ATTACHMENT 9

Lake Geraldine Dam Breach Analysis

Rev 1
April 24, 2026

LAKE GERALDINE DAM
DAM-BREACH ANALYSIS & INUNDATION MAPPING STUDY

Submitted to:

City of Iqaluit
Iqaluit, Nunavut

Submitted by:

AMEC Environment & Infrastructure
Edmonton, Alberta

May 2011

EW1033

L:\PROJECT\EW\1033 Lake Geraldine EPP\Report\Inundation Report Final 9May2012.docx

TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS (*cont*)

PAGE

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 Purpose and Objectives

This study was conducted by AMEC Environment & Infrastructure (AMEC) for the City of Iqaluit (the City). This study was undertaken in response to a recent Dam Safety Inspection[1] which recommended that an Emergency Preparedness Plan (EPP) be prepared for the Lake Geraldine Dam. This report presents the analyses and results of a Dam-Breach and Inundation Mapping Study, and constitutes the basis for preparation of the Lake Geraldine Dam EPP. The Lake Geraldine Dam EPP is a separate report completed by AMEC in May 2012.

The key objectives of the work were to:

- Develop appropriate hypothetical dam-breach scenarios and select the most critical;
- Determine the resulting dam-breach flood wave (outflow hydrograph);
- Route the dam-breach flood wave downstream;
- Map the extents of flood inundation; and
- Evaluate the nature and extent of impacts due to passage of the flood wave.

The study provides a context for understanding the consequence of a dam failure and was conducted in accordance with the Canadian Dam Association *Dam Safety Guidelines*[2].

1.2 Lake Geraldine Dam

Figure 1 provides a plan view of the study location area. Lake Geraldine Dam, located at the outlet of Lake Geraldine, impounds runoff from a 3.5 km^2 catchment area to provide domestic and municipal water for the City of Iqaluit. The dam is very close to the City. The dam is a composite structure with both concrete gravity and rockfill berm sections – all founded on bedrock. Raw water is drawn from the reservoir through a 15” diameter pipe and directed to the nearby water treatment plant located less than 400 m from the dam. When reservoir capacity is exceeded the excess water passes over the spillway. Spilled flows are conveyed via an unnamed drainage channel through the City and then discharge into Koojesse Inlet. The spillway was active for most of the summer of 2011[3].

The original dam was constructed in 1958 with a design crest elevation of 107.88 m and a spillway crest of 106.68 m. The dam has been raised incrementally in 1985, 1995, and 2006, to its present design elevation of 112.28 m and spillway crest elevation of 111.33 m. Appendix A provides a copy of selected dam design drawings completed for the 2006 dam improvements.

Figure 2 provides the relationship between stage and storage for Lake Geraldine and Table 1 provides a summary of storage volumes at key elevations.

---

[1] Concentric Associates International Incorporated. 2010. Lake Geraldine Dam Iqaluit, Nunavut Dam
[2] Canadian Dam Association. 2007. Dam Safety Guidelines, 2007.
[3] Personal communication – Paul Clow, City of Iqaluit (April 2012).

Table 1. Lake Geraldine Storage at Selected Lake Level Elevations

Lake Level Elevation (m)	Storage (1000 m^3)	Remark
101.30	55	Bedrock at Base of Concrete Dam and Spillway
106.50	611	Bedrock at North and Central Berm
106.68	644	1985 Spillway Elevation
107.60	839	Bedrock at South Berm
108.00	930	Central Berm Base Elevation
109.33	1280	1995 Spillway Elevation
111.33	1963	2006 (existing) Spillway Elevation
111.40	1993	Sunny Day Concrete Dam Failure
112.28	2349	North and South Concrete Dam Crest Elevation
112.50	2442	North, Central, and South Berm Crest Elevation
112.58	2479	Wet Weather Failure - Concrete Dam Failure

2.0 METHODOLOGY

2.1 Overview

Investigating the effect of a hypothetical dam breach involves two steps: (1) estimating the breach hydrograph, that is the discharge with respect to time exiting the reservoir through the breach; and (2) routing this breach outflow hydrograph downstream to define its change in depth, velocity and peak discharge with distance along the flow path.

The Canadian Dam Association 2007 Dam Safety Guidelines (CDA Guidelines) have been adopted for specific direction as they represent best practices. These guidelines recommend determination of the ultimate discharge from a hypothetical breach of the dam. The breach results in a large flood wave (breach hydrograph) immediately downstream of the breaching location. Flood inflow and reservoir conditions and breach development govern the magnitude of the breach hydrograph.

2.2 Failure Scenarios

2.2.1 Modes of Failure

The CDA Guidelines recommend consideration of the following two failure modes: “flood-induced” failure and “sunny-day” failure. A flood-induced failure is an overtopping dam failure resulting from a flood of a magnitude that is greater than what the dam can safely pass. A sunny-day failure is dam failure that occurs independent of flood conditions and is caused by other factors such as internal erosion, piping, earthquakes, slope failure, or structural failure.

The two failure modes were considered in identifying failure scenarios for each of the two structural components, i.e. the concrete component and the rock fill berm component, as each may fail separately:

Concrete Dam and Spillway Failure:

- sunny-day sudden failure of the concrete dam and spillway; and
- flood-induced failure of the concrete dam and spillway.

Rock Fill Berm Failure:

- sunny-day sudden failure of rockfill berms; and
- flood-induced slope failure of the rockfill berms.

Both the concrete and the berm components of the dam consist of a number of segments, as identified on the drawings in Appendix A. The several segments and their basic geometries are summarized in Table 2.

By inspection of the geometries and the path of the flood wave for each structural segment, it was determined that the most critical berm segment was the central berm and the most critical concrete segment was the spillway segment. The specific failure scenarios adopted are discussed below in subsequent sections. The general direction of the breach path is illustrated on Figure 1.

Table 2. Berm and Dam Segment Geometry

Berm/Dam Segment	Length[1] (m)	Crest Elevation[1] (m)	Base Elevation[2,3] (m)	Bedrock Elevation[4] (m)	Height of Berm/Dam (m)
North Berm	55.5	112.5	108.3	106.5	4.3
Central Berm	78.0	112.5	108.0	106.5	4.5
North Dam	13.3	112.3	102.6	101.3	11.0
Spillway	15.3	111.3	101.6	101.3	10.0
South Dam	39.1	112.3	102.6	101.3	11.0
South Berm	68.5	112.5	111.5	107.6	1.0

Notes:

1. Lengths and crest elevations based on Trow Associates Inc. (Trow) Earth and Concrete Work drawings (2006).
2. Base elevation for North Dam, Spillway, and South Dam based on the bathymetric depth contours surveyed by NRC, 2008.
3. Base elevation for North, Central, and South Berms based on Trow's Earth and Concrete Work drawings (2006).
4. Bedrock elevation based on both Trow's Earth and Concrete Work drawings (2006) and Trow's Borehole Logs (2004).

2.2.2 Concrete Dam and Spillway Failure

There is very little prescriptive guidance on concrete dam failure modes for dam breach studies. Based on the limited information found, the most plausible extreme failure scenario for the Lake Geraldine concrete gravity dam is failure of one or more vertical sections, along weak points in the structure (for example along an expansion joint). A complete near-instantaneous failure of the entire concrete dam structure may be a conceivable extreme failure mode for an arch dam structure; however, this was not considered to be a reasonable scenario for this case involving a concrete gravity dam section.

The breach location and geometry for the concrete dam and spillway failure was taken as the combined failure of the full 15.3 m section of spillway combined with a 9.0 m section of the south dam (the portion of the south dam extending from the spillway up to the first expansion joint). This failure geometry is consistent with the description of a plausible concrete gravity dam breach described in the CDA Guidelines: *"Concrete gravity dams tend to have partial breach, as one or more monolith sections formed during dam construction are forced apart".*

The spillway was selected as critical for failure as it is centered across the lowest point in the upstream reservoir, which would result in the most volume being released from the reservoir. In addition, the hydrostatic pressure would be the greatest at the base of the spillway, since it is at the lowest elevation along the dam cross-section. A portion of the South Dam up to the first expansion joint was also selected for failure in conjunction with the spillway. That length of dam was selected since the expansion joint extends from the crest to the base of the dam. The base of this section of the South Dam is also at an elevation comparable to the base of the spillway and would be subject to highest hydrostatic pressures. The concrete dam and spillway failure section is illustrated on the drawings in Appendix A.

2.2.3 Central Berm Failure

In addressing the potential failure of the berms, the Central Berm was selected since its base elevation is the lowest of all three berms, at El. 108.0 m (see Table 1). This results in the largest embankment height and largest potential magnitude for flood release corresponding to a berm failure. The berm could fail due to overtopping or piping – both of these scenarios were examined.

2.3 Adopted Models

2.3.1 Breach Hydrograph Model - FLDWAV

The FLDWAV model, Version 2.0, was used to simulate the development of a breach and the resulting breach outflow hydrographs, which define the rate of flow exiting the reservoir during the hypothetical failure scenario. FLDWAV was developed by the U.S. National Weather Service (Fread, D.L., 1998) and it replaces their earlier DAMBRK and DWOPER models.

2.3.2 Flood Routing – River2D

Typically, dam breach inundation studies employ an unsteady one-dimensional (1D) flow model. FLDWAV, while it does have the capability of routing outflow hydrograph downstream, is limited to very simple, one-dimensional channel geometries, and was not considered suitable for the current analysis. The US Army Corps of Engineers' HEC-RAS model is often used for 1D flood routing analysis. However, preliminary trials using the HEC-RAS model determined that flood wave flows were not adequately simulated in the downstream reaches. Therefore, a 2D model - the University of Alberta's River2D model - was adopted for the flood routing analysis. Model results were then post-processed for presentation using standard GIS-based tools (e.g. ArcGIS).

The routing model was constructed by extracting topographic information from available topography data provided by the City. Figure 3 presents the River2D model domain and finite element mesh used for computation. The mesh was constructed of 14,394 elements. A finer computational mesh was used through the approach channel and the area around the flow split (7.5 m spacing). A slightly coarser mesh (20 m spacing) was used throughout the lower flat areas downstream of the split.

The initial condition for the model was an initial flow rate of 25 m^3/s, a downstream fixed water level of El. 5.25 m at Koojesse Inlet, and a dry bed everywhere else. A constant bed roughness, k_{bed} = 0.50 m was applied to the entire model domain. This roughness is comparable to a Mannings bed roughness of 0.060 for shallow depths and 0.035 for depths greater than approximately 2 m.

2.4 Hydrologic Conditions for Failure Scenarios

2.4.1 Sunny-Day Failure

The sunny-day failure was assumed to occur during non-flood conditions, with the reservoir at full capacity and the spillway operating (this condition was observed for extended periods during the summer of 2011). The reservoir elevation was assumed to be at a nominal value of 111.40 m (7 cm above the spillway crest elevation of 111.33 m).

2.4.2 Flood-Induced Failure

The flood-induced failure condition was assumed to occur under an extremely high reservoir level. This condition may occur due to an extremely large runoff event (e.g. a probable maximum flood). Alternatively, a spillway blocked with ice combined with an aggressive spring melt could also result in an extreme reservoir level. During the later case, it is reasonable to assume that many of the creek culvert crossings are still blocked with snow and ice (Appendix B provides photographs of the downstream culvert crossings taken during the winter of 2011, clearly depicting culverts blocked by snow and ice). This later condition was adopted for the flood-induced failure condition. Since the spillway is blocked, it is assumed that there is effectively no flow in the drainage channel downstream of the spillway.

2.5 Dam Breach Parameters

The breach hydrograph model, FLDWAV, requires the input of several parameters used to simulate the development of a breach. These parameters describe the breach geometry and the expected failure time using the following: average breach width, breach side-slope ratio, and the breach formation time. Using reported case studies, Froehlich (2008) has developed comprehensive equations to estimate the breach parameters. This recently developed approach was adopted for this study. The adopted parameters were validated by comparison to Alberta recommendations for predicting the breach parameters based on the dam geometry and material. (Alberta Environmental Protection Dam Safety Branch, 1995).

2.5.1 Concrete Dam and Spillway Failure

Average Breach Width

The average breach width was determined by inspecting the dam cross-section and identifying the weak points in the concrete. The average breach width was taken to be the full spillway width (15.3 m) plus a portion of the South Dam (9 m), up to the first expansion joint, for a total average breach width of 24.3 m.

The Alberta Dam Safety Branch recommends the following for estimating the average breach width of a concrete gravity dam:

$$\bar{B} \leq 0.5\,W \bar{B} \leq 0.5\,W$$

Where $\bar{B}\bar{B}$ is the average breach width and *W* is the dam crest length. For a total dam crest length *W,* for the North Dam, Spillway, and South Dam, of 67.7 m, we obtain $\bar{B} \leq 34$, thus an average breach width of 24.3 m is within the recommended range.

Breach Side Slope Ratio

As previously stated, it is accepted that concrete gravity dams will fail in vertical sections. The Alberta Dam Safety Branch recommends the following for the horizontal component of the side slope of concrete dam breach:

$$Z = 0Z = 0$$

Thus vertical side slopes for the breach opening is vaildated.

Breach Formation Time

The Alberta Dam Safety Branch recommends the following for estimating the breach formation time of a concrete dam:

$$0.1\ hours\ \leq t_f \leq 0.3\ hours$$

Where $t_f t_f$ is the breach formation time. The breach formation time was selected to be 0.2 hours as it falls in the middle of the recommended range.

Summary Breach Parameters

Table 3 summarizes the dam breach parameters for the Spillway and partial South Dam breach.

Table 3. Dam Breach Parameters – Concrete Dam and Spillway Failure

Parameter	Sunny-Day Failure	Wet Weather Overtopping Failure
Spillway Crest	El. 111.33 m	El. 111.33 m
Dam Crest	El. 112.28 m	El. 112.28 m
Reservoir level at failure[1,2]	El. 111.4 m	El. 112.58 m
Final elevation of breach[3]	El. 101.3 m	El. 101.3 m
Average breach width	24.3 m	24.3 m
Breach side slopes	vertical	Vertical
Breach formation time	0.2 hours	0.2 hours
Initial flow out of the reservoir[4]	1 m^3/s	1 m^3/s
Downstream boundary condition	Normal depth	Normal depth

Notes:
1. Reservoir level for sunny-day failure set to 0.07 m above the spillway crest, so the spillway is actively passing flow.
2. Reservoir level for wet weather overtopping failure set to 0.3 m above the dam crest.
3. Final elevation of breach corresponds to bedrock elevation.
4. Initial outflow, included for model stability reasons, was set to 1 m^3/s. As soon as the dam starts to fail, the FLDWAV model reduces this flow to zero so it does not impact the results.

2.5.2 Central Berm Failure

Using 69 recorded case studies, the following equation was developed by Froehlich (2008) to predict the average breach width:

$$\overline{B} = 0.27 k_o V_w^{0.32} H_b^{0.04}$$

where k_o = 1.3 for overtopping failures and 1.0 for other failures; V_w is the volume of water above the bottom of the breach; and, H_b is the expected height of the breach.

For the current study the breach is assumed to develop to its maximum height equal to the height of the embankment, which for the central berm equals 4.5 m. The average breach width computed from the above equation then equals 36 m for the wet weather overtopping failure and 24 m for the sunny-day piping failure.

The Alberta Dam Safety Branch recommends the following for estimating average breach width, based on a multiple of the height of dam (HD):

$$HD \leq \overline{B} \leq 5HDHD \leq \overline{B} \leq 5HD$$ (usually between 2HD and 4HD)

The breach widths computed from the Froehlich equation above, of 36 m for the wet weather overtopping failure and 24 m for the sunny-day piping failure, correspond to values of 8HD and 5HD, respectively. Those values are high compared to the recommend range, however they are considerate appropriate because the Lake Geraldine reservoir volume is unusually large compared to the length of the dam, resulting in a longer period of discharge and more extensive erosion of the breach.

Breach Side Slope Ratio
Based on 70 case studies, Froehlich (2008) determined that the breach side slope is dependent on the failure mode. It was found that the horizontal component of the breach side slope, *z, equals* 1.0 for overtopping failures and 0.7 for other failure modes. Thus, z = 1.0 for the PMF overtopping failure and z = 0.7 for the sunny-day piping failure.

The Alberta Dam Safety Branch recommends the following for estimating the horizontal component of the breach side slope:

$$\frac{1}{4} \leq z \leq 1$$

The estimated values of z = 1.0 for the wet weather overtopping failure and z = 0.7 for the sunny-day piping failure fall into the recommended range.

Breach Formation Time
Froehlich (2008) developed the following equation for estimating the breach formation time based on 23 case studies:

$$t_f = 63.2\sqrt{\frac{V_w}{gH_b^2}} t_f = 63.2\sqrt{\frac{V_w}{gH_b^2}}$$

where V_w is the volume of water above the bottom of the breach; g is the gravitational acceleration; and, H_b is the expected height of the breach, taken in this case as equal to the height of the embankment.

For the Central Berm, the computed breach formation time is 1.6 hours for the wet weather overtopping failure and 1.3 hours for the sunny-day piping failure.

The Alberta Dam Safety Branch recommends the following for estimating the breach formation time:

$$0.1\ hr \leq t_f \leq 1.0\ hr$$

The computed values of 1.6 hours for the wet weather overtopping failure and 1.3 hours for the sunny-day piping failure are high compared to the recommend range, however they are considerate appropriate because the Lake Geraldine reservoir volume is unusually large compared to the length of the dam, resulting in a longer period of discharge and more extensive erosion of the breach.

Summary Breach Parameters

Table 4 summarizes the dam breach parameters for the Central Berm Breach.

Table 4. Dam Breach Parameters – Central Berm Failure

Parameter	Sunny-Day Failure	Wet Weather Overtopping Failure
Embankment crest	El. 112.5 m	El. 112.5 m
Reservoir level at failure[1,2]	El. 111.4 m	El. 112.8 m
Final elevation of breach[3]	El. 108.0 m	El. 108.0m
Average breach width	24 m	36 m
Breach side slopes (H:V)	0.7:1	1:1
Breach formation time	1.3 hours	1.6 hours

Notes:

1. Reservoir level for sunny-day failure set to 0.07 m above the spillway crest, so the spillway is actively passing flow.
2. Reservoir level for wet weather overtopping failure set to 0.3 m above the embankment crest.
3. Final elevation of breach corresponds to bedrock elevation.

3.0 RESULTS

3.1 Breach Hydrographs

Figure 4 presents the breach hydrographs for the four breach scenarios described in the previous section:

- Concrete Dam and Spillway – Sunny Day Failure
- Concrete Dam and Spillway – Wet Weather Failure
- Central Berm – Sunny Day Failure
- Central Berm – Wet Weather Failure

Table 5 presents a summary of the breach hydrograph properties for the four breach scenarios. The concrete dam and spillway failures release the largest volume of water and provide the highest peak discharge since the concrete dam and spillway failure have a much lower bottom of breach elevation and shorter breach formation time (refer to Tables 3 and 4). Clearly, the *concrete dam and spillway – wet weather* failure scenario is the critical dam breach scenario, and thus was selected for routing and inundation mapping.

Table 5. Summary of Dam Breach Hydrograph Properties

Hydrograph Property	Concrete Dam and Spillway		Central Berm	
	Sunny Day Failure	Wet Weather Failure	Sunny Day (Piping) Failure	Wet Weather (Overtopping) Failure
Peak Discharge (m^3/s)	1,219	1,436	142	305
Hydrograph Volume (m^3)	1,938,000	2,424,000	1,063,000	1,645,000
Time to peak Discharge (minutes)	12	12	66	96

The computed peak discharge for the concrete dam and spillway – wet weather failure scenario (1,436 m^3/s) was compared to observed dam breach events presented in the literature based on the depth of water at dam failure and the total reservoir storage (Wahl, 1998). It can be seen in Figure 5 and 6 that the computed value relates well to observed dam breach events elsewhere.

3.2 Flood Wave Routing and Inundation

Figure 7 presents the advance of inundation due to the flood wave, for selected times, T (in minutes), following the start of the breach. The maximum extent of inundation is also shown. The flood wave reaches the first crossing at Suputi Road in approximately 8 minutes. The wave then overtops Suputi Road and continues down the unnamed drainage channel, reaching the second crossing where Suputi Road meets Niaqunngusiaq Road (next to the General Hospital) in less than 10 minutes. The flood wave overtops the Suputi-Niaqunngusiaq Road Crossing (north of Inuksuk High School) and then the flow splits west down along Niaqunngusiaq Road and east along the drainage channel. The flow to the west, fans out over the flat area and eventually reaches Koojesse Inlet in approximately 20 minutes. Flows to the east, following the existing drainage channel reach the inlet in approximately 16 minutes. The maximum extent of inundation is reached in approximately 30 minutes. Figure 7 is provided to help illustrate the progression of the flood wave in time.

Figures 8 and 9 present a summary of the results of the flood wave routing and inundation analysis, illustrating maximum depths and velocities, respectively. Details on the flood wave properties are described below.

3.2.1 Maximum Flood Depth

Figure 8 details the extent of inundation, flood wave arrival time, and maximum flood depth. The maximum flood depth is illustrated by color bands. Maximum flood depths in excess of 5 m are computed along the primary flood paths. Depths across the main road crossings reach 3.5 meters. During the flood event, the roads will not be passable. It is likely that the roads will also be washed out during the flood event. Below the flow split, flow depths will range from 0.2 to 2.0 m. The passing flood wave will likely lift and float away cars and unsecured buildings as it passes through the residential areas downstream of the split. Significant erosion and damage to roads are also expected.

3.2.2 Maximum Velocity

Figure 9 details the extent of inundation, flood wave arrival time, and maximum velocity. The maximum velocity is illustrated by color bands. Maximum velocities in excess of 14 m/s are computed along the primary flood paths. Velocities across the main road crossings reach 10 m/s. Velocities downstream of the split, exceed 1 m/s throughout most of the full extent of the flood wave. Persons in the path of the flood way would likely be swept away in these high velocities. These velocities are sufficiently high to cause significant erosion and scour throughout the entire model domain.

3.2.3 Critical Infrastructure at Risk

The critical infrastructure at risk, as identified by the City, is indicated by alphabetically-labelled red polygons on Figures 8 and 9. Table 6 presents a summary of the maximum flood depth, maximum flood velocity, flood arrival time, and relative danger for the critical infrastructure. The relative danger is based on inspection of the computed flood wave velocity and depth at (or near) the critical infrastructure. The relative danger classification is based on the *Downstream Hazard Classification Guidelines* published by the U.S. Department of the Interior Bureau of Reclamation (USBR, 1988). Relevant excerpts from this publication are provided in Appendix C. The relative danger adopted for this study is based on the depth-velocity flood danger level relationship for adults (see Appendix C). A "high" level is recommended for all areas impacted by flood wave. The descriptions for the three levels were adopted from the USBR as follows.

- **LOW**: Denotes a relatively low danger where the number of lives in jeopardy is assumed to be zero.
- **MODERATE**: Denotes a moderate level of danger where some lives may be in jeopardy. Local knowledge and judgement are required to better estimate the potential for lives in jeopardy in this category.
- **HIGH**: Denotes a relatively high level of danger where lives are in jeopardy.

The above levels of danger are subjective and provide a relative comparison only. The flood wave passes through the City rapidly (less than 20 minutes). In such a short period of time, it may not be practical for emergency planners and or responders to differentiate between the various levels of danger. It may be more feasible to assume a high level of danger throughout the entire limits of inundation. Further, it may be most practical to consider the time at which a high level of danger develops or exists (or when lives are in jeopardy) to be immediate throughout the extent of inundation.

Table 6. Maximum Flood Depth, Maximum Flood Velocity, Flood Arrival Time, and Relative Danger for Critical Infrastructure

ID	Critical Infrastructure	Maximum Flood Depth (m)	Maximum Flood Velocity (m/s)	Approximate Flood Arrival Time[1] (minutes after breach)	Relative Flood Danger
A	Suputi Road - DAM ACCESS	2.0 to 3.0	10 to 12	6 to 8	High
B	Saputi/Niaqunngusiaq Crossing	2.0 to 3.5	6 to 9	8 to 10	High
C	West Hospital Access	0.5 to 2.0	8 to 10	10	High
D	Qikiqtani General Hospital	0	0	10	Low
E	Inuksuk High School	0	0	16	Low
F	Brown Building	0	0	16	Low
G	Creekside Village	0.5 to 2.0	1 to 8	12 to 14	High
H	Arctic Ventures	1	3 to 5	16	High
I	Grind and Brew	1.5 to 2.0	3 to 4	16	High
J	Library	0.5	1 to 3	22	High
K	Old Courthouse	0 to 0.2	0 to 0.3	30	Moderate
L	Elders Center	0	0	30 (nearby flooding)	Moderate
M	Northwestel Building	0.3	0.5 to 0.9	26	Moderate
N	Northmart	0 to 0.2	0 to 0.3	20	Moderate
O	Nakasuk School	0	0	30 (nearby flooding)	Low
P	Anglican Church	0	0	30 (nearby flooding)	Low
Q	Post Office	1.2	3 to 4	14	High
R	Capital Suites	0.5	1 to 2	24	High
S	Qamutiq Building	0.5	1 to 2	18	High
T	4 Corners	0.5	2 to 2.5	18 to 20	High
U	Royal Canadian Legion	0 to 0.2	0.2 to 0.5	30 to 60	Moderate
V	Legislature	0.4	0.8 to 1.5	22	High
W	Hotel Arctic	0.3 to 0.6	1.5 to 2.5	18	High
X	City Hall / Fire Hall	0.6	1.3 to 3	12 to 14	High
Y	RCMP Depot	0 to 0.3	0 to 0.3	30 to 60	Moderate
Z	Sewage Lift Station #1	0.25	1.5	20 to 22	High

[1]Approximate Flood Arrival Time is Representative of Maximum Flood Depth Arrival Time.

4.0 REFERENCES

Canadian Dam Association (CDA), 2007. *Dam Safety Guidelines and Technical Bulletin: Hydrotechnical Considerations for Dam Safety*.

Alberta Environmental Protection Dam Safety Branch (Canada), 1995. *Dam Owners' Guidelines for the Preparation of Emergency Preparedness Plans*. Publication D.S.B 6A, June 1995.

Fread, D.L., 1998. *NWS FLDWAV Model*. Hydrologic Research Laboratory, Office of Hydrology, National Weather Service (NWS), NOAA, Silver Spring, Maryland, 28 November 1998.

Froehlich, David C., 2008. *Embankment Dam Breach Parameters and Their Uncertainties*. Journal of Hydraulic Engineering, Vol. 134, No. 12, Pages 1708-1721.

USBR, 1988. *Downstream Hazard Classification Guidelines*. ACER Technical Memorandum No. 11, U.S. Department of the Interior, Bureau of Reclamation, Assistant Commissioner – Engineering and Research, Denver, Colorado, July 1998.

Wahl, Tony L., 1998. *Prediction of Embankment Dam Breach Parameters*. Water Resources Research Laboratory, U.S. Department of the Interior, Bureau of Reclamation, Dam Safety Office, July 1998.

5.0 CLOSURE

This report has been prepared for the exclusive use of the City of Iqaluit This report is based on, and limited by, the interpretation of data, circumstances, and conditions available at the time of completion of the work as referenced throughout the report. It has been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made.

Yours truly,

AMEC Earth & Environmental

REGISTERED PROFESSIONAL ENGINEER
D.J. HEALY
LICENSEE
NT/NU
9 MAY 2012

Dan Healy, Ph.D., P.Eng.
Associate Water Resources Engineer
Direct Tel.: (780) 944-6367
Direct Fax: (780) 944-6365
E-mail: dan.healy@amec.com

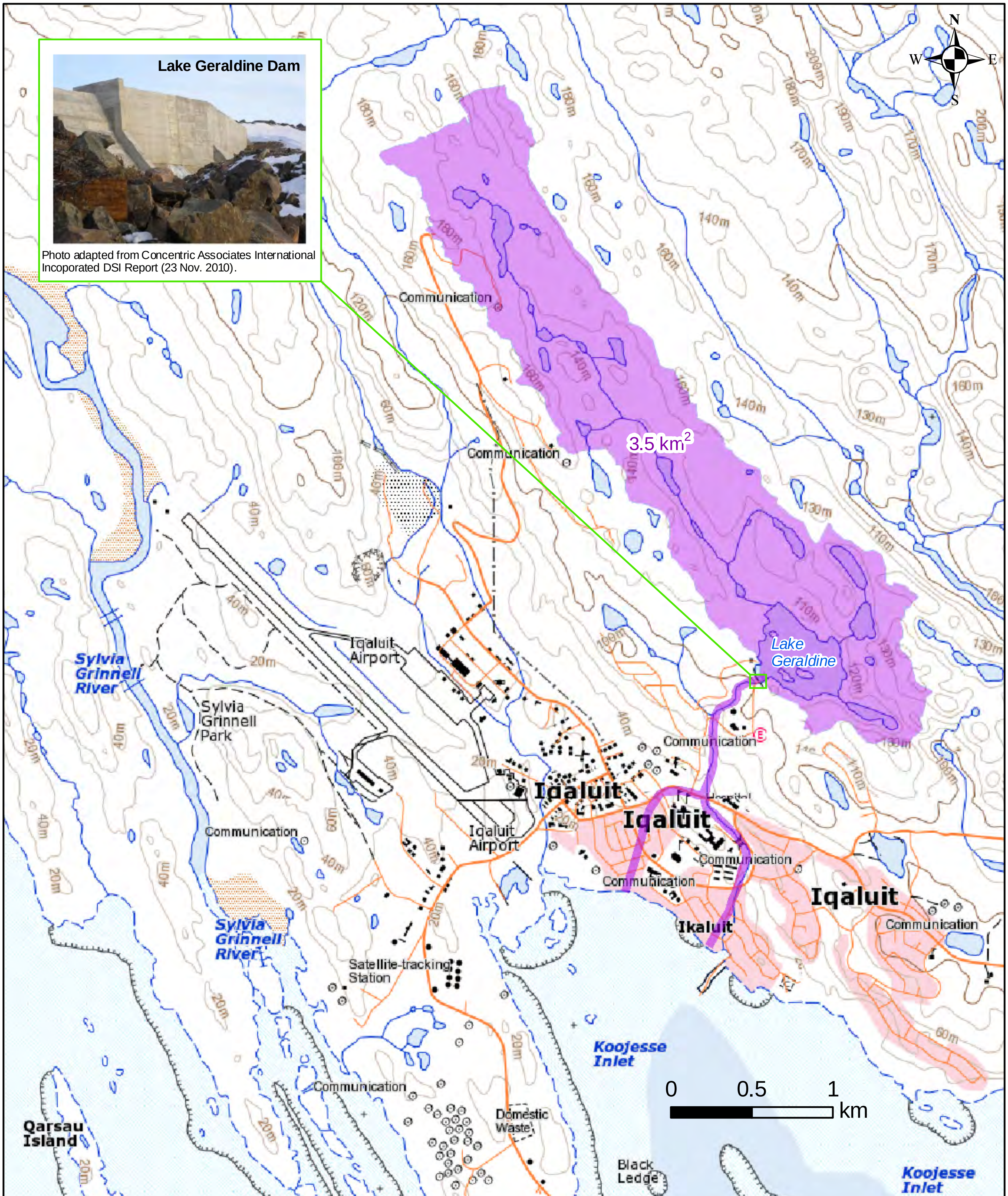
Reviewed By

REGISTERED PROFESSIONAL ENGINEER
LICENSEE
N.W.T.
9 May 2012

Neil van der Gugten, M.A.Sc., P.Eng.
Associate Hydrotechnical Engineer

DJH/NvdG

Figures



LEGEND:

Lake Geraldine Drainage Basin

Approximate Breach Path

CLIENT:

City of Iqaluit

LAKE GERALDINE DAM
DAM BREACH ANALYSIS &
INUNDATION MAPPING STUDY

May 2012

FIGURE 1

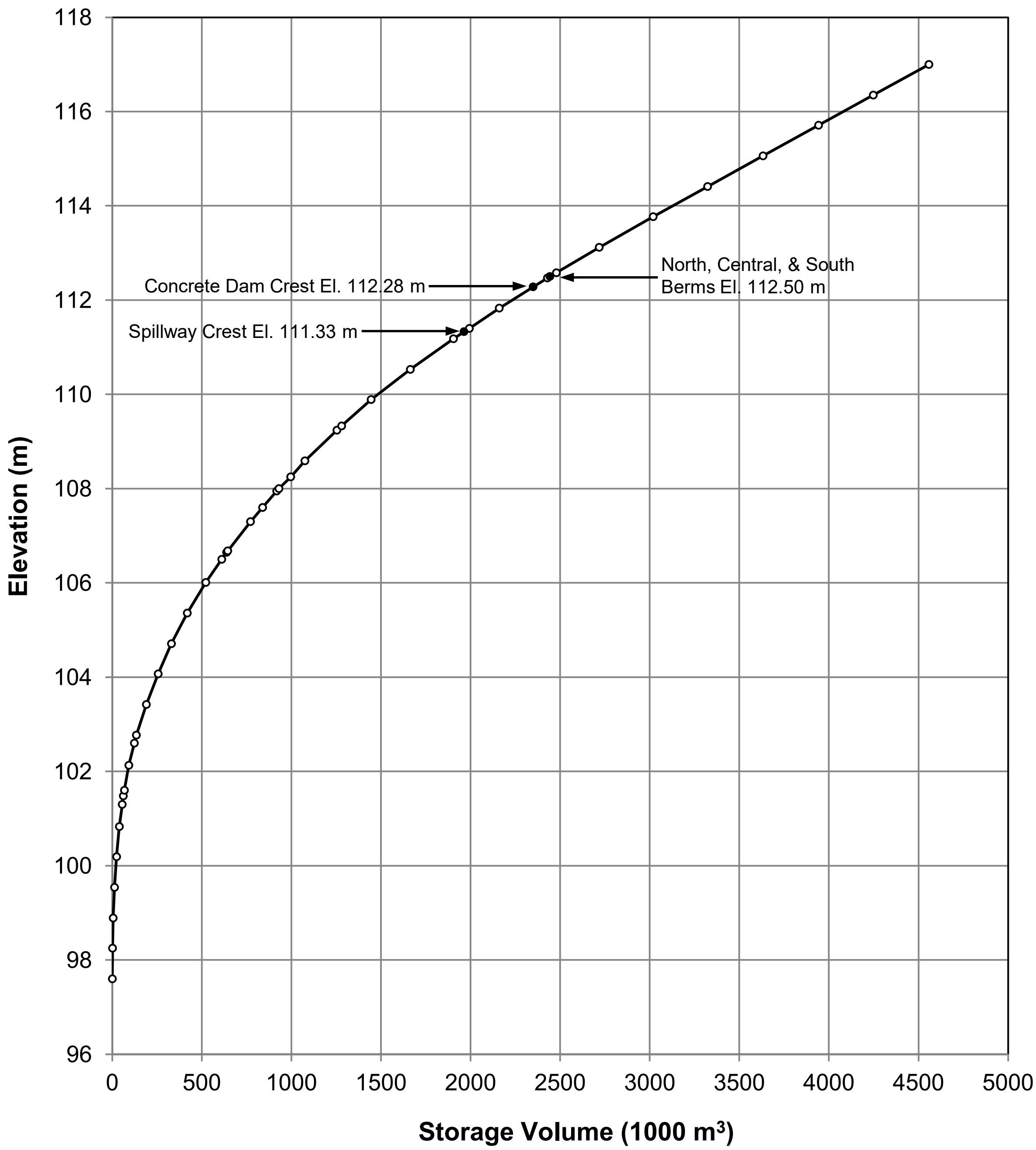
AMEC Project No.: EW1033

DWN BY: DJH

CHK'D BY: -

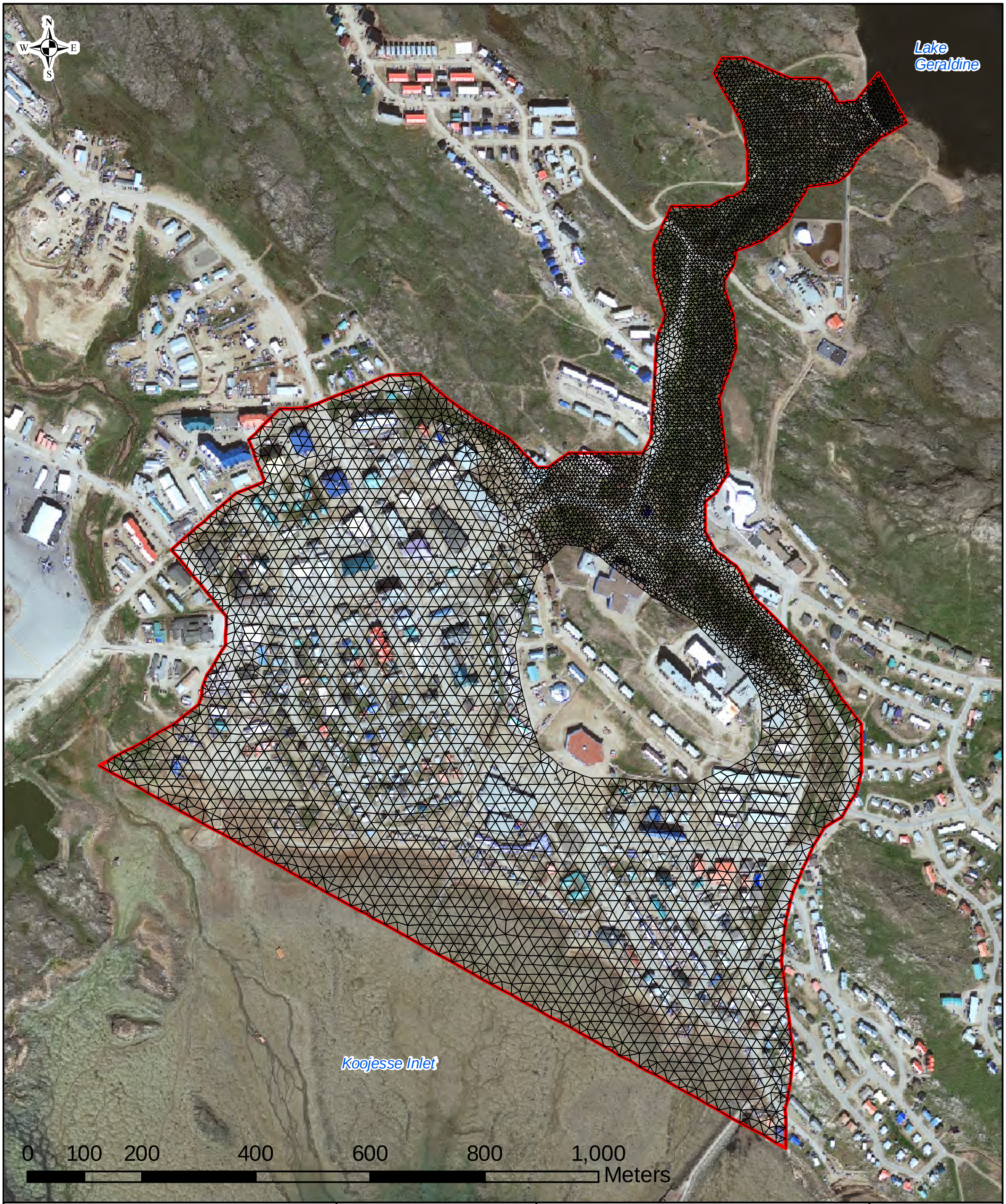
STUDY LOCATION

Map Path: L:\PROJECT\EW\1033 Lake Geraldine EPP\GIS\Figures



Note: Based on topography data provided by the City of Iqaluit and lake bathymetry data provided by National Research Council. (Watershed Mapping and Monitoring for Northern Community Impact Assessment, NRC, 2009.)

LEGEND:	CLIENT: City of Iqaluit	LAKE GERALDINE DAM DAM BREACH ANALYSIS & INUNDATION MAPPING STUDY	May 2012
			FIGURE 2
	AMEC Project No.: EW1033	LAKE STAGE-STORAGE RELATIONSHIP	amec
	DWN BY: DJH / CHK'D BY: N van der G		



LEGEND:

Model Domain

Model Mesh Link
(14,395 Model Elements)

CLIENT:
City of Iqaluit

AMEC Project No.: EW1033

DWN BY: DJH

CHK'D BY: -

LAKE GERALDINE DAM
DAM BREACH ANALYSIS &
INUNDATION MAPPING STUDY

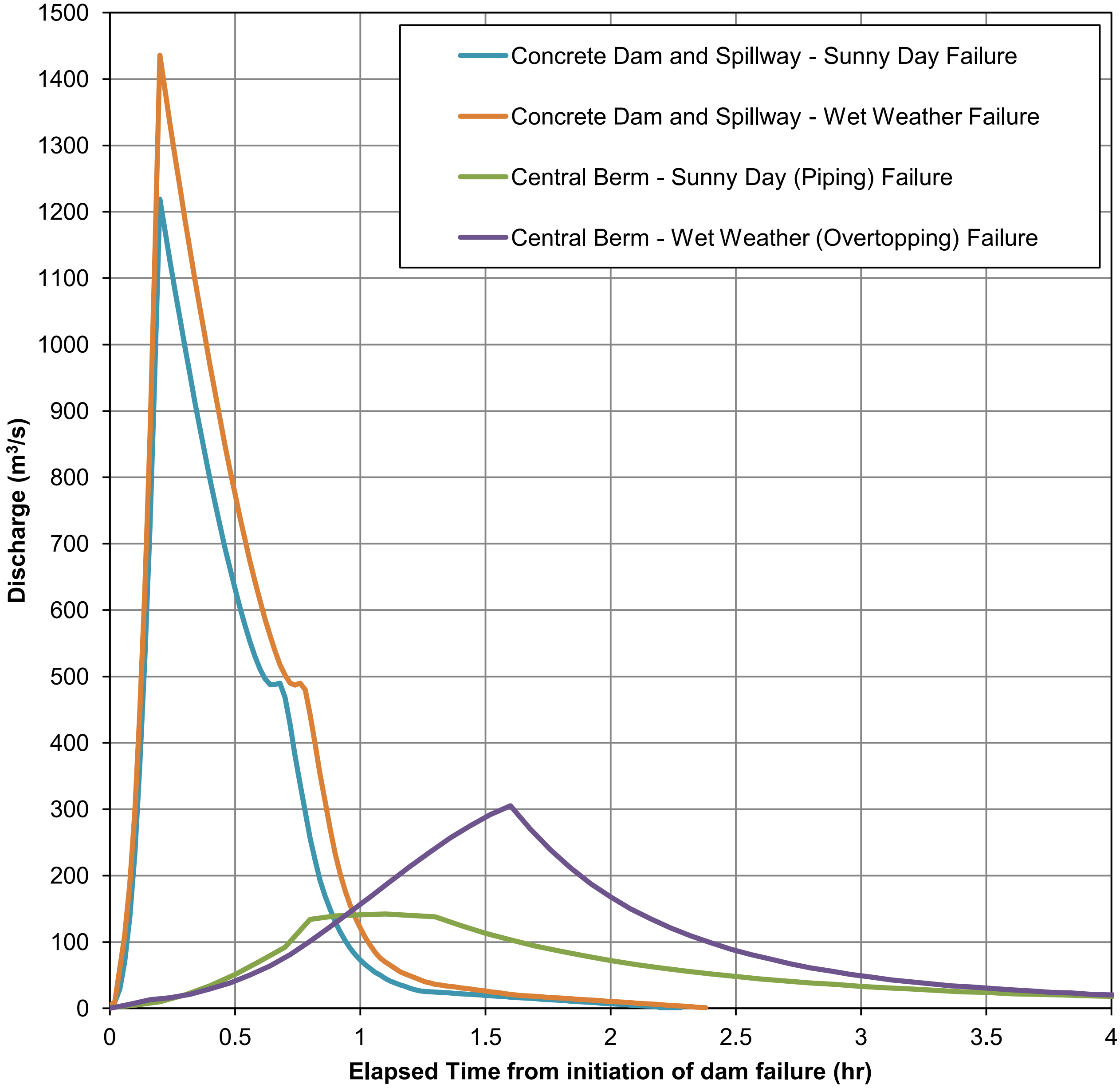
MODEL DOMAIN AND
COMPUTATIONAL MESH

May 2012

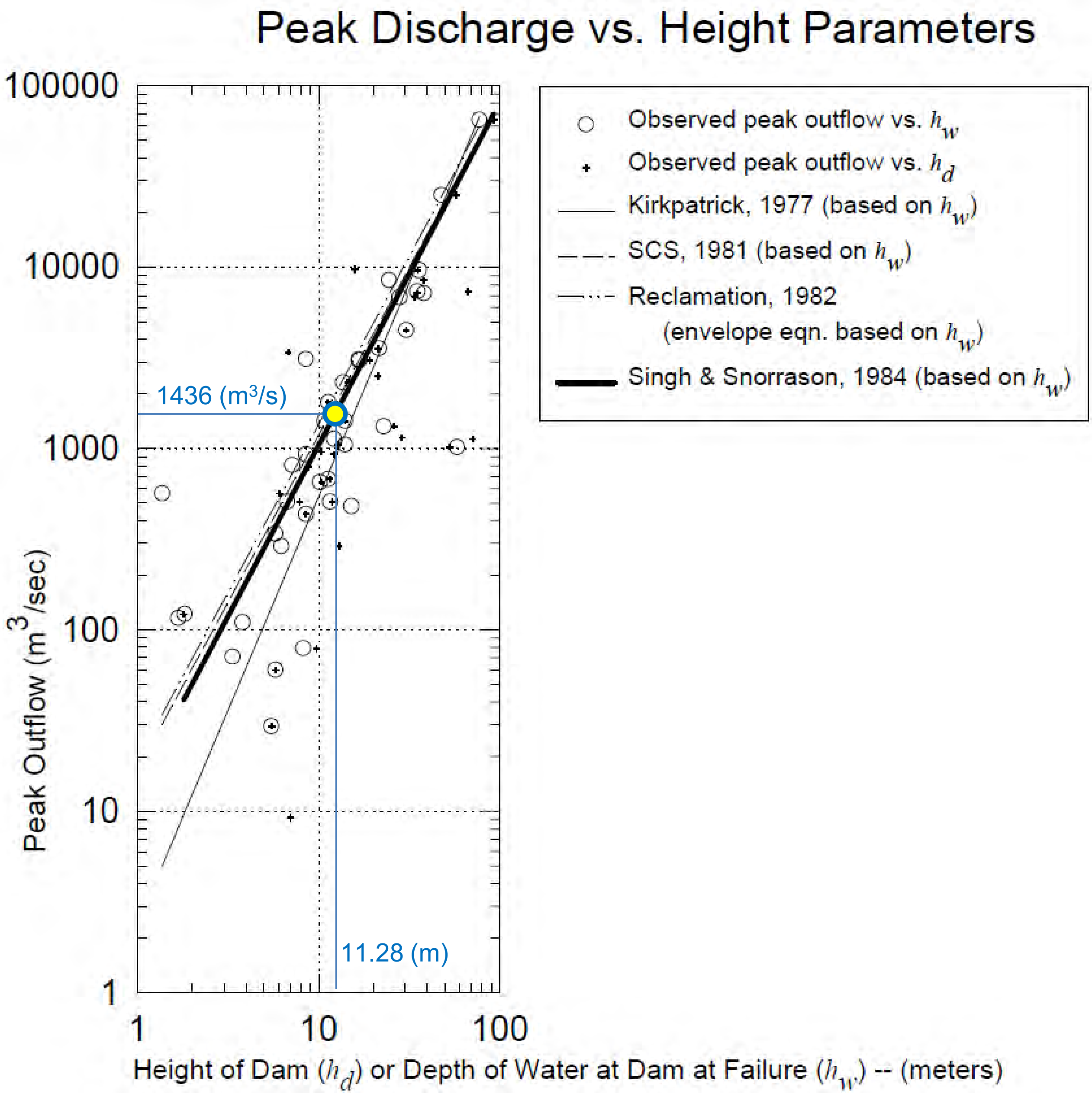
FIGURE 3

amec

Map Path: L:\PROJECT\EW\1033 Lake Geraldine EPP\GIS\Figures



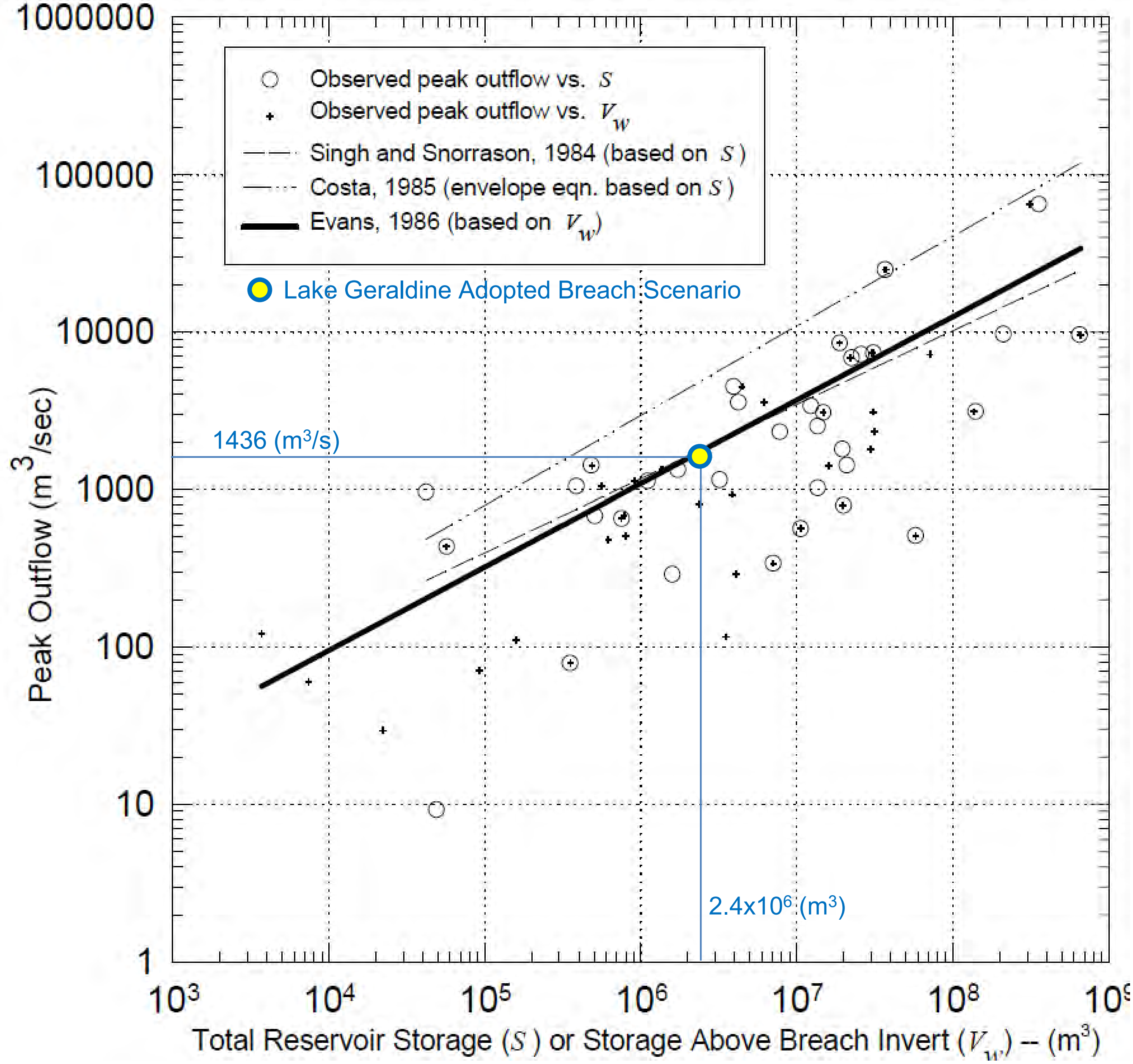
LEGEND:	CLIENT: **City of Iqaluit**	LAKE GERALDINE DAM DAM BREACH ANALYSIS & INUNDATION MAPPING STUDY	May 2012
			FIGURE 4
	AMEC Project No.: EW1033	BREACH HYDROGRAPHS	amec
	DWN BY: DJH / CHK'D BY: N van der G		



Note: Figure adopted from “Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment”, US Department of the Interior, Bureau of Reclamation, Dam Safety Office, July 1998.

LEGEND: Lake Geraldine Adopted Breach Scenario	CLIENT: **City of Iqaluit**	LAKE GERALDINE DAM DAM BREACH ANALYSIS & INUNDATION MAPPING STUDY	May 2012
	AMEC Project No.: EW1033	PEAK DISCHARGE AS A FUNCTION OF DEPTH AT FAILURE	FIGURE 5
	DWN BY: DJH / CHK'D BY: N van der G		amec

Peak Discharge vs. Storage Parameters



Note: Figure adopted from "Prediction of Embankment Dam Breach Parameters, A Literature Review and Needs Assessment", US Department of the Interior, Bureau of Reclamation, Dam Safety Office, July 1998.

LEGEND: Lake Geraldine Adopted Breach Scenario	CLIENT: **City of Iqaluit**	LAKE GERALDINE DAM DAM BREACH ANALYSIS & INUNDATION MAPPING STUDY	May 2012
			FIGURE 6
	AMEC Project No.: EW1033	PEAK DISCHARGE AS A FUNCTION OF VOLUME AT FAILURE	amec
	DWN BY: DJH / CHK'D BY: N van der G		

LEGEND:

Computed Extents of Inundation at Time, T (minutes) after Start of Breach

Maximum Extent of Inundation

CLIENT: City of Iqaluit		LAKE GERALDINE DAM DAM BREACH ANALYSIS & INUNDATION MAPPING STUDY	May 2012 FIGURE 7
AMEC Project No.: EW1033		EXTENT OF INUNDATION FOR TIME, T MINUTES AFTER START OF BREACH	amec
DWN BY: DJH	CHK'D BY: N van der G		

Map Path: L:\PROJECT\EW1033 Lake Geraldine EPP\GIS\Figures

ID	Critical Infrastructure	Maximum Flood Depth (m)	Maximum Flood Velocity (m/s)	Approximate Flood Arrival Time[1] (minutes after breach)	Relative Flood Danger
A	Suputi Road - DAM ACCESS	2.0 to 3.0	10 to 12	6 to 8	High
B	Saputi/Niaqunngusiaq Crossing	2.0 to 3.5	6 to 9	8 to 10	High
C	West Hospital Access	0.5 to 2.0	8 to 10	10	High
D	Qikiqtani General Hospital	0	0	10	Low
E	Inuksuk High School	0	0	16	Low
F	Brown Building	0	0	16	Low
G	Creekside Village	0.5 to 2.0	1 to 8	12 to 14	High
H	Arctic Ventures	1	3 to 5	16	High
I	Grind and Brew	1.5 to 2.0	3 to 4	16	High
J	Library	0.5	1 to 3	22	High
K	Old Courthouse	0 to 0.2	0 to 0.3	30	Moderate
L	Elders Center	0	0	30 (nearby flooding)	Moderate
M	Northwestel Building	0.3	0.5 to 0.9	26	Moderate
N	Northmart	0 to 0.2	0 to 0.3	20	Moderate
O	Nakasuk School	0	0	30 (nearby flooding)	Low
P	Anglican Church	0	0	30 (nearby flooding)	Low
Q	Post Office	1.2	3 to 4	14	High
R	Capital Suites	0.5	1 to 2	24	High
S	Qamutiq Building	0.5	1 to 2	18	High
T	4 Corners	0.5	2 to 2.5	18 to 20	High
U	Royal Canadian Legion	0 to 0.2	0.2 to 0.5	30 to 60	Moderate
V	Legislature	0.4	0.8 to 1.5	22	High
W	Hotel Arctic	0.3 to 0.6	1.5 to 2.5	18	High
X	City Hall / Fire Hall	0.6	1.3 to 3	12 to 14	High
Y	RCMP Depot	0 to 0.3	0 to 0.3	30 to 60	Moderate
Z	Sewage Lift Station #1	0.25	1.5	20 to 22	High

[1]Approximate Flood Arrival Time is Representative of Maximum Flood Depth Arrival Time.

LEGEND:

- (16) Flood Front Arrival Time, T (minutes)
- Maximum Extent of Innundation
- Critical Infrastructure
- Flood Front at Time, T (minutes)

Note: flood fronts ploted for 2, 4, 6, ..., 28, 30, & 60 minutes only

Max. Depth of Innundation (m)

- 5 - 10
- 3 - 5
- 2 - 3
- 1.5 - 2
- 1 - 1.5
- 0.75 - 1
- 0.5 - 0.75
- 0.25 - 0.5
- 0 - 0.25

LAKE GERALDINE DAM
DAM BREACH ANALYSIS &
INUNDATION MAPPING STUDY

MAXIMUM EXTENT, MAXIMUM DEPTH, AND FLOOD ARRIVAL TIME

CLIENT: **City of Iqaluit**

May 2012

AMEC Project No.: EW1033

FIGURE 8

DWN BY: DJH

CHKD BY: N van der G

SCALE: 1:2,500 (On 22x34" plot size)

amec

Map Path: L:\PROJECT\EW\1033 Lake Geraldine EPP\GIS\Figures

ID	Critical Infrastructure	Maximum Flood Depth (m)	Maximum Flood Velocity (m/s)	Approximate Flood Arrival Time[1] (minutes after breach)	Relative Flood Danger
A	Suputi Road - DAM ACCESS	2.0 to 3.0	10 to 12	6 to 8	High
B	Saputi/Niaqunngusiaq Crossing	2.0 to 3.5	6 to 9	8 to 10	High
C	West Hospital Access	0.5 to 2.0	8 to 10	10	High
D	Qikiqtani General Hospital	0	0	10	Low
E	Inuksuk High School	0	0	16	Low
F	Brown Building	0	0	16	Low
G	Creekside Village	0.5 to 2.0	1 to 8	12 to 14	High
H	Arctic Ventures	1	3 to 5	16	High
I	Grind and Brew	1.5 to 2.0	3 to 4	16	High
J	Library	0.5	1 to 3	22	High
K	Old Courthouse	0 to 0.2	0 to 0.3	30	Moderate
L	Elders Center	0	0	30 (nearby flooding)	Moderate
M	Northwestel Building	0.3	0.5 to 0.9	26	Moderate
N	Northmart	0 to 0.2	0 to 0.3	20	Moderate
O	Nakasuk School	0	0	30 (nearby flooding)	Low
P	Anglican Church	0	0	30 (nearby flooding)	Low
Q	Post Office	1.2	3 to 4	14	High
R	Capital Suites	0.5	1 to 2	24	High
S	Qamutiq Building	0.5	1 to 2	18	High
T	4 Corners	0.5	2 to 2.5	18 to 20	High
U	Royal Canadian Legion	0 to 0.2	0.2 to 0.5	30 to 60	Moderate
V	Legislature	0.4	0.8 to 1.5	22	High
W	Hotel Arctic	0.3 to 0.6	1.5 to 2.5	18	High
X	City Hall / Fire Hall	0.6	1.3 to 3	12 to 14	High
Y	RCMP Depot	0 to 0.3	0 to 0.3	30 to 60	Moderate
Z	Sewage Lift Station #1	0.25	1.5	20 to 22	High

[1]Approximate Flood Arrival Time is Representative of Maximum Flood Depth Arrival Time.

LEGEND:

16 Flood Front Arrival Time, T (minutes)

Maximum Extent of Innundation

K Critical Infrastructure

Flood Front at Time, T (minutes)

Note: flood fronts ploted for 2, 4, 6, ..., 28, 30, & 60 minutes only

Max. Velocity (m/s)

10 - 15	1.5 - 2
6 - 10	1 - 1.5
4 - 6	0.5 - 1
2 - 4	0 - 0.5

LAKE GERALDINE DAM
DAM BREACH ANALYSIS &
INUNDATION MAPPING STUDY

MAXIMUM EXTENT, MAXIMUM VELOCITY, AND FLOOD ARRIVAL TIME

CLIENT: **City of Iqaluit**

May 2012

AMEC Project No.: EW1033

FIGURE 9

DWN BY: DJH

CHKD BY: N van der G

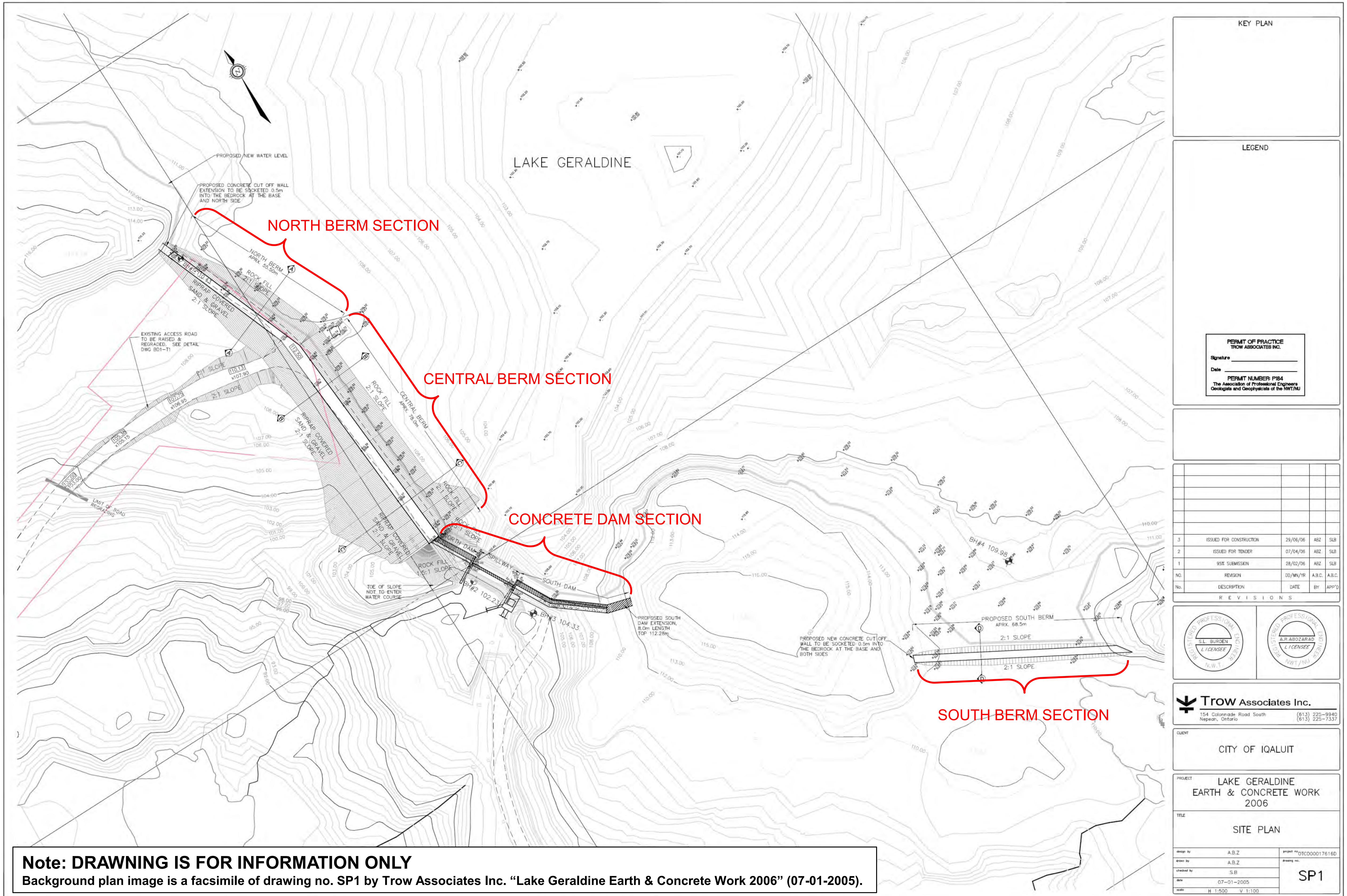
SCALE: 1:2,500 (On 22x34" plot size)

amec

Map Path: L:\PROJECT\EW\1033 Lake Geraldine EPP\GIS\Figures

Appendix A

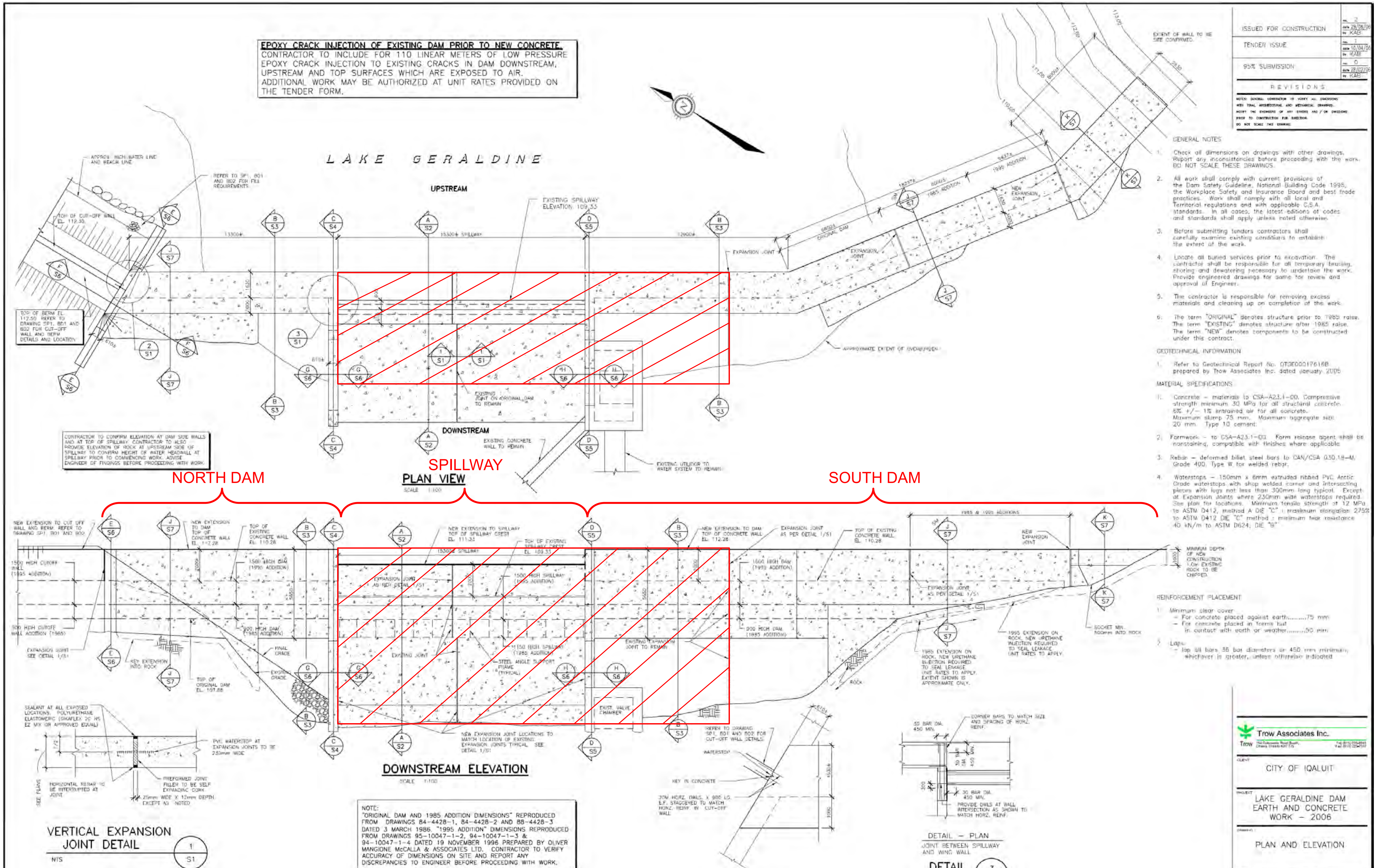
Selected Dam Design Drawings and Breach Section Schematic



Note: DRAWNING IS FOR INFORMATION ONLY

Background plan image is a facsimile of drawing no. SP1 by Trow Associates Inc. "Lake Geraldine Earth & Concrete Work 2006" (07-01-2005).

Plan View of Lake Geraldine Dam Breach Scenario Sections



Note: DRAWNING IS FOR INFORMATION ONLY

Background plan image is a facsimile of drawing no. S1 by Trow Associates Inc. "Lake Geraldine Earth & Concrete Work 2006" (07-01-2005).

Concrete Dam and Spillway Failure Section

Lake Geraldine Dam Concrete Dam Breach Scenario Sections

Appendix B

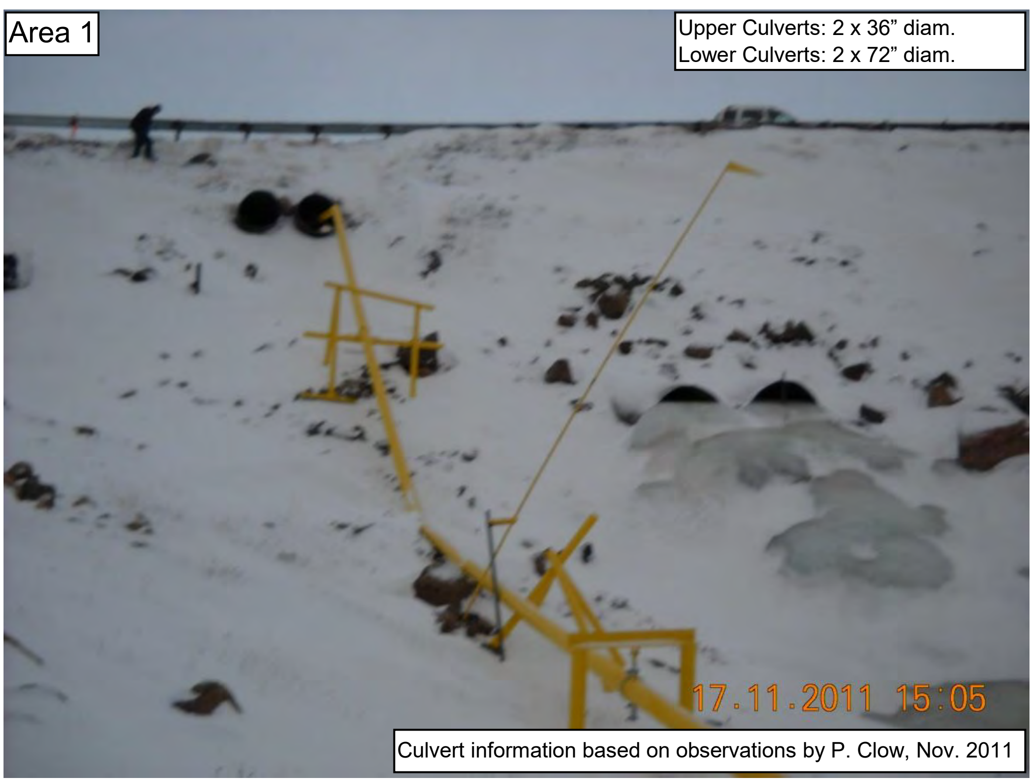
Culvert Crossing Photographs

CULVERT CROSSING PHOTOGRAPHS

Photos taken by Paul Clow, City of Iqaluit, November 2011.

Approximate Photo Locations Indicated Below

Saputi Road Crossing (looking downstream).



Saputi Road Crossing (looking upstream).

Niaqungusiariaq Road Crossing (looking downstream)

Niaqungusiariaq Road Crossing (looking upstream)

Elizabeth Way Crossing (looking downstream)

Elizabeth Way Crossing (looking upstream)

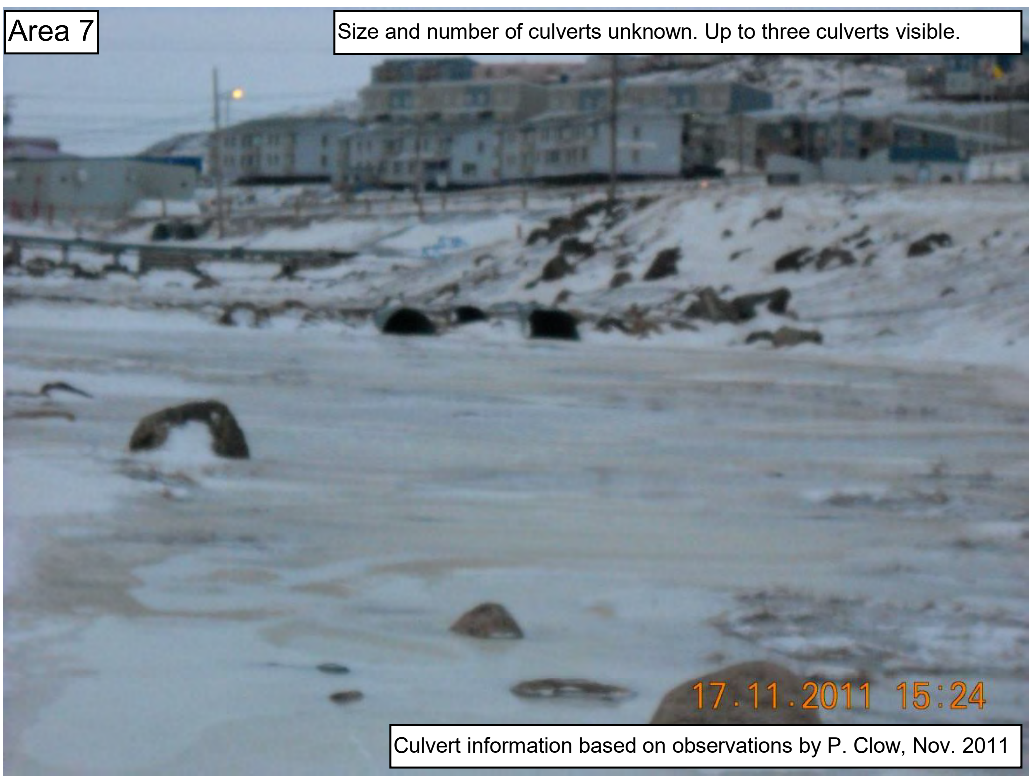
Nipisa Street Crossing (looking downstream)

Nipisa Street Crossing (looking upstream)

Sinaa Street Crossing (looking downstream)

Sinaa Street Crossing (looking upstream)

Astro Hill Terrace Bridge Crossing (looking downstream)



Pedestrian Crossing (upstream of Astro Hill Terrace Bridge Crossing)

Appendix C

Depth-Velocity Flood Danger Level Relationships

Depth-Velocity Flood Danger Level Relationships

The curves provided in this Appendix are facsimiles of figures taken from the following reference:

ACER TECHNICAL MEMORANDUM NO. 11
ASSISTANT COMMISSIONER - ENGINEERING AND RESEARCH
DENVER, COLORADO

DOWNSTREAM HAZARD CLASSIFICATION GUIDELINES

U.S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
1988

HIGH DANGER ZONE – Occupants of most houses are in danger from floodwater.
JUDGEMENT ZONE – Danger level is based upon engineering judgement.
LOW DANGER ZONE – Occupants of most houses are not seriously in danger from flood water.

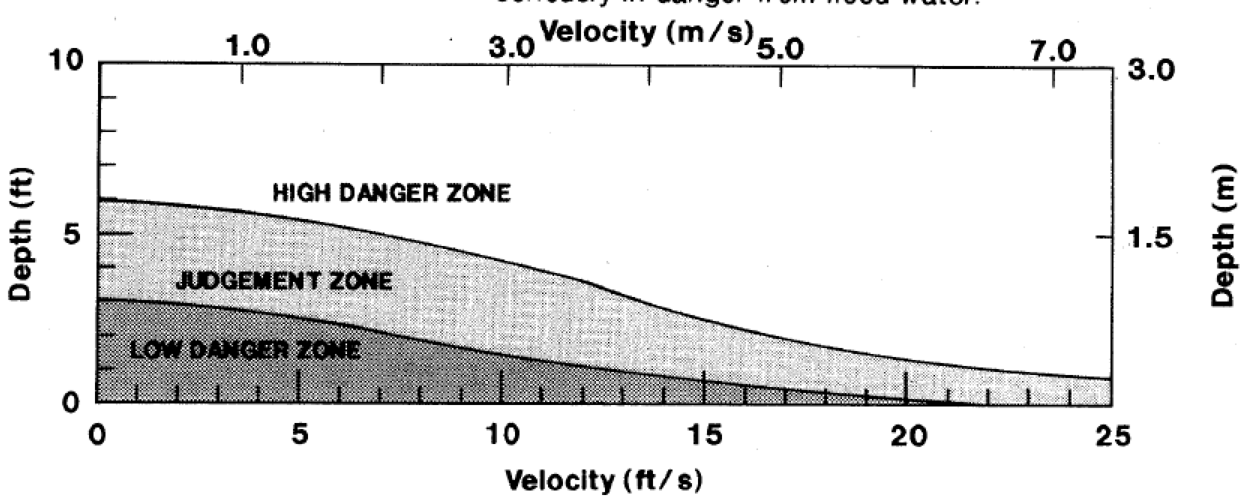


Figure 2. – Depth–velocity flood danger level relationship for houses built on foundations.

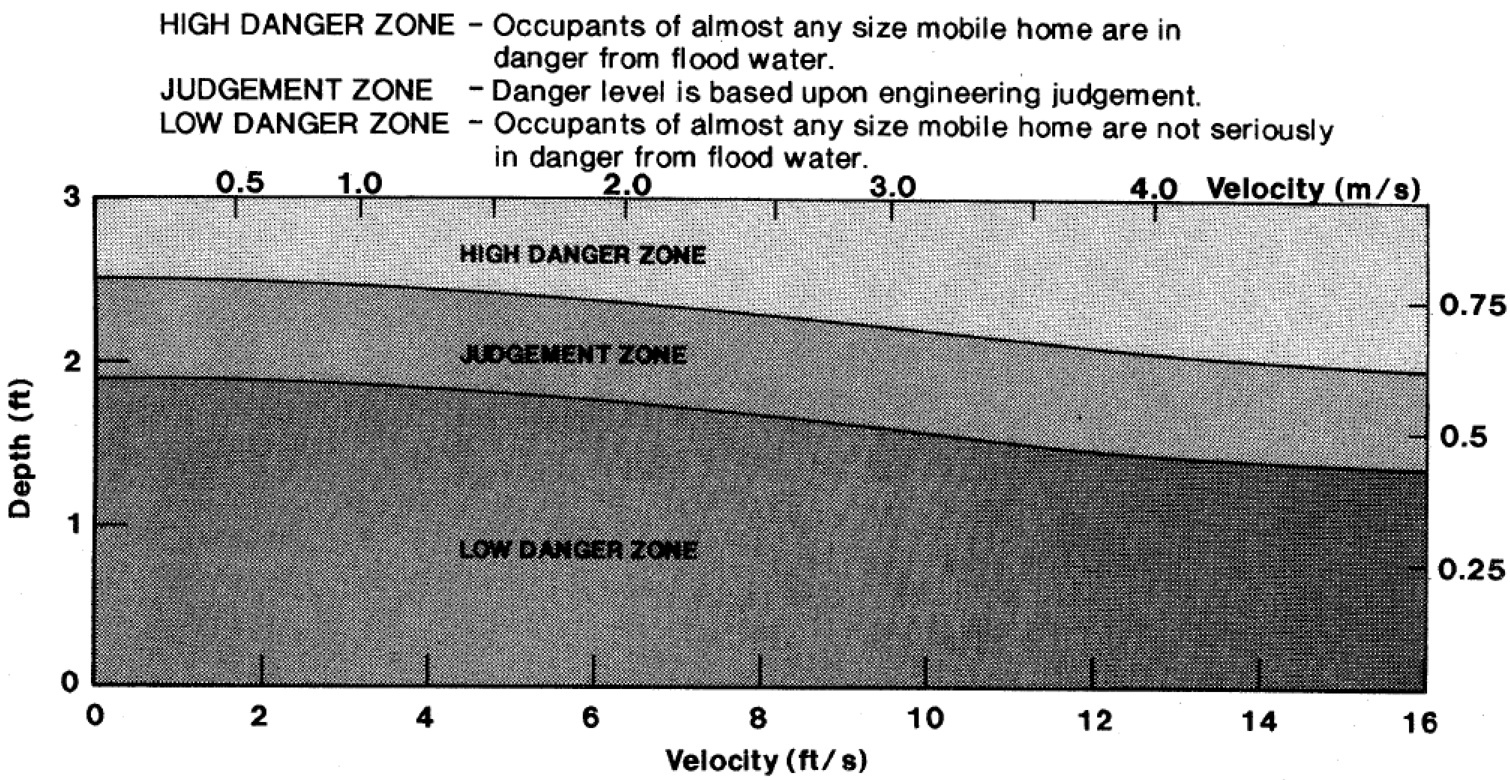


Figure 3. – Depth–velocity flood danger level relationship for mobile homes.

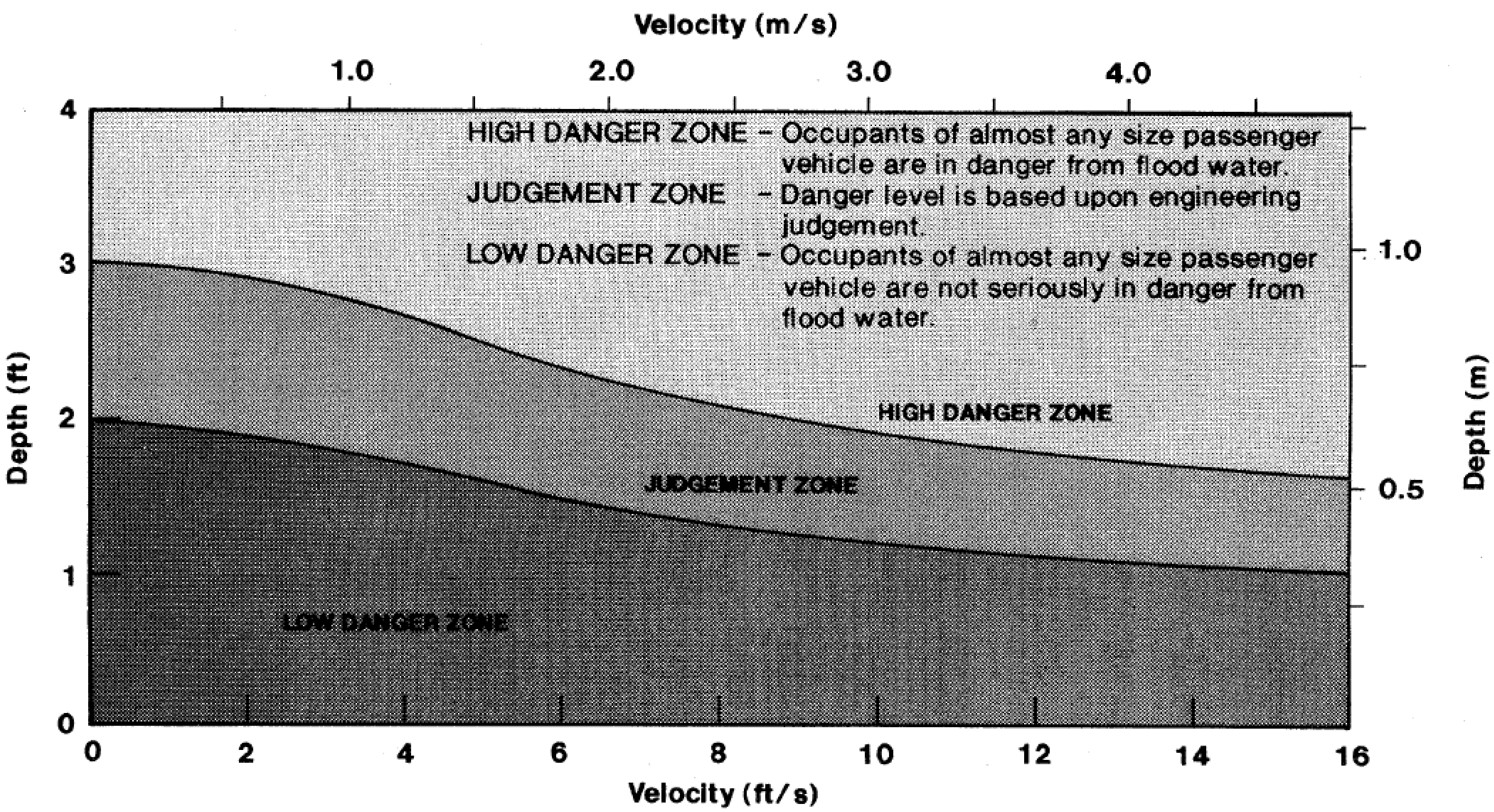


Figure 4. – Depth–velocity flood danger level relationship for passenger vehicles.

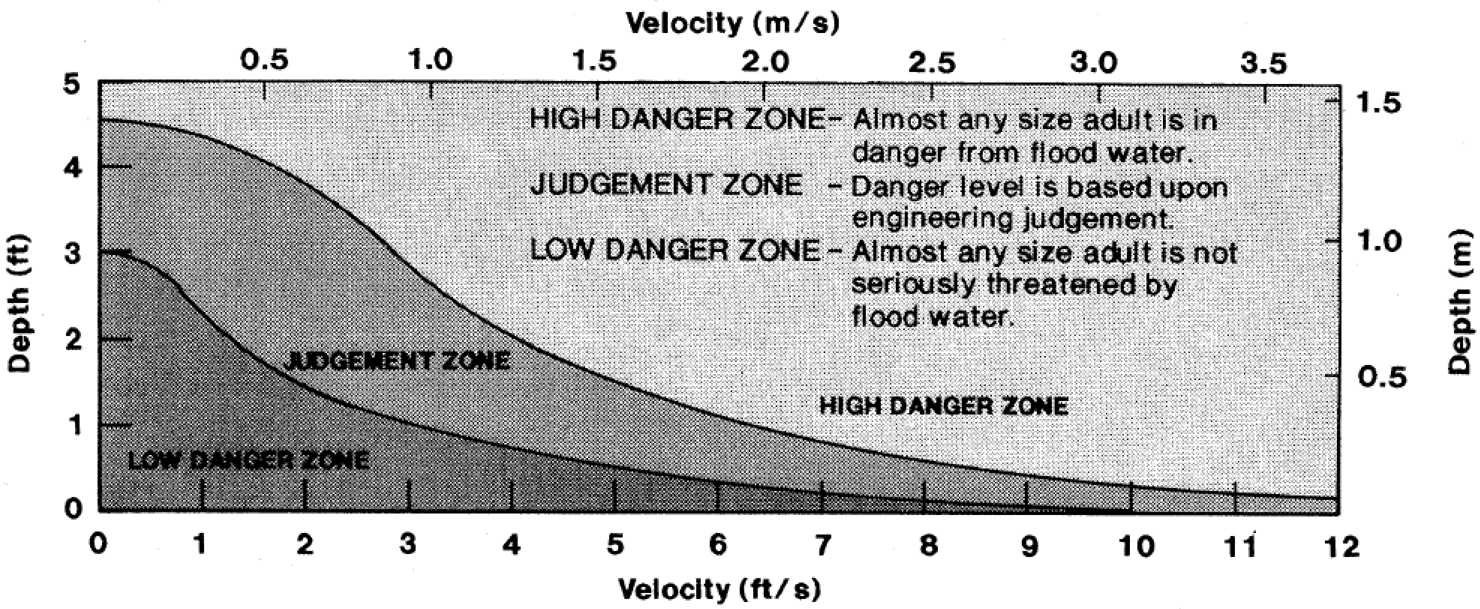


Figure 5. – Depth–velocity flood danger level relationship for adults.

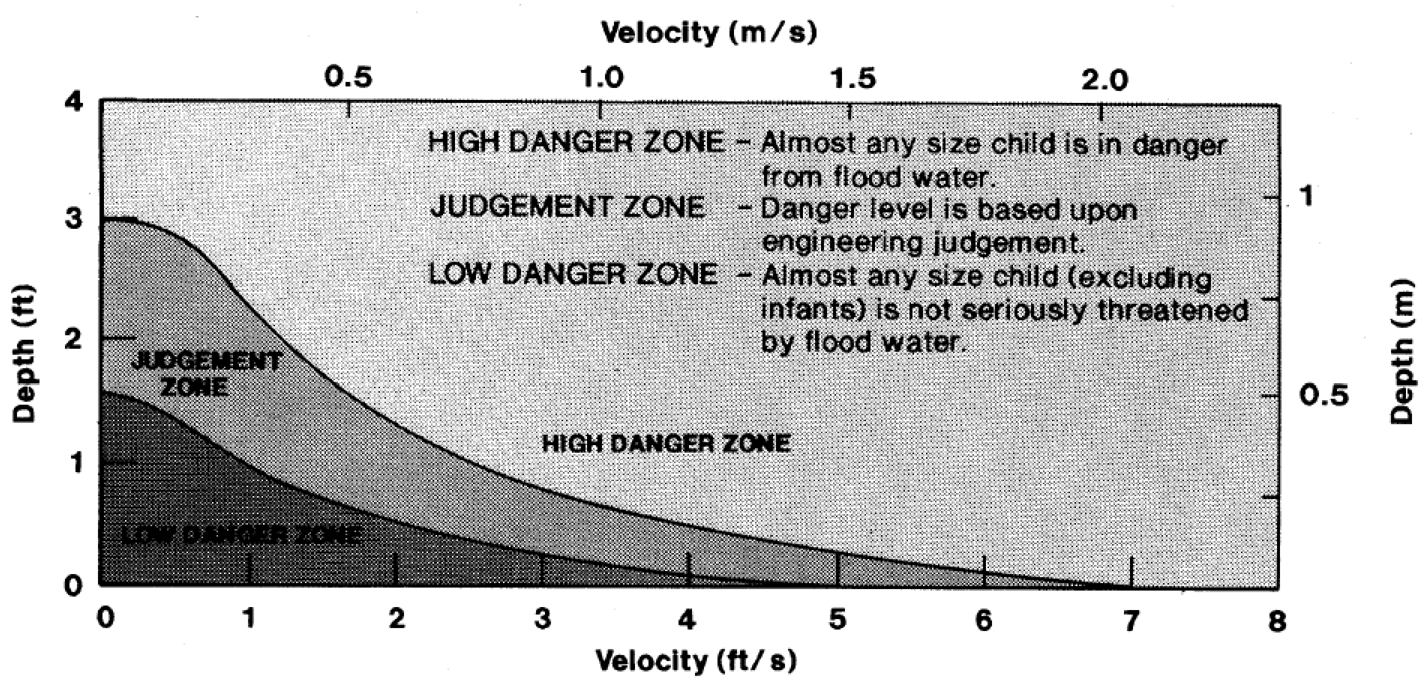


Figure 6. – Depth–velocity flood danger level relationship for children.

Appendix C

Glossary of Terms

Breach Damage to a dam that allows uncontrolled and undesirable flow out of the dam or its berms

EOC Emergency Operations Centre

EPP Emergency Preparedness Plan

ERP Emergency Response Plan

Flood An event that causes water levels greater than bank-full conditions for a given watercourse

Flood Warning Types

Potential
Imminent

GEOC Government Emergency Operations Centre

Hazardous Condition

Condition that poses no immediate threat to the structure, but if left unchecked may put the dam, its berms or appurtenant structures at risk. Notification is limited to within the organization, and no external notifications are required at this level.

ICS Incident Command System

SCP Site Command Post