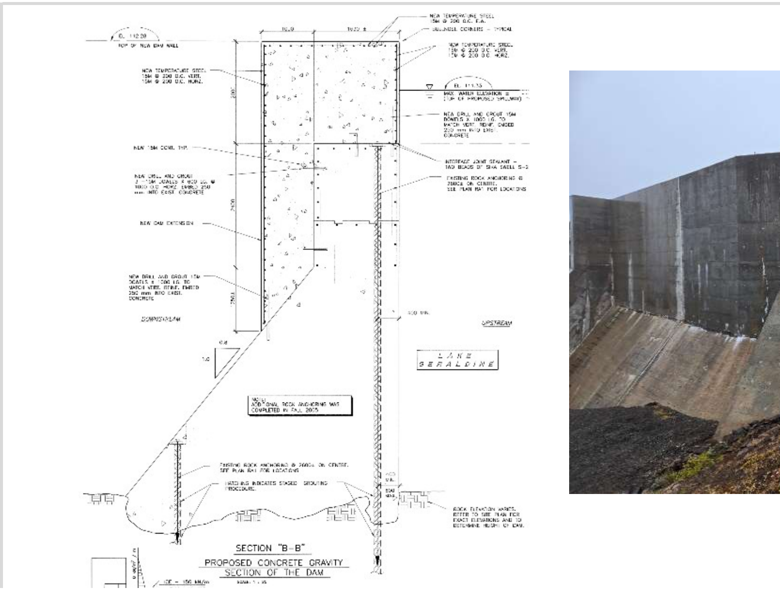
Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$		-111.1	-629.6	-2378.0
Normal H_2O	$Hydrostatic_{TWN}$		0.0	0.0	0.0
$Soil_{Upstream}$	Active		0.0	0.0	0.0
$Soil_{DownStream}$	At Rest		0.0	0.0	0.0
Anchor 1 Tension	Horizontal			175.0	0.0
Anchor 2 Tension	Horizontal			0.0	0.0
Pseudostatic		55.78	-1312.0	-151.8	-539.1
Westergaard			-5.81	-65.8	-298.4
Sum Seismic Horiz				**-672.3**	**-3215.5**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		55.78	128.1	1312.0	8173.6
Anchor 1 Tension	Vertical			751.0	6939.1
Anchor 2 Tension	Vertical			385.6	1731.1
$Uplift_{Normal(TW)}$			0.0	0.0	0.0
$Uplift_{Normal(Full)}$			-111.1	-639.1	-4706.9
$Uplift_{Normal(Remaining)}$			-111.1	-249.5	-746.9
Sum Seismic Vert				**1560.0**	**11390.0**

FoS Sliding		2.15		ΣMo	8669.4
				ΣMr	16843.8
				Resultant (m)	5.24
				Eccentricity (e)	-0.12
			+/-	e_{max}	5.12

Technical Analysis Procedure
Concrete Spillway Stability
Lake Geraldine Spillway

Load Condition	Horizontal		Vertical		Sliding Fos			Overturning		
	Load	Moment	Load	Moment	Req'd	Actual	Result	Req'd	Actual	Result
Usual	-604.6	-4032.5	1560.0	11390.0	1.5	2.39	OK	1.71	0.40	OK
Usual (No Ice)	-454.6	-2378.0	1560.0	11390.0	1.5	3.18	OK	1.71	-0.66	OK
Unusual (Flood)	-510.9	-2751.4	1486.0	11071.7	1.3	2.72	OK	2.56	-0.48	OK
Unusual (Wave)	-458.9	-2426.0	1560.0	11390.0	1.3	3.15	OK	2.56	-0.63	OK
Extreme (Earthquake)	-672.3	-3215.5	1560.0	11390.0	1.1	2.15	OK	5.12	-0.12	OK



Purpose

General stability assessment

Assumptions

- Design section on drawings represent the actual section
- Cracked base from heel up to location of Anchor 2
- Base level set at average bedrock elevation determined by Canadrill
- MFL and TWF retrieved from previous hydrotechnical study conducted by Meco
- Shear capacity of rock anchors taken as 60 % ultimate load

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. Lake Geraldine Spillway - Dam and Spillway Modifications (Drawing Set) (Trow, 2006)
4. Technical Analysis & Risk Assessment (Meco, 2020)
5. Williams Form Engineering Corp - Grout Bonded Anchors

Prepared by:__________________ Date:____________ Manager:______________________

Technical Analysis Procedure
Concrete Dam Stability
Lake Geraldine 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
U	- Hydrostatic Uplift Pressure	Unit Weight of Rock =	25	kN/m^3
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

30 degrees
36 degrees

Horizontal Thrust

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

0.33 $\text{Rankine}_{\text{Activ}}$
3 $\text{Rankine}_{\text{Pass}}$
0.5 $\text{Rankine}_{\text{AR}}$

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

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

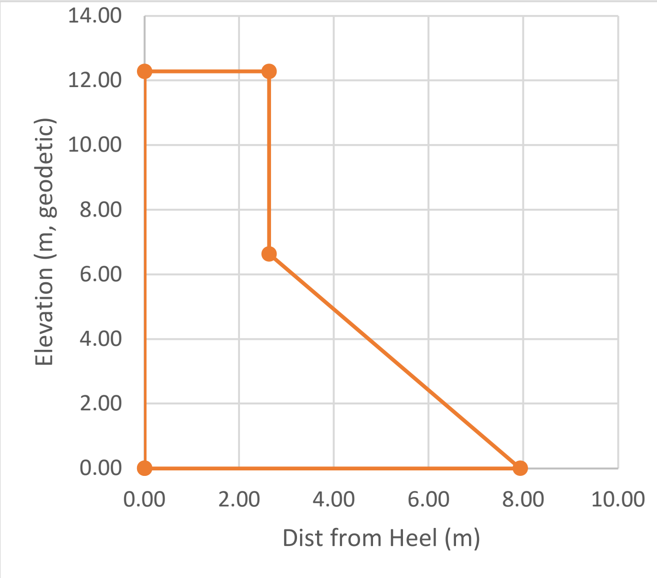
Where:
xr = Location of resultant force (m)
Mr = Resistant moment (kN·m)
Mo = Overturning moment (kN·m)
Pv = Sum of vertical forces (kN)

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
Concrete Dam Stability
Lake Geraldine Dam



X (m)	Y (m)				
0.00	0.00	Normal Operating Level	NOL	111.33	3.78
0.00	12.28	Maximum Flood Level	MFL	111.90	3.97
2.63	12.28	Normal Tailwater Level	TWN	100.00	0.00
2.63	6.63	Flood Tailwater Level	TWF	101.33	0.44
7.93	0.00	Wave Height	WH	0.60	11.56
0.00	0.00	Soil $_{Upstream}$	S1H	100.00	0.00
		Soil $_{Downstream}$	S2H	100.00	0.00

Top of Dam	CL	112.28	m
Base Level	BL	100.00	m
Height of wall	H =	12.28	m
Base width	B =	7.93	m

$Uplift_{NOL}$	5.3
$Uplift_{MFL}$	5.3

Centre of Gravity (m)	**2.40**	**4.75**
Area of Mass (m^2)	49.88	
Base Width (m)	7.93	

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		Anchor 1	Anchor 2
	Tension	1041	1041
	Angle (°)	0.0	0.0
	Dist From Toe (m)	7.334	1
	Spacing (m)	1.3	2.6

EDGM (PGA) (1/10000yr) 0.116

(1) Usual Load Condition

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-111.1	-629.6	-2378.0
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At Rest	0.0	0.0	0.0
Ice	Horizontal		-150.0	-1654.5
Anchor 1 Tension	Horizontal		0.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Normal Horiz			**-779.6**	**-4032.5**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	49.88	147.9	1173.2	6490.2
Anchor 1 Tension	Vertical		800.8	5872.8
Anchor 2 Tension	Vertical		400.4	400.4
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal(Full)}$		-111.1	-770.7	-3442.7
$Uplift_{Normal(Remaining)}$		-111.1	-55.6	-37.0
Normal			**1548.0**	**9283.6**
FoS Sliding	1.86		ΣMo	7512.2
			ΣMr	12763.4
			Resultant (m)	3.39
			Eccentricity (*e*)	0.57
		+/-	e_{max}	1.32

(1a) Usual Load Condition (No Ice)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	$Hydrostatic_{NOL}$	-111.1	-629.6	-2378.0
Normal H_2O	$Hydrostatic_{TWN}$	0.0	0.0	0.0
$Soil_{Upstream}$	Active	0.0	0.0	0.0
$Soil_{DownStream}$	At-Rest	0.0	0.0	0.0
Anchor 1 Tension	Horizontal		0.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Normal Horiz			**-629.6**	**-2378.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	49.88	147.9	1173.2	6490.2
Anchor 1 Tension	Vertical		800.8	5872.8
Anchor 2 Tension	Vertical		400.4	400.4
$Uplift_{Normal(TW)}$		0.0	0.0	0.0
$Uplift_{Normal(Full)}$		-111.1	-770.7	-3442.7
$Uplift_{Normal(Remaining)}$		-111.1	-55.6	-37.0
Normal			**1548.0**	**9283.6**
FoS Sliding	2.31		ΣMo	5857.7
			ΣMr	12763.4
			Resultant (m)	4.46
			Eccentricity (*e*)	-0.49
		+/-	e_{max}	1.32

(2) Unusual Load Condition (IDF)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Unusual H_2O	Hydrostatic_{MFL}	-116.7	-694.6	-2755.2
Unusual H_2O	Hydrostatic_{TWF}	13.0	8.7	3.8
$\text{Soil}_{Upstream}$	Active	0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest	0.0	0.0	0.0
Anchor 1 Tension	Horizontal		0.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Unusual Horiz			**-685.9**	**-2751.4**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	49.88	147.9	1173.2	6490.2
Anchor 1 Tension	Vertical		800.8	5872.8
Anchor 2 Tension	Vertical		400.4	400.4
$\text{Uplift}_{Flood(TW)}$		-13.0	-103.5	-410.7
Uplift_{Flood}		-103.7	-719.0	-3211.8
$\text{Uplift}_{Flood(Remaining)}$		-103.7	-51.8	-34.6
Unusual			**1500.0**	**9106.4**
FoS Sliding	2.07		ΣMo	6412.2
			ΣMr	12767.2
			Resultant (m)	4.24
			Eccentricity (*e*)	-0.27
		+/-	e_{max}	1.98

(3) Unusual Load Condition (Temperature / Wave)

Horizontal Loads	Name	Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	Hydrostatic_{NOL}	-111.1	-629.6	-2378.0
Normal H_2O	Hydrostatic_{TWN}	0.00	0.0	0.0
Normal H_2O	Hydrodymanic	-7.1	-4.2	-48.0
$\text{Soil}_{Upstream}$	Active	0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest	0.0	0.0	0.0
Anchor 1 Tension	Horizontal		0.0	0.0
Anchor 2 Tension	Horizontal		0.0	0.0
Sum Normal Horiz			**-633.9**	**-2426.0**

Vertical Loads	Load	Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass	49.88	147.9	1173.2	6490.2
Anchor 1 Tension	Vertical		800.8	5872.8
Anchor 2 Tension	Vertical		400.4	400.4
$\text{Uplift}_{Normal(TW)}$		0.0	0.0	0.0
$\text{Uplift}_{Normal(Full)}$		-111.1	-770.7	-3442.7
$\text{Uplift}_{Normal(Remaining)}$		-111.1	-55.6	-37.0
Normal			**1548.0**	**9283.6**
FoS Sliding	2.29		ΣMo	5905.7
			ΣMr	12763.4
			Resultant (m)	4.43
			Eccentricity (*e*)	-0.46
		+/-	e_{max}	2.64

(4) Extreme Load Condition (Seismic)

Horizontal Loads	Name		Pressure (kPa)	Force (kN)	Moment (kN*m)
Normal H_2O	Hydrostatic_{NOL}		-111.1	-629.6	-2378.0
Normal H_2O	Hydrostatic_{TWN}		0.0	0.0	0.0
$\text{Soil}_{Upstream}$	Active		0.0	0.0	0.0
$\text{Soil}_{DownStream}$	At Rest		0.0	0.0	0.0
Anchor 1 Tension	Horizontal			0.0	0.0
Anchor 2 Tension	Horizontal			0.0	0.0
Pseudostatic		49.88	-1173.2	-135.7	-645.4
Westergaard			-5.81	-65.8	-298.4
Sum Seismic Horiz				**-831.2**	**-3321.8**

Vertical Loads	Load		Pressure (kPa)	Force (kN)	Moment (kN*m)
Mass		49.88	147.9	1173.2	6490.2
Anchor 1 Tension	Vertical			800.8	5872.8
Anchor 2 Tension	Vertical			400.4	400.4
$\text{Uplift}_{Normal(TW)}$			0.0	0.0	0.0
$\text{Uplift}_{Normal(Full)}$			-111.1	-770.7	-3442.7
$\text{Uplift}_{Normal(Remaining)}$			-111.1	-55.6	-37.0
Sum Seismic Vert				**1548.0**	**9283.6**
FoS Sliding		1.75		ΣMo	6801.5
				ΣMr	12763.4
				Resultant (m)	3.85
				Eccentricity (e)	0.12
			+/-	e_{max}	3.97

Load Condition	Horizontal		Vertical		Sliding Fos			Overturning		
	Load	Moment	Load	Moment	Req'd	Actual	Result	Req'd	Actual	Result
Usual	-779.6	-4032.5	1548.0	9283.6	1.5	1.86	OK	1.32	0.57	OK
Usual (No Ice)	-629.6	-2378.0	1548.0	9283.6	1.5	2.31	OK	1.32	-0.49	OK
Unusual (Flood)	-685.9	-2751.4	1500.0	9106.4	1.3	2.07	OK	1.98	-0.27	OK
Unusual (Wave)	-633.9	-2426.0	1548.0	9283.6	1.3	2.29	OK	1.98	-0.46	OK
Extreme (Earthquake)	-831.2	-3321.8	1548.0	9283.6	1.1	1.75	OK	3.97	0.12	OK