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CITY OF IQALUIT DAM SAFETY REVIEW FOR LAKE GERALDINE DAM

Trow Consulting Engineers Ltd.

P.O. Box 6
Building 1080
Iqaluit, Nunavut. XOA OXO
e-mail: trow@trowot.com
Phone: (800) 514-9044
Fax: (613)-225-7337

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Executive Summary

The City Of Iqaluit retained Trow Consulting Engineers Ltd., (Trow), in October 2001, to prepare a Dam Safety Review (DSR) for the Lake Geraldine Dam.

The DSR was conducted in October and November 2001, in accordance with the Dam Safety Guidelines, (DSG) prepared by the Canadian Dam Association.

As a result of the DSR, the following conclusions and recommendations have been made regarding Lake Geraldine Dam:

1. The dam has been classified as having a High Consequence Category.

2. The concrete gravity sections of the dam is in a safe and serviceable condition at this time, and are in general compliance with the required design and performance standards of the DSG, where applicable and appropriate for the structure. Safety improvements are not required at this time.

3. The embankment section of the dam is in a serviceable condition at this time, however, concerns exist with the future stability of the steeper sloped areas. This portion of the dam does not meet the design and performance standards of Sections 5 and 8 of the DSG. Specifically, factors of safety for dynamic and static stability are significantly lower than required minimums, particularly on the steeper upstream face of the embankment. It is recommended that the design of safety improvements be undertaken, for implementation in 2002.

4. The entire dam is in non-compliance with the requirements of Sections 3 and 4 of the DSG. The following documents do not exist at this time:

   - Permanent File
   - OMS Manual
   - Logbook
   - Emergency Preparedness Plan

   The above documents need to be developed and maintained. An understanding to proceed with the creation of these documents should be demonstrated as soon as practically possible.

5. The structures should have a Dam Safety Inspection (DSI) conducted in 2002. This is essentially a yearly non-invasive review comprising a visual inspection to identify any changes in condition, or any observed concerns.

6. An underwater inspection of the submerged structures should be done in 2002. This inspection should be coordinated with, and be under the direction of, the DSI recommended in Item 5, above.

1. Introduction & Scope Of Work

The Nunavut Water Board, as a condition of the City's water licence, required the City Of Iqaluit to complete an inspection of the Lake Geraldine Dam in accordance with the Canadian Dam Association publication, Dam Safety Guidelines (DSG), published in January 1999.

The DSG applies to those structures that are at least 2.5 meters in height, and which have at least 30,000 cubic meters of storage capacity. The Lake Geraldine Dam exceeds these minimums.

The DSG document is far reaching in terms of applicability and requirements for conformance. This is understandable as the type and complexity of structures that fall under the jurisdiction of the document varies considerably, from relatively small and simple embankments or dikes to massive and complex dams associated with hydroelectric generating facilities, irrigation, flood control, etc.

The DSG requires that all structures exceeding the height and volume minimums described above be classified according to their "consequence category", that is, the consequence of dam failure in terms of life safety, and socio-economic impact. The category assigned may range from very low to very high. The consequence category dictates the requirement and frequency of Dam Safety Reviews.

A Dam Safety Review (DSR) is a comprehensive, formal review process, conducted at regular intervals, that involves completion of checklist items in accordance with the Dam Safety Guidelines. The DSR forms a baseline of dam history, condition, repair requirements, and extensive documentation of monitoring, operating, safety and emergency procedures.

The frequency of DSR's varies depending on consequence category. For structures where significant life safety and/or socio-economic consequence exist, the DSR is usually conducted every five (5) to ten (10) years. In the case of the Lake Geraldine Dam, the required interval is seven (7) years.

It is required in the DSG document that in the interval between DSR's, a Dam Safety Inspection (DSI) would be performed on an annual basis. The DSI is a much less comprehensive review, comprising a visual inspection to identify any changes in condition, or any observed concerns. The results of the DSI are incorporated into the DSR documentation. A DSI may trigger repairs, or changes in standard operating procedures.

Under the current DSG, a DSR was required for the Lake Geraldine Dam simply because no previous DSR exists.

The level of detail required to conduct a DSR is influenced by several factors as follows:

- Importance of structure
- Complexity of structure

- Consequences of failure
- Completeness, continuity, and availability of record documentation
- Current condition

Reasonably extensive documentation exists for the Lake Geraldine Dam, however, as this will be the benchmark document for any subsequent inspection, we have included a complete review of the required tasks.

A summary of the methodology to complete the work is presented below:

1. Acquired and assembled chronological documentation, including but not limited to:

    - Design Documents
    - Repair Specifications
    - Past Condition Assessment Reports
    - Records of Alteration

    The bulk of the record documents were retrieved from the City records.

2. Reviewed all available record documentation.

3. Performed a site inspection to assess the current condition of the structures. No invasive work was performed; the condition assessment was visual in nature. Invasive assessment was not believed necessary given the amount of record documentation available.

4. Interviewed maintenance and management personnel as required and appropriate.

5. Executed the DSR checklist of items.

6. Preparation of the draft DSR report, complete with site surveys, photographs, structural sections, field notes, discussions and recommendations as required and appropriate. Submission to and discussion with the City Engineer.

7. Submission of the final DSR report.

2. History & Background

In the following chronological summary, record documents have been referenced. After each reference, a number appears in parenthesis. That number corresponds to tabulated record document numbers in Section 4, where details are provided on the document source.

2.1 Reservoir

The City of Iqaluit derives its water supply from Lake Geraldine, which is retained by a structure consisting of a cast in place concrete gravity dam incorporating a spillway section and a cast in place concrete cut-off wall and embankment. All concrete structures are believed to be founded on rock, and engage rock at their abutments.

Lake Geraldine is a natural body of water in an irregularly shaped basin. It is fed by rainfall and snow/ice melt from a watershed with an area of approximately 385 hectares.

2.2 History

In the late 1950's, the demand for a reliable year round source of water resulted in the construction of a cast in place concrete gravity dam, and a section of earth berm with a central cast in place concrete cut off wall. The project was designed and built by the Department Of National Defense. According to the literature, the original construction took place circa 1958.

Since that time, as the City has grown and water demands have risen, the dam has been raised three times to increase the storage capacity.

The first height increase of 0.3m reportedly took place in 1979. This involved a concrete extension, which was dowelled into the existing structure.

The second construction took place in 1985, and increased the height of the spillway structure by approximately 1.15m. The embankment portion was widened aand heightened as well to accommodate the increased storage capacity. Again, the extension was constructed of concrete dowelled into the existing structure, and incorporated a steel formwork frame over the spillway section, which remains to this day.

The third extension was done in 1995, and increased the height of the gravity dam structure by a further 1.5m of concrete, with a corresponding increase in berm geometry. Based on analysis done prior to the extension, it was determined that the gravity dam would not have an adequate factor of safety against overturning if the extension was simply "dowelled-in" as before. The 1995 alteration therefore included an extensive rock-anchoring program for the gravity dam portion to provide the required stability to the structure.

In the time span of the available historical data, which extends back to 1984, there have been only a few notable events relating to the safety and serviceability of the dam structures.

- In November 1984 joint and patch repairs were made to localized areas on the upstream side of the spillway structure by diving contractors. Reporting was minimal.

- In June 1990 an inspection report (3) of the structure by diving contractors was made following construction blasting. The 1984 repair areas were also assessed. The 1984 repairs were noted to have generally deteriorated. No conclusions were made. Reporting was minimal.

- In June 1990 a visual inspection report (4) was prepared for the City by an engineering consultant, as a result of the construction blast. No significant damage was noted, and no recommendations were made for repair.

- In July 1990 a dam inspection and stability report (5,6) was conducted for the City by an engineering consultant. Recommendations were made regarding repair of leaking joints, and provisions to increase stability should the dam be raised in the future.

- In September 1990, a diving contractor performed crack repairs and prepared an inspection report (7). Repair material used was oakum. These repairs appeared to generally address areas observed in the June 1990 diving inspection. Reporting was minimal.

- In October 1997 a visual inspection report (10) was prepared for the City by an engineering consultant. Leaking cracks were identified, however, these were not viewed as being structurally significant. It was recommended that leaking cracks be chemically grouted. This work was not done.

- In June 1998, a study (11) was prepared for the Department Of Public Works by an engineering consultant to assess the hydrological impact of a dam failure on a proposed downstream hospital site.

- A diving inspection was reportedly carried out in 1999. A report was not submitted. A video record was provided. The video provides images of the water intake, but no record of the condition of the dam.

2.3 Description Of Structure

A general arrangement drawing of the dam is provided in Appendix 4 of this report. The concrete sections on either side of the spillway have an elevation of 110.28 m, with the crest of the 15.7 m wide spillway section being at elevation of approximately 109.33 m, which represents the normal operating level of the lake. At this level, the dam has approximately 0.95 m of freeboard. The southern section of the dam extends approximately 32.7 m to the south rock abutment. The northern concrete section of the dam extends 13.3 m to the north of the spillway section, where it joins an extended, narrow (~200 mm wide) concrete cut-off wall, which is supported on both sides by a sand & gravel and rock fill embankment. The embankment cut-off wall extends approximately 80 m to the north rock abutment. The concrete dam and the concrete cut-off wall are reportedly founded on bedrock. The top of the concrete cut-off wall is reportedly at elevation 110.30 with the top of the embankment being at elevation 110.50m. It is therefore our understanding that the elevation of the top of the concrete cut-off wall in the embankment portion of the dam is at least equal to the elevation of the top of the concrete gravity dam portion of the dam.

2.4 Relevant Record Documents

The following documentation has been utilized in the preparation of this report. Other record documentation was provided but not directly applicable to the DSR.

TABLE I
RELEVANT RECORD DOCUMENTATION
LAKE GERALDINE DAM

No.	Date	Description	Author
1	December 1957	Water Storage Dam at Lake Geraldine (3 Drawings)	DND
2	August 1984	Lake Geraldine Water Supply Report	OMM
3	January 1985	Water Supply Improvement Report	OMM
4	June 1990	General Diving Report	Arctic Divers
5	June 1990	Dam Inspection for Blast Damage	Hardy BBT
6	July 1990	Dam Inspection & Leakage Repair	Acres
7	July 1990	Dam Stability	Acres
8	Sept. 1990	Diving Report	Arctic Divers
9	Feb. 1995	Lake Geraldine Storage Report	OMM
10	June 1995	Lake Geraldine Storage Design Dwgs. & Specifications	OMM
11	October 1997	Dam Inspection	Trow
12	June 1998	Dam Failure Study	EBA

3. Commentary On Dam Safety Review Requirements

According to the Dam Safety Guidelines, the document applies to those structures that are at least 2.5 meters in height, and which have at least 30,000 cubic meters of storage capacity. The Lake Geraldine Dam exceeds these minimums.

The Dam Safety Guidelines document is far reaching in terms of applicability and requirements for conformance. This is understandable as the type and complexity of structures that fall under the jurisdiction of the document varies considerably, from relatively small and simple embankments or dikes to massive and complex dams associated with hydroelectric generating facilities, irrigation, flood control, etc.

The document requires a systematic checklist review, which includes the following items. For each item, the applicable Section number from the Dam Safety Guidelines is shown in parenthesis.

1. Dam Classification (1)
2. Site Inspection (2)
3. Design & Construction Review: (2)
   - 3.1. Earthquakes (5)
   - 3.2. Floods (6)
   - 3.3. Discharge Facilities (7)
   - 3.4. Geotechnical Considerations (8)
   - 3.5. Concrete Structures (9)
   - 3.6. Reservoir & Environment (10)
   - 3.7. Construction (11)
4. Operation & Testing (2,3)
5. Maintenance (2,3)
6. Surveillance & Monitoring (2,3)
7. Emergency Preparedness (2,4)
8. Compliance With Previous Reviews (2)

4. Lake Geraldine Dam DSR

DAM CLASSIFICATION (DSG SECTION 1)

Based on the Dam Safety Guidelines, and the dam structure itself, the Lake Geraldine Dam has a consequence category of "High" for both the *Life Safety* and *Socioeconomic, Financial and Environmental* categories. The "High" classification is assigned by the DSG, in the case of life safety, if loss of life would likely occur as a result of dam failure. A "High" category is assigned in the socioeconomic category if, in the event of a dam failure, the cost to the community in terms of social and financial impact would be significant. Under the guidelines it is required to have a Dam Safety Review every seven (7) years for those structures with a high consequence category.

SITE INSPECTION (DSG SECTION 2)

A visual site inspection of the dam structures was performed in October 2001. The inspection was non-invasive in nature, and did not include an underwater assessment. A photographic and video record of our inspection was made, and appends this report.

A summary of observed conditions is as follows:

- The visible portions of the concrete structures are generally in good condition with minor concrete durability issues, such as localized scaling and spalling, visible. In addition, a few localized areas exhibited concrete spalling due to embedded formwork remnants.

- Several actively seeping cracks were observed. These cracks are generally vertical in orientation and hairline in width, as would be expected for shrinkage (non-structural) cracking. The location and extent of shrinkage cracking has not changed significantly since the 1997 inspection. The recommendation from the 1997 report to chemically grout actively leaking cracks was not carried out.

- The rate of leakage through the control joint south of the spillway section has not changed significantly since the 1997 inspection. Some digging is required here to expose the joint down to rock and determine whether an upstream or downstream repair is likely to be more successful.

- There was no evidence of distress or overstressing of any portion of the visible concrete structures.

- Minor corrosion of the 1985 spillway extension frame. As this frame was primarily used for forming, cleaning and re-painting are considered to be desirable, albeit cosmetic repairs.

- The north cut-off wall, including the embankment section, appears to be founded on bedrock. The crest width of the embankment is approximately 4.5 m to 5.0 m, with a wider section near the north abutment. The crest material consists of granular material.

- The upstream side slopes of the embankment section consist of 300 mm to 600 mm sized rock fill, placed at a relatively flat slope near the northern abutment but at a much steeper angle near the interface with the concrete gravity section of the dam (i.e. slope angles appeared to be approximately at 1.25H:1.0V). An approximately 20 m length of the upstream slope, just north of the concrete gravity dam terminus, extending ~ 1.0 m to 1.5 m into the crest, has slumped by approximately 100 mm. This is believed to be a result of the overly steep side slopes in this area.

- The downstream slope of the embankment section ranges from approximately 1.5H:1V at the deepest section to 3H:1V in the shallow areas. The downstream face is protected with a layer of minus 300 mm sized rock fill.

Design & Construction Review (DSG Section 2)

This section constitutes the bulk of the Dam Safety Review process. The intent is to determine if the existing dam configuration satisfies performance criteria given in the DSG for safety and serviceability, in response to likely loads impinging on the structure. We have followed the format in the DSG document for convenience and clarity.

Earthquakes (DSG Section 5)

According to the DSG, dams shall be evaluated to withstand a Maximum Design Earthquake (MDE) without release of the reservoir. For a High Consequence Category, the DSG requires evaluation at 50% - 100% of the Maximum Credible Earthquake (MCE). To paraphrase the DSG, the MCE is defined as the largest reasonably conceivable earthquake that appears possible under the presently known tectonic framework.

Concrete Gravity Dam Portion

For the concrete portions of the dam, two sections were assessed; the spillway section and the gravity dam itself. For each section, the worst case was assessed, which corresponded to the maximum retained height, that is, the maximum distance from the lake bottom at the base of the dam, to the crest of the dam. As the lake bottom tends to undulate, the retained height varies to some degree.

Our analysis has based the MCE on statistical seismic data in the National Building Code (NBC) 1997, specifically zonal velocities and accelerations for the Iqaluit area. The calculation involves deriving a resultant force proportional to the mass of the structure, and includes an allowance for the inertial effect of the retained water. We have used 100% of the calculated MCE as our Maximum Design Earthquake. That value is 68 kN per meter width of the gravity dam section, and 73 kN per meter width of the spillway section. Summary calculations append this report.

The MDE loads were applied in combination with other loads in accordance with Section 9.4 of the DSG, Load Combinations, Concrete Structures.

In the load combination case involving the MDE, the overall contribution of seismic loads is less than 10%, and is not considered significant compared to uplift and hydrostatic forces, and therefore does not govern the performance of the structure.

Embankment Dam Portion

Stability analyses indicate somewhat lower than acceptable factors of safety for the upper shallow section of the steeper slope along the downstream face of the embankment under the design seismic conditions, assuming a horizontal and vertical acceleration of 0.1g. However, the main body of the downstream embankment is considered stable under earthquake loading conditions. The upstream section of the embankment has factors of safety of less than 1, indicating potentially unstable conditions. Since the main lower section of the embankment was originally constructed and compacted in the dry, a liquefaction type of failure is not anticipated. Remediation of the upstream section is recommended, and is discussed below.

Floods (DSG Section 6)

According to the DSG, dams shall be evaluated to safely pass an Inflow Design Flood (IDF), which is based on Consequence Category and the Probable Maximum Flood (PMF). The PMF is an estimate of the most severe "reasonably possible" flood at a particular location and time of year. For a High Consequence Category, the DSG requires an IDF with an Annual Exceedence Probability (AEP) of between 1/1000 and the PMF.

For this review, the Inflow Design Flood (IDF) used had an AEP of 1:1000 years. The IDF was determined through statistical analysis. The maximum daily flow rates for the nearby Sylvia Grinnel River was used in the analysis. The drainage area for the river is approximately 298,000 hectares. There are 29 years of flow measurements with 17 years that include the maximum daily discharge. To determine the IDF seventeen years of data were ranked in descending order. The Cunane formula was used to plot the data on lognormal probability paper. The 1:1000 year maximum daily discharge for the Sylvia Grinnel River was estimated with this method as 765 cubic meters per second. The flow was then transferred to the 385 hectare Lake Geraldine drainage area.

The results indicate that the estimated maximum daily inflow for Lake Geraldine is 14.1 (m^3/s).

To estimate the flow capacity of the existing spillway configuration, the weir formula is used to estimate the flow, as follows:

$Q = CLH^{3/2}$ (ft^3/s)

With $C = 3.9$ (weir coefficient)
$L = 49.2$ feet (15.7 metres)
$H = 3$ feet 1.5 inches (0.95 metres)

Therefore, $Q = 1056$ ft^3/s (30.0 m^3/s)

It should be noted that the difference between drainage areas in the analysis is large therefore some errors may occur. However the above calculations show that the spillway can handle significantly higher flows.

Based on our analysis, the spillway structure, in its current configuration, can safely pass the estimated Inflow Design Flood.

Discharge Facilities (DSG Section 7)

Section 7 of the DSG has a broad applicability that includes flow control equipment, instrumentation, and emergency backup equipment, which are relevant to more complex structures. In the case of the Lake Geraldine Dam, the applicability really only involves the spillway section.

According to the DSG, discharge facilities shall be capable of passing an Inflow Design Flood (IDF) without adversely affecting the freeboard. Freeboard is defined as the vertical distance between the water surface elevation and the lowest elevation of the top of the containment structure.

The freeboard should satisfy the requirements of section 7.2 of the DSG, Freeboard. That section indicates that sufficient freeboard be provided such that the percentage of overtopping waves during extreme flood or wind conditions is limited to an amount that would not lead to dam failure.

The Lake Geraldine Dam essentially has only one effective discharge facility, that being the spillway section. Normal water levels are at or slightly below the spillway discharge elevation, which yields a freeboard of approximately 0.95m (3 feet 1.5 inches).

Based on our analysis, the spillway is capable of passing an Inflow Design Flood. Wave action overtopping the gravity structure is not considered significant given the relatively small fetch of the lake.

Section 7 also requires consideration of the following for discharge facilities:

- Resistance to erosion
- Capability to pass floating debris

Erosion is not considered significant because of the rock foundation and concrete construction. Floating debris has historically not been a problem because of geographic location. The spillway dimensions and configuration are such that there are no obstructions to impede debris.

Geotechnical Considerations (DSG Section 8)

Section 8 of the DSG presents Geotechnical considerations for proposed dams, as well as for several configurations of existing dams.

Concrete Gravity Dam Portion

Not applicable, due to the dam being concrete on a competent rock foundation.

Embankment Dam Portion

The stability of the deepest section of the embankment section of the dam was analysed under various loading conditions. Static stability analyses, under both normal and flood level lake conditions on the upstream side of the dam, were carried out.

The Dam Safety Guidelines require a minimum factor of safety of 1.5 for steady state seepage conditions with a maximum storage pool. The stability analyses indicate a low factor of safety (~1.205) along the steep face of the dam and 1.34 for a deeper-seated failure.

With a maximum upstream water level, the dam has a factor of safety of 1.44 against a total failure, which would cause a major breach in the dam. The compacted granular fill and the presence of the concrete cut-off wall provides some stability, but their primary function is to maintain a low phreatic (water) level across the embankment, and the overall contribution of the cut-off wall to the embankment dam stability is not significant.

The upstream side of the embankment, with its steep slopes, appears to be potentially unstable. Some slumping of a section of the embankment was observed during the site visit. Factors of safety range from 1.025 to 1.168, which is considerably less than required.

Concrete Structures (DSG Section 9)

Section 9 of the DSG applies to concrete structures founded on strong, competent rock. Based on our review of the record data, and the dam performance over the last 43 years, it is believed that the foundations are indeed competent rock, and that invasive conformation and/or assessment is not warranted at this time.

Our assessment follows the format of Section 9 as follows:

Section 9.2 – Condition Of Structures And Site

The structure was visually inspected on site as described above. At the time of our visit, we did not observe any conditions that would adversely affect the structural adequacy and/or performance.

Observations that are considered minor at this time include:

- Several shrinkage cracks, actively seeping at a low rate.
- One joint leak estimated at 2-4 liters per minute, south of the spillway section.
- A few areas of spalling and delamination, due to embedded formwork.
- Corrosion of the 1985 spillway extension frame.

No general concrete deterioration problems were noted.

Section 9.3 – Loads

Loads used in our assessment of structural stability were in conformance with this sub-section, with the exception that Temperature (T) and backfill/silt deposit loads (S) were not considered applicable.

Temperature effects were not considered due to the relatively small size and length of the gravity dam.

Backfill/silt loads were not considered as there is no evidence of significant silt accumulation in the documentation.

A summary of loads considered is as follows:

- D Dead loads of permanent structures
- H, H_F Maximum normal and flood headwater levels, respectively
- U Internal (uplift) water pressure
- I Thrust created by an ice sheet
- Q Maximum design earthquake

Section 9.4 – Load Combinations

Load combinations used in our assessment of structural stability were in conformance with this sub-section, with the exception that the "Unusual Loading" case was not considered applicable. A summary of load combinations considered is as follows:

- Usual $D+H+I+U$
- Flood $D+H_F+U$
- Earthquake $D+H+Q+U_Q$

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Section 9.5 – Design And Analysis

A static and seismic analysis was performed on the dam using the above loads and combinations, and considering the following:

- Sliding
- Overturning
- Overstressing

Based on our analysis, and site inspection, the dam structure is deemed adequate to resist the above effects. The minimum factor of safety for overturning and sliding were calculated to be 1.33 and 1.72, respectively.

Typically, we would require minimum factors of safety for overturning and sliding of 1.5 and 2.0, respectively, for an existing structure.

Although the calculated values are slightly below our usual norms of acceptance, it is our opinion that the stability of the dam is adequate and would satisfy minimum factors of safety under more sophisticated analysis, for the following reasons:

1. The calculated factors are for the worst case load combination at the spillway section, which is the weakest link of the structure. The section was analyzed as if it was a stand-alone structure. In reality, the spillway section is relatively narrow in elevation, and directly engages the wing walls of the gravity section at each end. Given these boundary conditions, we believe the factor of safety would be similar to the 1.6 calculated for the gravity section.

2. The sliding calculation does not take into account the resistance offered by the rock anchors, which is significant, and would likely result in a factor of safety against sliding well in excess of 2.0.

Copies of our sliding and overturning calculations append the report.

Section 9.6 – Performance Indicators

The DSG recommends that the assessment of concrete dams include the following performance indicators:

- Position of resultant force
- Normal stresses at the heel and toe
- Sliding factors
- Observed conditions, based on records of permanent monitoring equipment such as joint meters, plumb lines, monument displacement, piezometric pressures, extensometers, and accelograms.

We have determined that the resultant force for the "Usual" load case is within the middle third of the section, as required. Normal stresses are also within acceptable limits.

Expressions to determine sliding factors are not considered applicable due to the rock anchor retrofit program of 1995. To our knowledge, no permanent monitoring instrumentation exists at or remote from the site.

Section 9.7 – Acceptance Criteria

This sub-section presents commonly accepted values for sliding factors. As mentioned above, the expressions for sliding factors are not considered applicable due to the rock anchor retrofit program of 1995. Concrete strength factors are within acceptable limits.

Reservoir & Environment (DSG Section 10)

According to the DSG, the following conditions should be assessed as they relate to the reservoir and environment:

a) The stability of slopes around the reservoir rim.
b) Detrimental affects of groundwater, reservoir water, soil, etc., on dam safety.
c) Silt deposition affecting discharge facilities or dam stability.
d) Hazards to local ecology.
e) Reservoir draw down capability.
f) Reservoir debris and ice should not present an unacceptable risk to dam safety.

The above items are discussed below:

The reservoir rim is a natural rock formation with mild to moderate gradient, and no history of instability.

No detrimental effects, such as aggressive chemical agents, are believed to exist in the groundwater or soils; there is certainly no evidence to warrant consideration.

As discussed above, there is no evidence of significant silt deposits behind the dam.

No significant ecological hazards are known to exist in the reservoir area.

Based on our review and inspection, the only significant items are e), reservoir draw down capability; and f), reservoir ice and debris as it relates to dam safety

Regarding item e), the reservoir does not have rapid draw down capability. Section 10.5 of the DSG indicates a requirement for rapid draw down for those dams subject to severe damage by earthquake, or where a high potential for internal erosion exists. In our opinion, these risk factors do not apply, and rapid draw down is not required.

Regarding item f), we have allowed for ice thrust in our stability analysis. We note that the ice thrust loads may be reduced by partial draw down of the reservoir before major ice loads are developed; this was discussed in the July 1990 Acres report. Debris has historically not been a problem, however, it should be cleared periodically from the upstream face to allow underwater inspections.

Construction (DSG Section 11)

This section applies to new construction and therefore is not applicable.

OPERATION & TESTING (DSG SECTION 2)

The applicable reference section of the DSG is Section 3: Operation, Maintenance, and Surveillance.

The interpretation of this section of the DSG requires clarification on the meaning and intent of the words "operation" and "testing". In this section of the DSG, "testing" generally refers to the testing of equipment required to operate discharge facilities. In the case of the Lake Geraldine Dam, the primary discharge facility is the spillway section. No actual equipment exists. Therefore, there is no testing requirement for this structure.

The word "operation" in the DSG is associated with the premise that "the operation of a dam shall not violate any important design assumptions that could impair the safety of the dam." Review of Section 3.2 of the DSG indicates that this section applies to more complex dams with operable flow control equipment, ice rakes, trash racks, penstocks, etc. Simply stated, the Lake Geraldine Dam is not really "operable".

Notwithstanding these limitations, there should be some basic operational procedures for ice management and cleaning of upstream debris that would form part of the OMS Manual (see below).

Other applicable requirements of Section 3 are described below.

In the DSR checklist of required items, the DSG indicate that a Permanent Record File (PRF) suitable for transfer to the regulatory agency be maintained as an ongoing historical reference. The file should contain the following:

- OMS Manual (see below)
- Permanent Log Book (see below)
- History and photographic record
- As-Built Drawings
- Performance reports
- All design data
- Records of all inspections and DSR's

Based on our review and correspondence, a PRF does not exist, however, most of the raw data is readily available.

The DSG indicate that a dam Operation, Maintenance, and Surveillance (OMS) Manual shall be provided for every dam structure. The manual may be quite involved depending on the complexity of the dam. For the Lake Geraldine Dam, an acceptable manual would likely be relatively simple and concise. The manual should contain information and procedures that include the following:

- General description, history, location, access, etc.
- Chain of operational responsibilities
- Requirements for training of involved staff
- Responsibility and mechanism for review and update, including DSR input
- Requirements for operation, maintenance and surveillance as per Sections 3.2, 3.3, and 3.4 of the DSG (See below)

Based on our review and correspondence, an OMS Manual does not exist.

The DSG indicate that a Permanent Logbook shall be provided for every dam structure. The logbook should contain notations or records of the following:

- Changes to normal operation
- Unusual events or conditions
- Inspection activity
- Weather conditions and trends
- Unusual maintenance activities
- Tests of any control equipment

Based on our review and correspondence, a Permanent Logbook does not exist.

MAINTENANCE (DSG SECTION 2)

The applicable reference section of the DSG is Section 3: Operation, Maintenance, and Surveillance.

Maintenance Procedures (MPS) as described in Section 3.3 of the DSG are intended to ensure that the structures are maintained in a safe and serviceable condition.

No formal Maintenance Procedures exist. These should form part of the OMS Manual.

SURVEILLANCE & MONITORING (DSG SECTION 2)

The applicable reference section of the DSG is Section 3: Operation, Maintenance, and Surveillance.

Surveillance Procedures (SPS) as described in Section 3.4 of the DSG are intended to ensure adequate inspection and monitoring. Applicable considerations are as follows:

a) Procedures or requirements for routine visual inspection by staff, including inspection records.
b) Procedures for implementation of any required action as a result of a routine inspection.
c) Procedures or requirements for more detailed regular inspections, such as underwater assessments.
d) Procedures or requirements for special inspections due to extreme events or unusual observations.

No formal Surveillance Procedures exist. These should form part of the OMS Manual.

Emergency Preparedness (DSG Section 2)

Section 4 of the DSG involves Emergency Preparedness. The primary requirement is that an Emergency Preparedness Plan (EPP) exists. An EPP should describe the actions to be taken by the owner and operator in the event of an emergency. The EPP should include the following:

- Emergency identification and evaluation
- Preventative action
- Notification procedure and flowchart
- Response during darkness, adverse weather, etc.
- Available resources and their allocation
- Inundation maps, based on an Inundation Study

Based on our review and correspondence, no formal Emergency Preparedness Plan exists.

Compliance With Previous Reviews (DSG Section 2)

No previous Dam Safety Review documents exist at this time.

5. Summary

Based on our inspection, review, and analyses, we summarize the results of the DSR as follows:

1. In accordance with Section 1 of the DSG, the dam has been classified as having a High Consequence Category.

2. The concrete gravity section of the dam is in a safe and serviceable condition at this time, with no significant changes in visible condition compared to the last (1997) inspection. These sections of the dam are in general compliance with the required design and performance standards of the DSG, Sections 5 through 11, where applicable and appropriate for the structure, as discussed above. Safety improvements are therefore not recommended at this time.

3. The embankment section of the dam is in a serviceable condition at this time, however, concerns exist with the future stability of the steeper sloped areas, particularly on the upstream face. This section of the dam is in general compliance with the required design and performance standards of the DSG, Sections 5 through 11, with the exception of Sections 5 and 8, which involve stability considerations. Safety improvements are recommended below.

4. The dam is in non-compliance with the requirements of Sections 3 and 4 of the DSG. The following documents do not exist at this time:

   - Permanent File
   - Operation, Maintenance & Surveillance Manual
   - Logbook
   - Emergency Preparedness Plan

5. Based on the available record documentation, the submerged portion of the dam has not had assessment **and** reporting of any kind completed since 1990. Although a diving video was reportedly prepared in 1999, there does not appear to be a **thorough** assessment and report prepared over the entire documentation period (45 years).

6. Recommendations & Required Action

1. The structures should have a Dam Safety Inspection (DSI) conducted in 2002, preferably by mid-October of that year. This is essentially a yearly non-invasive review comprising a visual inspection to identify any changes in condition, or any observed concerns. The summary written report generated would form a permanent record document to be included in the Permanent Record File.

2. An underwater inspection, assessment, and detailed reporting of the submerged structures should be considered in 2002. This inspection should be coordinated with, and be under the direction of, the DSI recommended in Item 1, above. Underwater inspections should be carried out with at least the same frequency as Dam Safety Reviews, i.e. every seven years.

3. The embankment portion of the dam does not meet the requirements of Sections 5 and 8 of the DSG. Specifically, factors of safety for dynamic and static stability are significantly lower than required minimums, particularly on the steeper upstream face of the embankment. At this time, it is recommended that provisions be made for the design of repairs, which should be implemented in 2002. Repairs would likely include flattening of upstream slopes.

4. The dam is in non-compliance with the requirements of Sections 3 and 4 of the DSG. The following documents need to be developed and maintained.

   - Permanent File
   - Operation, Maintenance & Surveillance Manual
   - Logbook
   - Emergency Preparedness Plan

5. Conduct a Dam Safety Review by the year 2009.

In terms of time to compliance, it is our opinion that an understanding to proceed be demonstrated as soon as practically possible so as to show intent to comply with the DSG.

Trow

We would be pleased to discuss this report with you at your convenience.

Yours truly,

Trow Consulting Engineers Ltd.

Prepared By:

REGISTERED PROFESSIONAL ENGINEER
A. D. MURRAY
LICENSEE
N.W.T.

LICENSED PROFESSIONAL ENGINEER
A. D. MURRAY
PROVINCE OF ONTARIO

Allan Murray, P.Eng.,
Manager,
Special Projects Group

Prepared By:

FOR/ Andy Schell, P.Eng.,
Senior Geotechnical Engineer

APPENDIX 1
SITE PHOTOGRAPHS
LAKE GERALDINE DAM

Photograph No. 1 – Lake Geraldine Dam overview, upstream side, looking northwest.

Photograph No. 2 – Top of Gravity Dam, looking northwest.

Photograph No. 3 – Spillway section, looking northwest.

Photograph No. 4 – Lake Geraldine Reservoir, looking northwest to embankment portion of dam. Spillway section is at extreme left.

Photograph No. 5 – Close-up of embankment portion at tie-in with Gravity section (at left), upstream face.

Photograph No. 6 – Close up of embankment portion at north termination, upstream face.

Photograph No. 7 – Overview of north end of Gravity elevation and Spillway elevation. Note leaching cracks on Gravity face.

Photograph No. 8 – Overview of southern portion of Spillway section, and south Gravity elevation. Note leaching cracks in Gravity face.

Photograph No. 9 – Overview of south Gravity elevation. Note leaching cracks.

Photograph No. 10 – Southern terminus of Gravity section.

APPENDIX 2
SITE PHOTOGRAPHS
LAKE GERALDINE DAM
DAMAGE PATH

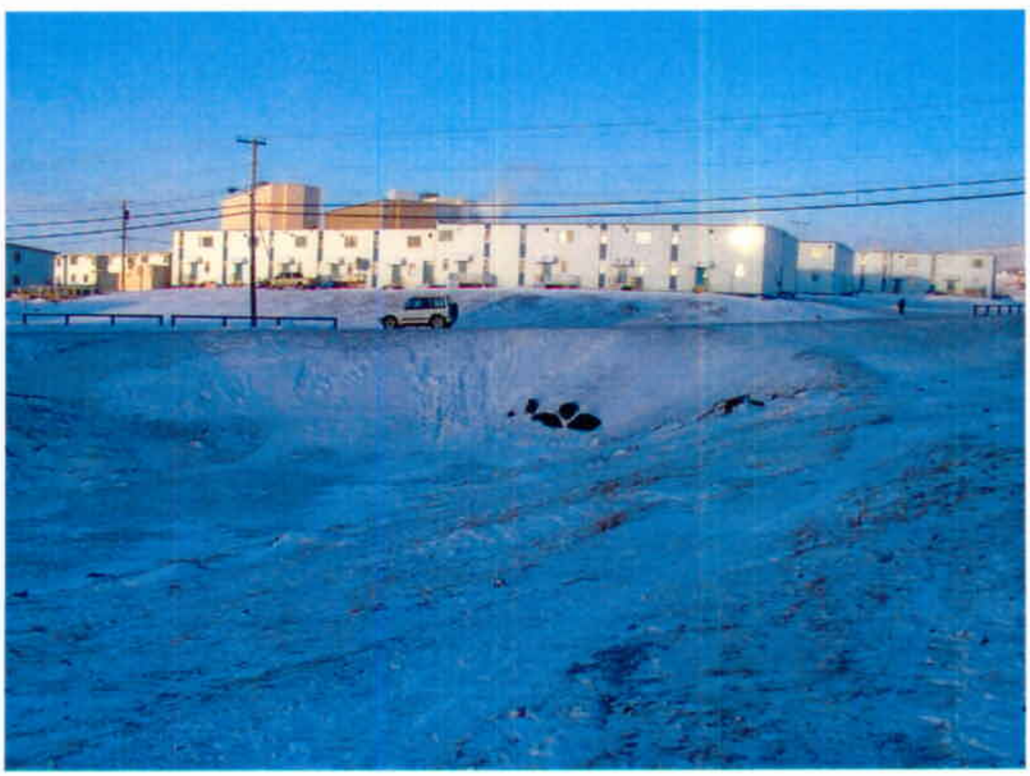
Photograph No. 1 – Looking upstream at Basin, southwest of Dam (in background).

Photograph No. 2 – Looking upstream at Power Station Road Crossing. This crossing would be the first washed out (#1) in the event of a dam breach.

Photograph No. 3 – Looking upstream at Apex Road Crossing, Power Station in background. A second washout (#2) would be expected here.

Photograph No. 4 – Moving further downstream to Old Riverbed, looking north, just west of UIVVAQ Road.

Photograph No. 5 – Further downstream, looking north at bridge to Frobisher Inn. This bridge would likely be lost in the event of a dam breach.



Photograph No. 6 – Looking upstream at UIVVAQ Road Crossing, which would be road washout #3.

Photograph No. 7 – Further downstream, looking northeast along Nanvq Street. Housing is in the damage path.

Photograph No. 8 – Moving further downstream to Ikaluit Street Crossing, which would be washout #4.

Photograph No. 9 – Moving further downstream to Sinnz Street Crossing, which would be washout #5. All buildings are in damage path.

Photograph No. 10 – Terminus of Breach Flow at Frobisher Bay Beach.

APPENDIX 3
SUMMARY STABILITY CALCULATIONS
LAKE GERALDINE DAM

IQALUIT WATER RESERVOIR - DAM

FORCES and DATA

$\gamma c := 24000 \frac{N}{m^3}$ $\gamma w := 9810 \frac{N}{m^3}$ $B := 6.5\ m$ $H := 8.8\ m$ $Hw := 7.6\ m$ $KN := 1000\ N$

$Hwf := 8.5\ m$

Case1 Usual Loading (D+H+I+S+U)

Resisting Forces

$GDam := 30.62\ m^2\ 1\ m \cdot \gamma c$ $GDam = 734.88 \cdot KN$ $MGDam := GDam \cdot 4.35 \cdot m$

$MGDam = 3.197 \cdot 10^3\ m \cdot KN$

$Anchor := 255\ KN$ $MAnchor := Anchor \cdot 6.1\ m$

$MAnchor = 1.556 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWat := ((\gamma w \cdot Hw)\ Hw) \cdot 1\ m \cdot 0.5$ $HWat = 283.313 \cdot KN$ $MHWat := HWat \cdot \left(\frac{Hw}{3} + 0.3\ m\right)$

$MHWat = 802.72\ m \cdot KN$

$UWat := \gamma w \cdot (Hw) \cdot B \cdot 1\ m \cdot 0.5$ $UWat = 2.423 \cdot 10^5 \cdot N$ $MUWat := UWat \cdot B \cdot 0.666$

$MUWat = 1.049 \cdot 10^3\ m \cdot KN$

$Ice := 150\ KN$ $MIce := Ice \cdot (Hw + 0.3\ m)$

$MIce = 1.185 \cdot 10^3\ m \cdot KN$

$$e1 := \frac{(MGDam + MAnchor - MHWat - MUWat - MIce)}{GDam + Anchor - UWat}$$

$e1 = 2.295\ m$ $\frac{B}{3} = 2.167\ m$ $B \cdot 0.66 = 4.29\ m$

Case2 Flood Working Condition (D+H+S+U)

Resisting Forces

$GDamf := 30.62\ m^2\ 1\ m \cdot \gamma c$ $\quad GDamf = 734.88 \cdot KN$ $\quad MGDamf := GDam \cdot 4.35 \cdot m$

$MGDamf = 3.197 \cdot 10^3\ m \cdot KN$

$Anchorf := 255\ KN$ $\quad MAnchorf := Anchorf \cdot 6.1\ m$

$MAnchorf = 1.556 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWatf := ((\gamma w \cdot Hwf)\ Hwf) \cdot 1\ m \cdot 0.5$ $\quad HWatf = 354.386 \cdot KN$ $\quad MHWatf := HWatf \cdot \left(\frac{Hwf}{3} + 0.3\ m\right)$

$MHWatf = 1.11 \cdot 10^3\ m \cdot KN$

$UWatf := \gamma w \cdot (Hwf) \cdot 1\ m \cdot B \cdot 0.5$ $\quad UWatf = 2.71 \cdot 10^5 \cdot N$ $\quad MUWatf := UWatf \cdot B \cdot 0.666$

$MUWatf = 1.173 \cdot 10^3\ m \cdot KN$

Case3 Earthquake Loading (D+H+S+Q+U)

Resisting Forces

$GDamq := 30.62\ m^2\ 1\ m \cdot \gamma c$ $\quad GDamq = 734.88 \cdot KN$ $\quad MGDamq := GDam \cdot 4.35 \cdot m$

$MGDamq = 3.197 \cdot 10^3\ m \cdot KN$

$Anchorq := 255\ KN$ $\quad MAnchorq := Anchorq \cdot 6.1\ m$

$MAnchorq = 1.556 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWatq := ((\gamma w \cdot Hw)\ Hw) \cdot 1\ m \cdot 0.5$ $HWatq = 283.313 \cdot KN$ $MHWatq := HWatq \cdot \left(\frac{Hw}{3} + 0.3\ m\right)$

$MHWatq = 802.72\ m \cdot KN$

$UWatq := \gamma w \cdot (Hw) \cdot B \cdot 1\ m \cdot 0.5$ $UWatq = 2.423 \cdot 10^5 \cdot N$ $MUWatq := UWatq \cdot B \cdot 0.666$

$MUWatq = 1.049 \cdot 10^3\ m \cdot KN$

Seismic Forces

Iqaluit

$Za := 1$ $Zv := 0$ $v := 0.05$

$V := Ve \cdot \frac{U}{R}$

$T1 := 0.09 \cdot \frac{H}{\sqrt{B \cdot 1\ m}}$ $T1 = 0.311$

Za<Zv for T1=0.311s => $S := 4.2$ $I := 1.5$ for dams $F := 1$ $U := 0.6$ $R := 3.5$

$W := GDamq$ $W = 7.349 \cdot 10^5\ N$

$Ve := v \cdot S \cdot I \cdot F \cdot W$ $Ve = 231.487 \cdot KN$

$V := Ve \cdot \frac{U}{R}$

$V = 39.684 \cdot KN$

Full Reservoir

$Pe1 := 0.1 \cdot (\gamma w \cdot Hw)$ $Pe1 = 7.456 \cdot 10^3\ Pa$ $E1 := Pe1 \cdot \frac{Hw \cdot 1\ m}{2}$ $E1 = 28.331 \cdot KN$

$Q := V + E1$ $Q = 68.015 \cdot KN$ $MQ := (Q) \cdot (Hw \cdot 0.33 + 0.3\ m)$

$MQ = 190.986\ m \cdot KN$

Case1 Usual Loading (D+H+I+S+U)

1) Overturning

$$FOSoverturning := \frac{(MGDam + MAnchor)}{(MHWat + MUWat + MIce)}$$

$FOSoverturning = 1.565$

2) Sliding

$$FOSoverturning := \frac{(GDam + 220\ KN - UWat) \cdot \tan(50\ deg)}{(HWat + Ice)}$$

$FOSoverturning = 1.96$

Case2 Flood Working Condition (D+H+S+U)

1) Overturning

$$FOSoverturning := \frac{(MGDamf + MAnchorf)}{(MHWatf + MUWatf)}$$

$FOSoverturning = 2.081$

2) Sliding

$$FOSsliding := \frac{(GDamf + 220\ KN - UWat) \cdot \tan(50\ deg)}{(HWatf)}$$

$FOSsliding = 2.396$

Case3 Earthquake Loading (D+H+S+Q+U)

1) Overturning

$$FOSoverturning := \frac{(MGDamq + MAnchorq)}{(MHWatq + MUWatq + MQ)}$$

$$FOSoverturning = 2.326$$

2) Sliding

$$FOSsliding := \frac{(GDamq + 220\ KN - UWatq) \cdot \tan(50\ deg)}{HWatq + Q}$$

$$FOSsliding = 2.417$$

IQALUIT WATER RESERVOIR - SPILLWAY

FORCES and DATA

$\gamma c := 24000 \dfrac{N}{m^3}$ $\gamma w := 9810 \dfrac{N}{m^3}$ $B := 7\ m$ $H := 8.5\ m$ $Hw := 8.2\ m$ $KN := 1000\ N$

$Hwf := 9.1\ m$

Case1 Usual Loading (D+H+I+S+U)

Resisting Forces

$GDam := 28.14\ m^2\ 1\ m \cdot \gamma c$ $GDam = 675.36 \cdot KN$ $MGDam := GDam \cdot 4.12 \cdot m$

$MGDam = 2.782 \cdot 10^3\ m \cdot KN$

$GWat := 7.87\ m^2 \cdot (1\ m) \cdot \gamma w$ $GWat = 77.205 \cdot KN$ $MGWat := GWat \cdot 6.15 \cdot m$

$MGWat = 474.809\ m \cdot KN$

$Anchor := 255\ KN$ $MAnchor := Anchor \cdot 5.9\ m$

$MAnchor = 1.504 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWat := ((\gamma w \cdot Hw)\ Hw) \cdot 1\ m \cdot 0.5$ $HWat = 329.812 \cdot KN$ $MHWat := HWat \cdot \left(\dfrac{Hw}{3} + 0.3\ m\right)$

$MHWat = 1 \cdot 10^3\ m \cdot KN$

$UWat := \gamma w \cdot (Hw) \cdot B \cdot 1\ m \cdot 0.5$ $UWat = 281.547 \cdot KN$ $MUWat := UWat \cdot B \cdot 0.666$

$MUWat = 1.313 \cdot 10^3\ m \cdot KN$

$Ice := 150\ KN$ $MIce := Ice \cdot (Hw + 0.3\ m)$

$MIce = 1.275 \cdot 10^3\ m \cdot KN$

$$e1 := \frac{(MGDam + MGWat + MAnchor - MHWat - MUWat - MIce)}{GDam + GWat + Anchor - UWat}$$

$e1 = 1.617\ m$ $\dfrac{B}{3} = 2.333\ m$ $B \cdot 0.66 = 4.62\ m$

Case2 Flood Working Condition (D+H+S+U)

Resisting Forces

$GDamf := 28.14\ m^2\ 1\ m \cdot \gamma c$ $\quad GDamf = 675.36 \cdot KN$ $\quad MGDamf := GDam \cdot 4.12 \cdot m$

$MGDamf = 2.782 \cdot 10^3\ m \cdot KN$

$GWatf := 9.5\ m^2 \cdot (1\ m) \cdot \gamma w$ $\quad GWatf = 93.195 \cdot KN$ $\quad MGWatf := GWatf \cdot 6.14 \cdot m$

$MGWatf = 572.217\ m \cdot KN$

$Anchorf := 255\ KN$ $\quad MAnchorf := Anchorf \cdot 5.9\ m$

$MAnchorf = 1.504 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWatf := ((\gamma w \cdot Hwf)\ Hwf) \cdot 1\ m \cdot 0.5$ $\quad HWatf = 406.183 \cdot KN$ $\quad MHWatf := HWatf \cdot \left(\frac{Hwf}{3} + 0.3\ m\right)$

$MHWatf = 1.354 \cdot 10^3\ m \cdot KN$

$UWatf := \gamma w \cdot (Hwf) \cdot B \cdot 1\ m \cdot 0.5$ $\quad UWatf = 312.449 \cdot KN$ $\quad MUWatf := UWatf \cdot B \cdot 0.666$

$MUWatf = 1.457 \cdot 10^3\ m \cdot KN$

Case3 Earthquake Loading (D+H+S+Q+U)

Resisting Forces

$GDamq := 28.14\ m^2\ 1\ m \cdot \gamma c$ $\quad GDamq = 675.36 \cdot KN$ $\quad MGDamq := GDam \cdot 4.12 \cdot m$

$MGDamq = 2.782 \cdot 10^3\ m \cdot KN$

$GWatq := 7.87\ m^2 \cdot (1\ m) \cdot \gamma w$ $\quad GWatq = 77.205 \cdot KN$ $\quad MGWatq := GWatq \cdot 6.15 \cdot m$

$MGWatq = 474.809\ m \cdot KN$

$Anchorq := 255\ KN$ $\quad MAnchorq := Anchorq \cdot 5.9\ m$

$MAnchorq = 1.504 \cdot 10^3\ m \cdot KN$

Overturning Forces

$HWatq := ((\gamma w \cdot Hw) \, Hw) \cdot 1 \; m \cdot 0.5$ $\quad HWatq = 329.812 \cdot KN$ $\quad MHWatq := HWatq \cdot \left(\frac{Hw}{3} + 0.3 \; m\right)$

$MHWatq = 1 \cdot 10^3 \; m \cdot KN$

$UWatq := \gamma w \cdot (Hw) \cdot B \cdot 1 \; m \cdot 0.5$ $\quad UWatq = 2.815 \cdot 10^5 \cdot N$ $\quad MUWatq := UWatq \cdot B \cdot 0.666$

$MUWatq = 1.313 \cdot 10^3 \; m \cdot KN$

Seismic Forces

Iqaluit

$Za := 1$ $\quad Zv := 0$ $\quad v := 0.05$

$V := Ve \cdot \frac{U}{R}$

$T1 := 0.09 \cdot \frac{H}{\sqrt{B \cdot 1 \; m}}$ $\quad T1 = 0.289$

$Za > Zv$ for T=0.289s => $S := 4.2$ $\quad I := 1.5$ for dams $\quad F := 1$ $\quad U := 0.6$

$R := 3.5$

$W := GDamq + GWatq$ $\quad W = 7.526 \cdot 10^5 \; N$

$Ve := v \cdot S \cdot I \cdot F \cdot W$ $\quad Ve = 237.058 \cdot KN$

$V := Ve \cdot \frac{U}{R}$

$V = 40.638 \cdot KN$

Full Reservoir

$Pe1 := 0.1 \cdot (\gamma w \cdot Hw)$ $\quad Pe1 = 8.044 \cdot 10^3 \; Pa$ $\quad E1 := Pe1 \cdot \frac{Hw \cdot 1 \; m}{2}$ $\quad E1 = 32.981 \cdot KN$

$Q := V + E1$ $\quad Q = 73.62 \cdot KN$ $\quad MQ := (Q) \cdot (Hw \cdot 0.33 + 0.3 \; m)$

$MQ = 221.301 \; m \cdot KN$

Case1 Usual Loading (D+H+I+S+U)

1) Overturning

$$\text{FOSoverturning} := \frac{(\text{MGDam} + \text{MGWat} + \text{MAnchor})}{(\text{MHWat} + \text{MUWat} + \text{MIce})}$$

$$\text{FOSoverturning} = 1.327$$

2) Sliding

$$\text{FOSsliding} := \frac{(\text{GDam} + \text{GWat} + 220\ \text{KN} - \text{UWat})\cdot\tan(50\ \text{deg})}{(\text{HWat} + \text{Ice})}$$

$$\text{FOSsliding} = 1.716$$

Case2 Flood Working Condition (D+H+S+U)

1) Overturning

$$\text{FOSoverturning} := \frac{(\text{MGDamf} + \text{MGWatf} + \text{MAnchorf})}{(\text{MHWatf} + \text{MUWatf})}$$

$$\text{FOSoverturning} = 1.729$$

2) Sliding

$$\text{FOSsliding} := \frac{(\text{GDamf} + \text{GWatf} + 220\ \text{KN} - \text{UWat})\cdot\tan(50\ \text{deg})}{(\text{HWatf})}$$

$$\text{FOSsliding} = 2.074$$

Case3 Earthquake Loading (D+H+S+Q+U)

1) Overturning

$$FOSoverturning := \frac{(MGDamq + MGWatq + MAnchorq)}{(MHWatq + MUWatq + MQ)}$$

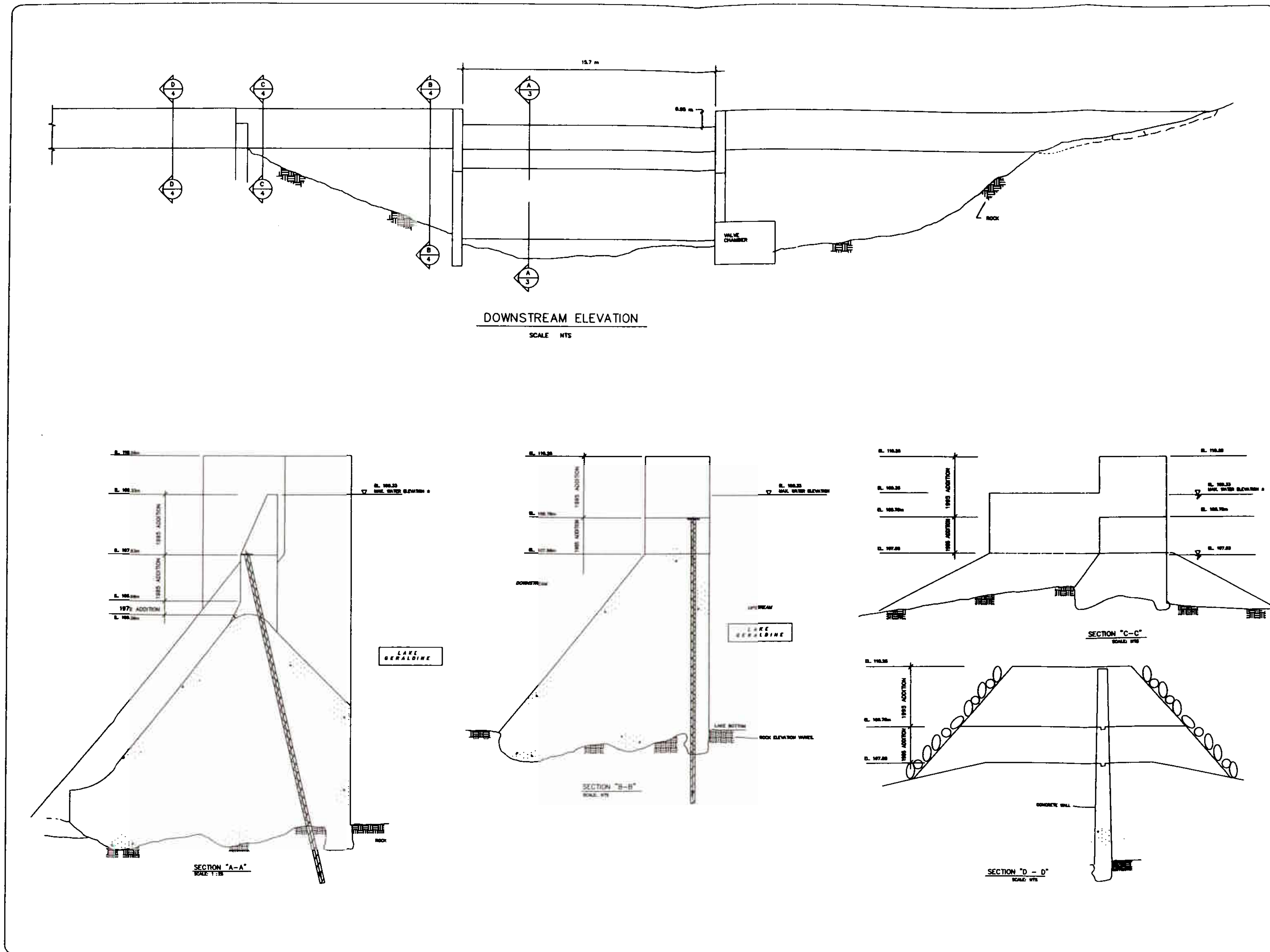
$$FOSoverturning = 1.879$$

2) Sliding

$$FOSsliding := \frac{(GDamq + GWatq + 220000\ N - UWatq) \cdot \tan(50\ deg)}{HWatq + Q}$$

$$FOSsliding = 2.041$$

APPENDIX 4
GENERAL ARRANGEMENT DRAWING
LAKE GERALDINE DAM



Trow
Trow Consulting Engineers Ltd.
15 Cuddy Boulevard
London, Ontario, N5V 3Y3

NO.	REVISION	DATE

Client:
CITY OF IQALUIT

Project Name:
LAKE GERALDINE DAM

Project Address:
IQALUIT, NUNAVUT

Sheet Title:
GENERAL ARRANGEMENT

Design:
Drawn: R.M.
Approv.:
Scale: NOT TO SCALE
Date: October 2001

Project No.
MU15152

Dwg. No.
1
OF 1