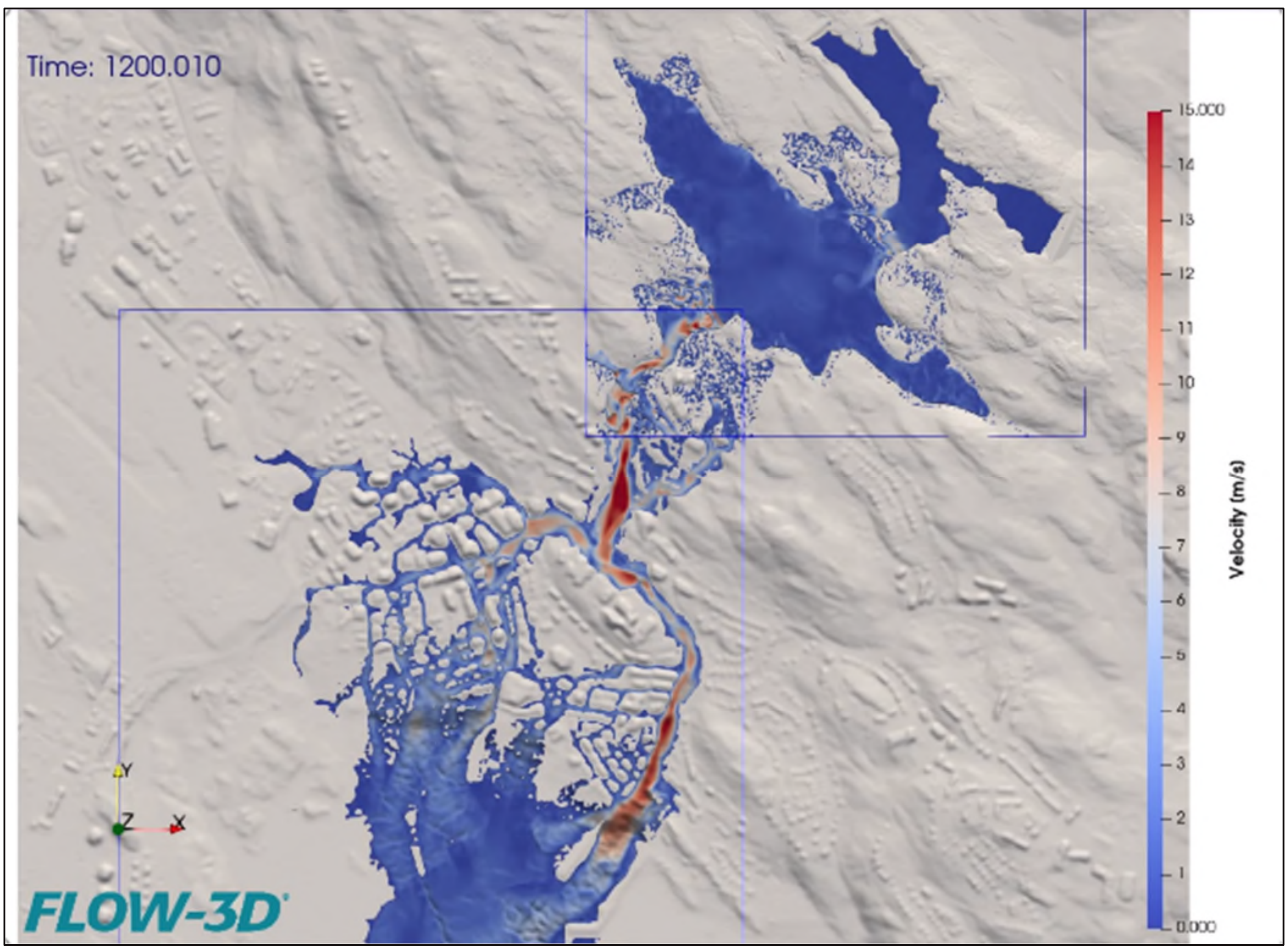
City of Iqaluit

Dam Breach and Inundation Analysis – DRAFT

New Reservoir Dam 1 Structure – 50% Design

August 2026



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Acronyms and Abbreviations

3D	Three-dimensional
CGVD2013	Canadian Geodetic Vertical Datum of 2013
CGVD28	Canadian Geodetic Vertical Datum of 1928
CSRS	Canadian Spatial Reference System
DEM	Digital Elevation Model
DTM	Digital Terrain Model
GIS	Geographic Information System
HR	High Resolution Digital Elevation Model
IDF	Inflow Design Flood
LiDAR	Light Detection and Ranging
LTWP	Long Term Water Project
masl	metres above sea level
NACA	National Advisory Committee for Aeronautics
NAD83	North American Datum of 1983
PMF	Probable Maximum Flood
RCMP	Royal Canadian Mounted Police
UAV	Unmanned Aerial Vehicle
UTM	Universal Transverse Mercator

Executive Summary

Arcadis prepared this preliminary Dam Breach and Inundation Analysis for the City of Iqaluit as part of the Long Term Water Project, which is being advanced to provide additional raw water storage for the community. The proposed new reservoir is located adjacent to Lake Geraldine and, at the current 50% design stage, is understood to comprise one principal dam structure and seven associated dyke structures. The work summarized in this report is intended to support design development, risk understanding, and future planning by illustrating how flood water could move if a breach were to occur under selected conservative assumptions.

This report is a draft and partial submission. Because the reservoir configuration, detailed dam and dyke designs, operating criteria, and related project assumptions have not yet been finalized, the analysis presented at this stage focuses on a theoretical worst-case scenario rather than the full suite of dam breach cases. The results should therefore be interpreted as preliminary and conditional on the information currently available. They are not intended to represent final design conclusions, emergency response mapping, or regulatory determinations.

The modelling completed to date considers a rainy-day complete breach of Dam 1. Dam 1 was selected for this preliminary analysis because it is the largest proposed structure in the reservoir system and is located upstream of, and closest to, the City of Iqaluit. Under the modelled rainy-day condition, the new reservoir was assumed to be at the Probable Maximum Flood water level, and Lake Geraldine was assumed to be at a conservative high-water level. The breach was represented as an instantaneous structural failure, which is a conservative simplifying assumption because it allows the full breach opening to release water immediately.

The analysis used a detailed hydraulic model developed with the available project survey, terrain, bathymetric, and hydrologic information. High-resolution topographic data, Lake Geraldine bathymetry, and broader regional elevation data were combined to represent the landscape over which flood water could travel. The modelling approach uses three-dimensional simulation near the dam and dyke structures, where flow behaviour is complex, and a shallow-water approach farther downstream, where the flood wave can be represented more efficiently over a larger area.

For the preliminary Dam 1 rainy-day breach scenario, the model indicates that released water would move from the proposed reservoir toward Lake Geraldine and then along drainage pathways toward the City. The key area of interest for this scenario is the southward flow path toward Iqaluit, including locations associated with existing drainage channels, access roads, the power plant area, Qikiqtani General Hospital, schools, community facilities, road crossings, and other important infrastructure. Monitoring points were selected throughout these areas to support interpretation of estimated flood depths, velocities, arrival times, and timing of peak conditions.

The inundation maps included in **Appendix B** show the preliminary maximum flooded area and related hydraulic information for the modelled Dam 1 rainy-day breach case. These maps are intended to help the project team and the City understand potential flow routes, areas where water could concentrate, and locations where critical access or services may require further review. They should be read together with the assumptions and limitations in the report, because mapped flood limits and hydraulic values may change as the reservoir design is refined and as additional modelling scenarios are completed.

A key purpose of this draft report is to demonstrate the modelling framework and identify the study areas that should be carried forward into the complete dam breach and inundation analysis. The final analysis is expected to include additional structural failure scenarios for Dam 1, Dyke 2, Dyke 6, and Dyke 8 under both sunny-day and rainy-day conditions. These future scenarios will allow the City and design team to compare potential flow paths toward Iqaluit and toward the Niaqunguk River valley and Apex area. Potential cascade effects involving the

existing Lake Geraldine dam have not been addressed in this preliminary report and are expected to be considered as part of the future complete analysis once the design is more advanced.

The results are influenced by several important assumptions. The breach is assumed to occur immediately rather than develop gradually over time; the reservoir and Lake Geraldine starting water levels are based on the selected hydrologic condition; surface roughness values are selected to represent expected flow routes; and buildings, road features, culverts, small surface obstructions, snow and ice conditions, debris, and future changes to site grading may not be fully captured. These assumptions are appropriate for a preliminary planning-level assessment, but they mean that site-specific depths, velocities, arrival times, and inundation limits should not be treated as final values.

From a client decision-making perspective, the main finding is that the proposed reservoir should continue to be evaluated using a structured dam breach and inundation modelling program as the design advances. The preliminary analysis identifies credible downstream flow paths and confirms that the Dam 1 rainy-day breach scenario warrants careful consideration because it is associated with potential flow toward populated areas and critical infrastructure in Iqaluit. The modelling provides a useful basis for focusing future analysis, but it should not be used on its own to make final emergency planning, land-use, infrastructure protection, or regulatory decisions.

Recommended next steps are to complete the full scenario matrix once the reservoir and dam/dyke designs are sufficiently advanced; review the sensitivity of results to key design and modelling assumptions; evaluate any potential interaction with the existing Lake Geraldine dam; and use the updated results to support dam safety documentation, emergency preparedness planning, infrastructure review, and regulatory engagement as appropriate. The inundation mapping and monitoring-point outputs should also be reviewed with the City to confirm whether additional locations of interest should be added before the final analysis is completed.

1 Introduction

1.1 Long Term Water Project

The City of Iqaluit (City) is the newest Capital City in Canada and is rapidly developing into a regional center for the Territory with many northern businesses in Nunavut making it their base of operations. This has led to a rapid growth in population (3% - 4% growth rate expected annually). As a result, the Lake Geraldine raw water reservoir is no longer sufficient to supply or store the required amount of water needed to support potable water needs for the projected growth rate of the City. As such, the City of Iqaluit has retained Arcadis as part of their Long Term Water Project (LTWP) to design a new reservoir for raw water storage, to be constructed using dam/dykes adjacent to the existing Lake Geraldine reservoir.

This design process is currently underway. Arcadis has presented the reservoir and appurtenances design in the *50% Detailed Design Report: Long Term Water Program – Supply and Storage*, provided January 2026 under separate cover. The new reservoir comprises eight embankment structures which, in conjunction with the local topography, will serve to contain 1,646,000m^3 of raw water for City use. This design has not yet been finalized.

1.2 Modelled Scenarios

As the reservoir design has not yet been finalized, dam breach studies and inundation analyses have currently been conducted for theoretical, worst case scenario only. Once the final reservoir sizing and configuration is confirmed, full-scale analyses will be performed. This report is intended to demonstrate a working dam breach and inundation model and provide a basis for the study areas and scope for future analyses. Please note that cascade failure of the Lake Geraldine dam is not discussed in this preliminary report; these analyses will be performed as part of the full suite of scenario models to be included once the project design (and reservoir volumes, etc.) have been finalized.

To that end, the simulation discussed herein is the rainy-day failure of Dam 1, the largest proposed structure in the new reservoir design and the one directly upstream and in closest proximity to the City of Iqaluit. The overall new reservoir hydrodynamic model is discussed in the Hydrology Analysis (supplied as part of the LTWP design documents) and provided here as **Appendix A**, with further input data discussed in **Section 5**. General simulation parameters are discussed below in **Section 4,** and **Section 5** discusses specific scenario models and results. Inundation maps are discussed in **Section 6**, and are provided in full (for the single, Dam 1 rainy-day complete breach scenario) in **Appendix B**.

2 Project Background

The proposed design for the storage portion of the LTWP consists of a reservoir comprised of eight embankment structures; one dam with a service corridor through which water will flow into Lake Geraldine (Dam 1), and seven dykes (Dykes 2-8, with Dyke 7 configured for use as a spillway).

The location of the proposed reservoir is adjacent Lake Geraldine, to the northeast of the City itself, and is shown on **Figure 1** below.

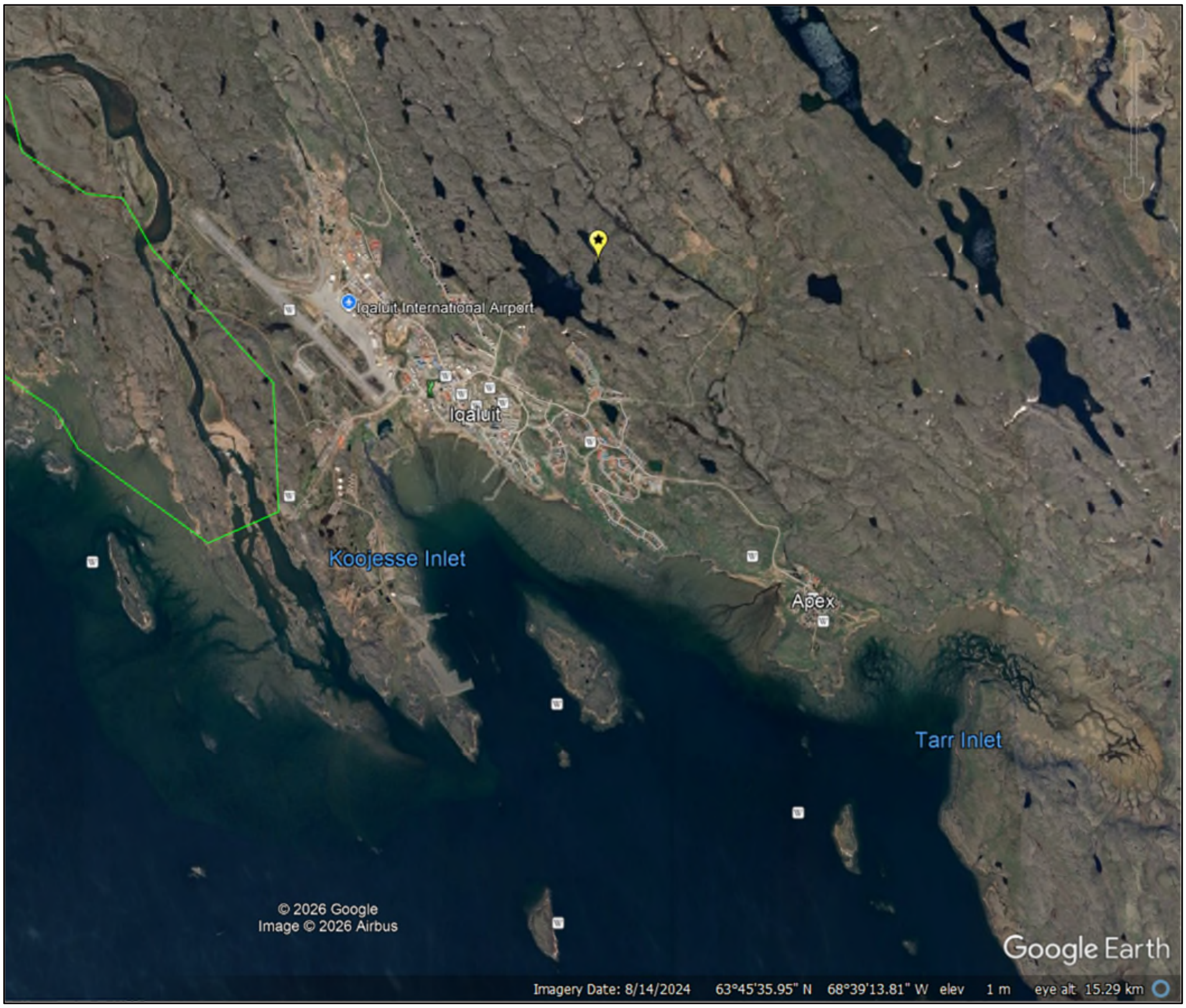

Figure 1: Proposed Reservoir Location

Please note that cascade failure of the Lake Geraldine dam is not discussed in this preliminary report; these analyses will be performed as part of the full suite of scenario models to be included once the project design (and reservoir volumes, etc.) have been finalized. The current reservoir design configuration is shown in **Figure 2** below, with the numbered dam and dyke structures.

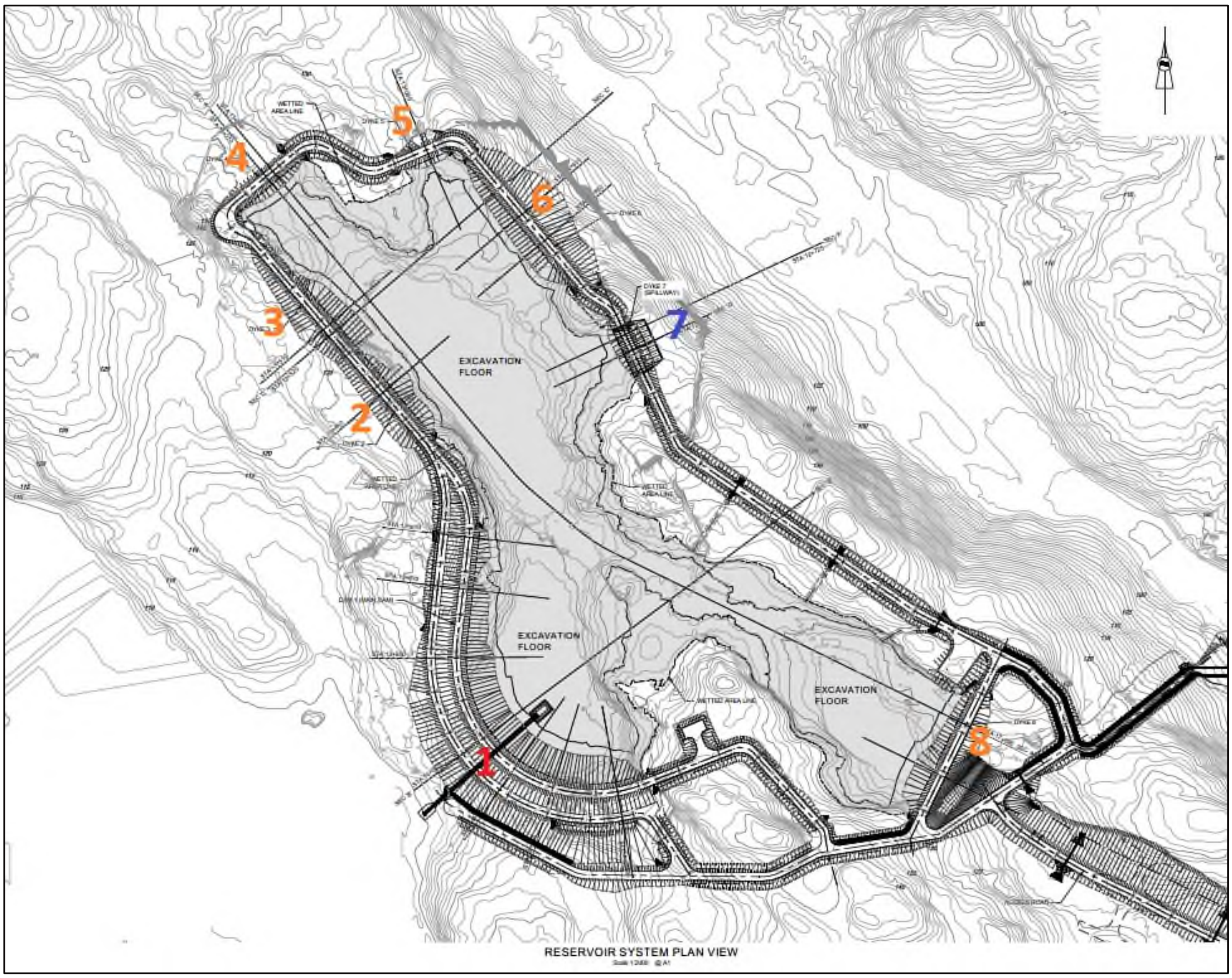

Figure 2: Proposed Reservoir Design Configuration

The reservoir dam/dyke crests uniformly have a design elevation of 131.5masl, as the maximum operating pond level is designed to be 129.5masl (also the elevation of the spillway invert). As noted, this preliminary report discusses exclusively the theoretical worst-case scenario; a rainy-day complete breach of Dam 1. The proposed design for Dam 1 includes a maximum height of 15.5m by 400m long (with varying heights and depths over the span). The Dam Classification for the system focuses predominantly on Dam 1; the largest structure to be built as part of this system with the largest risk for the system itself, downstream infrastructure, and the population at risk

(PAR). This includes critical infrastructure such as the water purification plant, power plant and hospital in addition to residential areas of the City. Based on Table 2-1 of the CDA Dam Safety Guidelines, Dam 1 of the reservoir system would be classified as an 'Extreme' risk. This is based on a) a permanent PAR with the potential loss of life in the event of a catastrophic dam failure estimated at more than 100 lives and b) the probability of extreme losses affecting critical infrastructure or services (e.g., potable water facilities, power plant, hospital).

Water is conveyed through the service corridor beneath Dam 1 to Lake Geraldine and flow is controlled via a series of valves. Dyke 7 acts as a spillway and provides a discharge capacity well in excess of the 17.94m/s required at the full PMF capacity level of 129.825masl. Please see **Appendix A** for greater detail on the reservoir hydrology.

3 Data Review

3.1 Topographic Data and Surfaces

Terrain data used for hydraulic modelling consisted of a high-resolution UAV LiDAR-derived Digital Terrain Model (DTM) acquired Sept 2025, supplemented by the Arctic High Resolution Digital Elevation Model (Arctic HRDEM), derived from ArcticDEM for coverage beyond the limits of the drone survey (Apex, City of Iqaluit). Within the drone/LiDAR survey area, the detailed survey data were used directly. The Arctic DEM-Arctic HRDEM dataset was horizontally and vertically aligned to the UAV LiDAR survey using 37 common points and merged into a continuous terrain surface referenced to NAD83(CSRS) / UTM Zone 19N with elevations in CGVD2013. Other project information, including the reported reservoir and lake water levels, is referenced to CGVD28. Lake Geraldine bathymetric survey data collected by GeoVerra in 2024 were incorporated into the terrain model to accurately represent underwater reservoir morphology. The final terrain model was used as the basis for the dam breach analyses.

The Natural Resources Canada GPS·H vertical datum conversion tool (Natural Resources Canada 2026) was used to evaluate the difference between CGVD28 and CGVD2013 at representative locations within the project area. At a representative location near downtown Iqaluit (63.75°N, 68.52°W), the calculated conversion offset was approximately 0.002 m, with the CGVD2013 elevation being slightly higher than the corresponding CGVD28 elevation. This difference is negligible compared with the 1 m DEM resolution, the computational mesh resolution, and other uncertainties inherent in the dam-breach analysis. It is therefore not expected to materially affect the predicted flood-wave propagation, water depth, velocity, or inundation extent.

For reporting purposes, project elevations and water levels are presented using the source datum associated with each dataset. The local difference between CGVD28 and CGVD2013 is negligible for the scale and accuracy of this analysis and is not expected to materially affect the model results.

The horizontal coordinate system used for model development, mapping, and result presentation is NAD83(CSRS) / UTM Zone 19N. To avoid repetition, the vertical datum is not specified each time an elevation value is presented.

3.2 Probable Maximum Flood (PMF)

Classification as an Extreme Consequence Dam requires that the PMF be used as the Inflow Design Flood (IDF) for this system. The PMF for the new reservoir has been modelled by Arcadis as part of the Hydrology Analysis, presented as part of the 50% Design documents and provided here as **Appendix A**. The calculated inflow and maximum water levels under precipitation events are modelled based on historical data and are listed in **Table 1** below.

Table 1: New Reservoir Maximum Water Levels under Precipitation Events

Parameter	Precipitation Amount (m)	Volume of Rain (m^3)	Maximum Water Level Increase Above 129.5 (m)	Maximum Water Surface Elevation (masl)
100-year 24-hour event	0.054	16,134	0.074	129.574
1000-year 24-hour event	0.075	22,408	0.102	129.602
PMF Event	**0.24**	**71,705**	**0.325**	**129.825**

3.3 Stage/Storage Relationship for the New Reservoir

A stage-storage relationship for the new reservoir was developed based on the existing topography, as proposed excavation for construction is exclusively below the planned pond operating level (i.e., 129.5masl). Please refer to **Appendix A** for greater detail. A graphical depiction is shown in **Figure 3**.

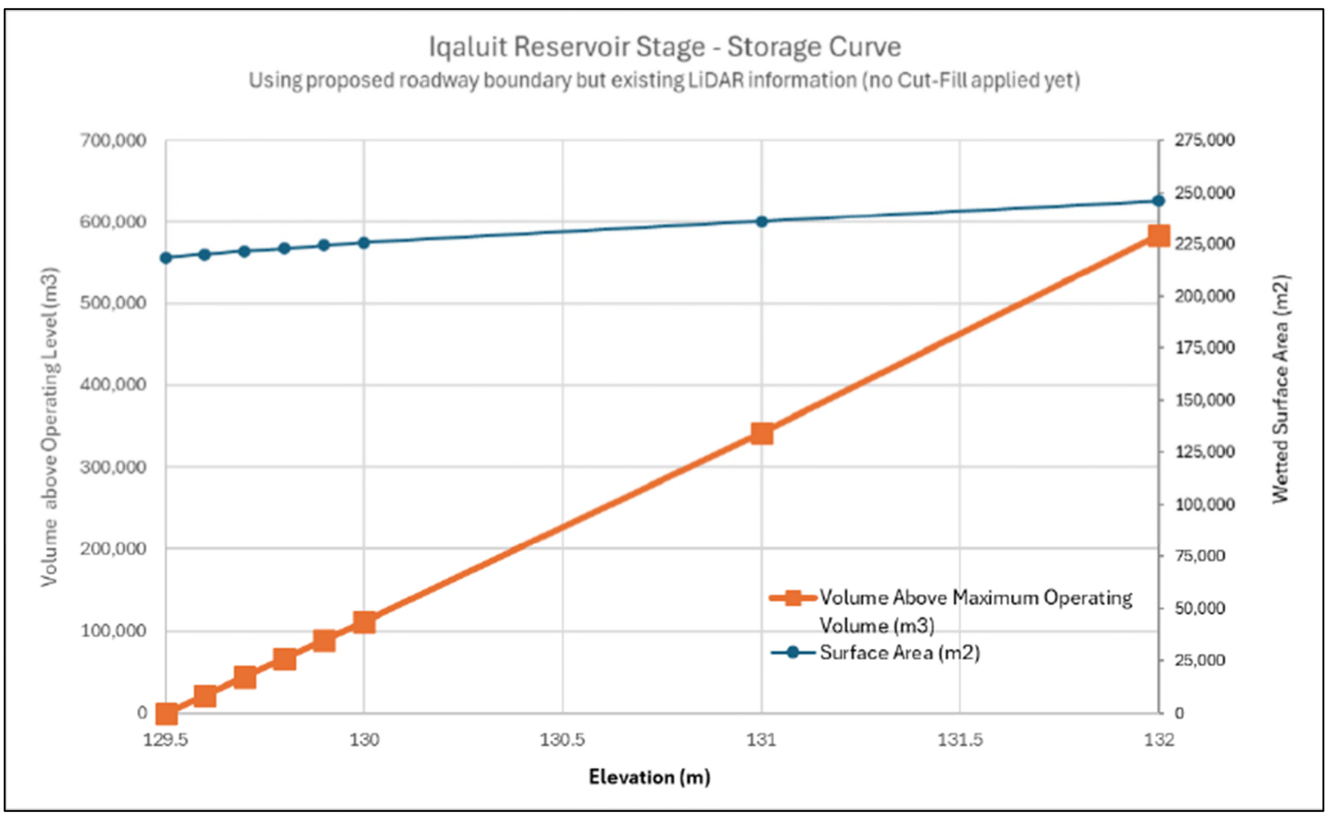

Figure 3: Reservoir Elevation-Storage Relationship Above Max. Operating Level (129.5masl)

4 Dam Breach Simulation Scenarios

As noted previously, this report is intended to provide a theoretical analysis of the presumed worst-case scenario in an effort to inform future, full-scale dam breach modelling and inundation analyses once the reservoir and retention structure designs have been finalized. This section of the report describes the various scenarios that are in the process of being modelled for **all** dam breach scenarios, but the results discussed and the inundations maps included in **Appendix B** are only for the worst-case scenario, i.e., a rainy-day complete failure of Dam 1.

In this study, the dam/dyke breaches are represented by an instantaneous structural failure rather than a gradual breach development process. Specifically, for each failure scenario, one selected major dam/dyke section was assumed to collapse immediately and was removed from the model at the beginning of the simulation. This allowed the impounded water to discharge through the full breach opening without a breach formation delay.

This approach is considered conservative compared with a gradual breach formation assumption. In a gradual breach scenario, the breach opening would enlarge progressively with time, and the outflow would increase gradually as the breach develops. By contrast, the instantaneous removal assumption allows the maximum breach opening to be available at the start of the simulation. As a result, the initial release rate, flood wave generation, flow velocity, and potential downstream inundation depth are expected to be higher than those from a slower breach development process. Therefore, the instantaneous breach assumption provides a conservative basis for evaluating the potential maximum flood impact.

Four structural failure scenarios will be evaluated in the future, full analyses: Dam 1, Dyke 2, Dyke 6, and Dyke 8. As shown in **Figure 2** above, Dam 1 and Dyke 2 are located on the south-facing side of the reservoir. Breach outflow from these two openings would flow directly toward the Iqaluit area. Dyke 6 and Dyke 8 are located on the north-facing side of the reservoir. Breach outflow from these two openings would enter the natural downstream channel (Niaqunguk River), follow the channel toward the Apex area, and eventually discharge into the bay. The plan view of the overall major drainage paths is shown in **Figure 4**.

Figure 4: Initial Model: General Flowpath Schematic

Two hydrologic conditions will be evaluated for each structural failure scenario: a sunny-day condition and a rainy-day condition.

Under the sunny-day condition, the reservoir water level was assumed to be at the normal operating water level of 129.5 m. This scenario represents a dam failure under normal reservoir storage conditions, without an extreme rainfall event or flood inflow. Under the rainy-day condition, the reservoir water level was assumed to be 129.825m, corresponding to the Probable Maximum Flood (PMF) condition described in the project hydrology review (Arcadis Canada Inc. 2026).

Since no hydrology analysis is currently available for Lake Geraldine, the Lake Geraldine water level was assumed to be 111.33 m for the sunny-day condition, corresponding to the spillway invert elevation. For the rainy-day condition, the Lake Geraldine water level was assumed to be 111.9 m, which is approximately the elevation of the dam structure. Based on communication with the local dam operator, the spillway would be operated during high-flow conditions to control the lake level near this elevation. Therefore, the rainy-day scenario represents a conservative high-water-level condition for Lake Geraldine during an extreme rainfall or flood event.

The simulated model cases to be performed as part of the future full analyses are summarized in **Table 2**. The results for Case 2 are presented in the following sections, and in the inundation maps provided as **Appendix B**.

Table 2: Dam Breach Simulation Scenarios (full results to be included in future report)

	Dyke Removal	Reservoir Elevation (m)	Lake Geraldine Elevation (m)	Hydrologic Condition
Case 1	Dam 1	129.5	111.33	sunny-day
Case 2	Dam 1	129.825	111.9	rainy-day
Case 3	Dyke 2	129.5	111.33	sunny-day
Case 4	Dyke 2	129.825	111.9	rainy-day
Case 5	Dyke 6	129.5	111.33	sunny-day
Case 6	Dyke 6	129.825	111.9	rainy-day
Case 7	Dyke 8	129.5	111.33	sunny-day
Case 8	Dyke 8	129.825	111.9	rainy-day

Bottom roughness is an important parameter affecting flood-wave propagation, flow velocity, and inundation depth. In this study, separate roughness-related parameters were assigned to the shallow-water and three-dimensional regions of the model. A bottom drag coefficient of 0.006 was used in the shallow-water region, which is approximately equivalent to a Manning's (n) of 0.025–0.028 for representative flow depths of 1–2 m. A roughness height of 0.02 m was used in the 3D region and was interpreted as a Nikuradse equivalent sand roughness, (k_s). These values were selected based on the expected flow paths in this study. After water is released from the reservoir, the flood wave is expected to quickly enter either the populated area or the natural downstream channel. Therefore, the flow is not expected to travel over a large distance as shallow overland sheet flow through heavily vegetated or highly irregular floodplain areas. Instead, the main flow paths are expected to include urban streets, road corridors, drainage paths, gravel/rocky surfaces, and natural channel sections. The shallow-water drag coefficient was selected with reference to standard Manning's roughness guidance for low-to-moderate roughness channels and floodplain surfaces, while the 3D roughness height was selected as an equivalent bed roughness consistent with the Nikuradse roughness concept (Chow, 1959; Nikuradse, 1933).

5 Dam Breach Flooding Simulation

5.1 Hydrodynamic Model Setup

The dam breach flood wave was simulated using FLOW-3D v23.2. FLOW-3D solves transient free-surface flow using a three-dimensional computational fluid dynamics approach and is suitable for simulating rapidly varied flow conditions associated with dam-break flooding. Three-dimensional numerical models have been increasingly used in dam-break studies, particularly where local topography, hydraulic structures, buildings, flow separation, and non-hydrostatic effects may be important. Recent reviews and application studies have shown that 3D hydrodynamic models can provide detailed representation of dam-break flood propagation over complex real-world terrain. (Maranzoni and Tomirotti, 2023; Muñoz et al., 2020)

The model domain included the proposed reservoir, dam and dyke structures, surrounding terrain, and nearby populated areas. The terrain was represented using a 1 m resolution DEM as discussed in **Section 3** above. The Lake Geraldine bathymetry (GeoVerra 2024) was incorporated into the terrain model to represent the reservoir storage and lake-bottom geometry. Water in the reservoir and Lake Geraldine was initialized at the assumed water levels for each scenario, and the breach flow was allowed to develop naturally under gravity after the selected dyke section was removed from the model.

The breach scenarios modelled herein assume instantaneous removal of selected dam or dyke sections, which is generally conservative for estimating rapid release conditions but does not represent a time-dependent physical breach formation process. Actual dam-breach behaviour would depend on the future final design, materials, foundation conditions, reservoir operations, initiating failure mechanism, erosion resistance, local hydraulic response, and other factors that cannot be fully characterized at this stage.

A combined 3D and shallow-water model was used. The 3D domain was applied around the dam and dyke structures, where flow contraction, vertical acceleration, complex geometry, and rapidly varied flow are important. The downstream area was represented using a shallow-water domain to reduce computational cost where the flow can be reasonably approximated as depth-averaged. This hybrid approach allows detailed 3D flow simulation near the breach while maintaining computational efficiency for downstream flood-wave propagation. FLOW-3D's hybrid shallow-water/3D model is specifically designed for cases where detailed 3D flow around structures is coupled with larger shallow-water domains, such as dams, reservoirs, and hydraulic structures. (Flow Science, 2026).

5.2 Computational Mesh

A mesh sensitivity test was performed to determine an appropriate mesh resolution for the final model. The test results showed that when the mesh size was refined to approximately 3 m or smaller in the 3D region, the main results, including flood-wave speed, arrival time, water depth, and velocity, became less sensitive to further refinement.

For the final model, the shallow-water region used a 1 m horizontal resolution to match the DEM resolution. In the 3D region, the horizontal mesh size was set to approximately 3 m. Vertically, the cells near the water surface were refined to approximately 1 m to better represent the free surface and breach hydraulics. This mesh configuration provides a balance between model accuracy, terrain representation, numerical stability, and computational efficiency.

The total number of fluid cells varied between model cases depending on the downstream flow path and model domain extent. For Cases 1 to 4, the model included approximately 6 million active fluid cells. For Cases 5 to 8, the model included approximately 11 million active fluid cells.

5.3 Boundary Conditions

The lateral boundaries of the model domain were set to allow flow to freely enter or leave the model where appropriate, reducing artificial reflection of the flood wave at the model edges. The top boundary was assigned atmospheric pressure to allow natural free-surface movement. The bottom boundary was treated as a wall boundary with the specified roughness parameters, representing the solid ground surface and terrain.

Buildings and other surface structures were included through the DEM, and their representation is therefore limited by the DEM resolution. As a result, small structures, narrow flow paths, and localized obstructions may not be fully resolved in the model.

5.4 Model Results

The total simulation period was set to 2000 seconds (approximately 30 mins), which was sufficient for the flood wave to travel through the downstream area, discharge into the bay, and reach the maximum inundation extent within the model domain. Spatial model results were saved every 60 seconds and were used to calculate the maximum flood extent, maximum water depth, and maximum velocity throughout the model domain.

Model results were also extracted at selected point locations representing important infrastructure and populated areas in Iqaluit, including the hospital, schools, government buildings, and other critical facilities. A summary of the selected monitoring locations is provided in Error! Reference source not found.. At each monitoring point, results from the 3 × 3 mesh cells surrounding the point location were extracted and averaged to represent the hydraulic conditions at that location. This approach reduces the influence of local cell-scale fluctuations and provides a more representative estimate of water depth and velocity at each monitoring point.

In addition, three cross-sections were placed in the natural channel northeast of the reservoir to record flow rate and velocity through the channel. For each cross-section, additional monitoring points were selected near the associated hydraulic control location, such as a road crossing or shallow river section, to estimate the representative flow elevation.

The locations of the selected monitoring points in the City centre are shown on below, and the monitoring cross-sections in the Niaqunguk River Valley are shown on .

Table 3: Monitoring Points in Iqaluit

Point ID	Easting (m)	Northing (m)	Description
P1	524272.7	7069900	Geraldine Creek downstream from Geraldine Lake Dam
P2	524375.5	7069801	Access road to the Iqaluit Power Plant
P3	524128.7	7069741	Geraldine Creek at Saputi Road
P4	524121.9	7069471	Geraldine Creek near Qikiqtani General Hospital
P5	524245.9	7069415	Access road between Iqaluit Power Plant and Qikiqtani General Hospital
P6	524056.3	7069260	Niaqunngusiariaq Road between Qikiqtani General Hospital and Inuksuk High School
P7	524118.8	7069189	Geraldine Creek near Qikiqtani General Hospital
P8	523994.3	7069286	Access road to Inuksuk High School
P9	523878.6	7069322	Access road to Nunavut Arctic College
P10	524299.6	7068976	Lower access-road drainage crossing near Astro Hill Terrace
P11	523782.4	7069265	Access road to Iqaluit Aquatic Centre from Niaqunngusiariaq Road
P12	523564	7069484	West-side drainage confluence near the Royal Canadian Mounted Police building
P13	523595	7069188	Niaqunngusiariaq Road and Federal Road crossing
P14	523746.5	7069129	Mattaaq Crescent near Iqaluit Stone Sculpture Park
P15	524270.7	7068698	Queen Elizabeth II Way road culvert near Kuugaaq Street
P16	523092.7	7069338	Access road to Iqaluit International Airport at Mivvik Street
P17	523110.3	7069059	Access road to Iqaluit International Airport at Allannguaq Street
P18	524192.4	7068505	Geraldine Creek at Nipisa Street
P19	524120.1	7068421	Geraldine Creek at Sinaa Street
P20	523801.8	7068834	Access road to NorthMart
P21	523595.9	7068863	Natsiq Street and Ben Ell Drive crossing

Figures showing the point locations follow.

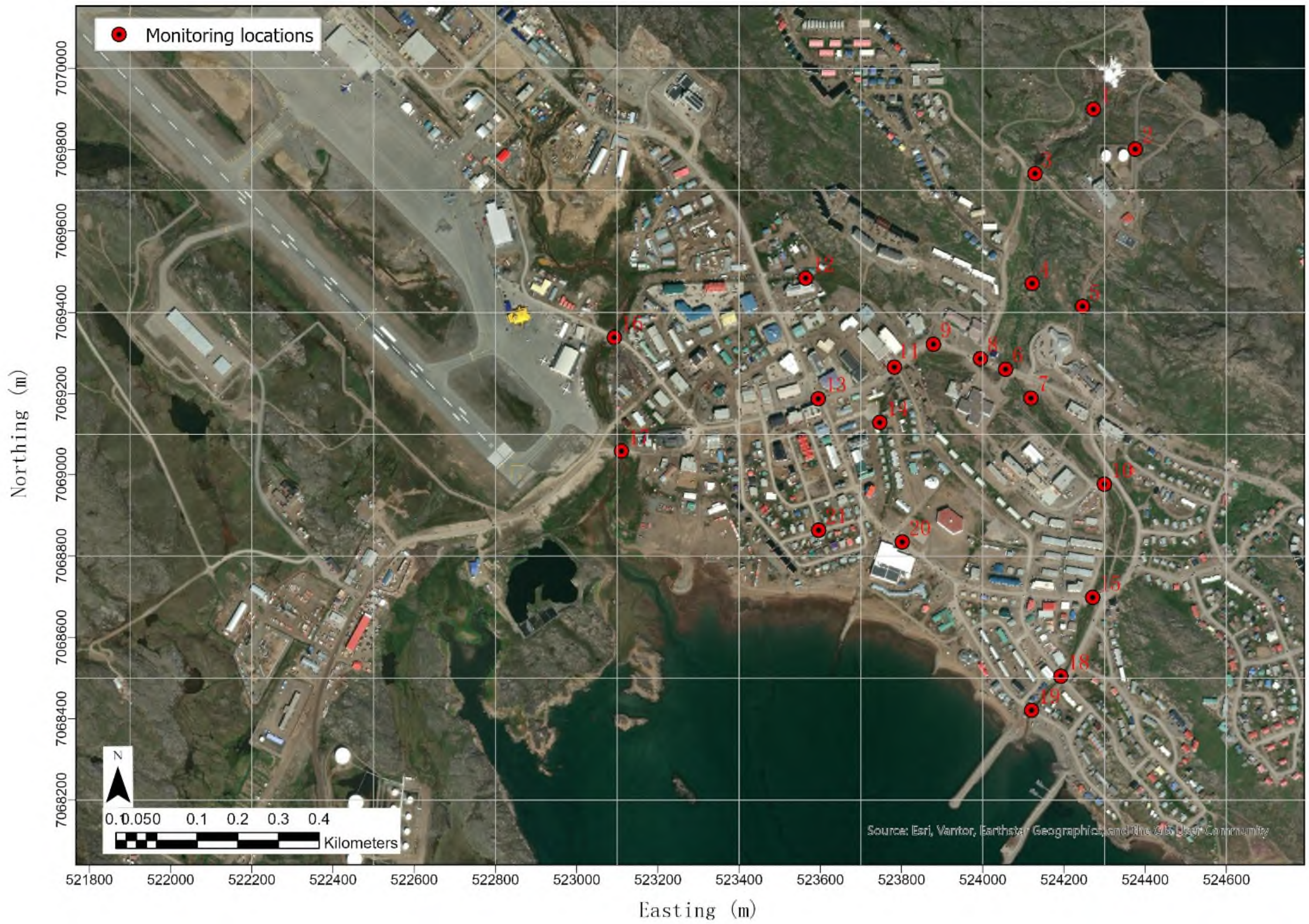

Figure 5: Monitoring Points in Iqaluit (Iqaluit Flowpath)

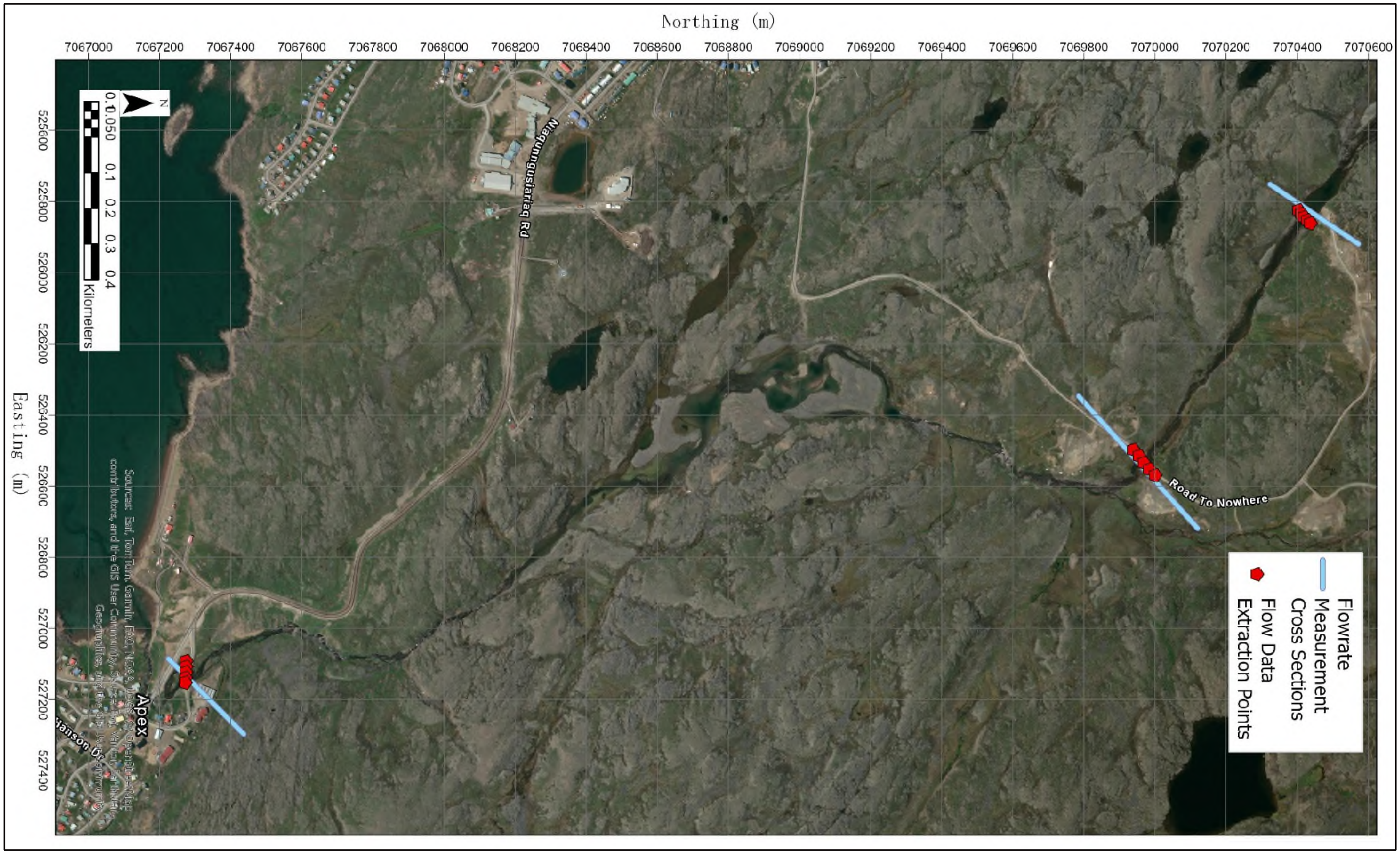

Figure 6: Monitoring Cross-Sections in the Niaqunguk River Valley (Niaqunguk Flowpath)

6 Inundation Maps

As noted above, the inundation maps included here are those for the Case 2 scenario as presented in **Table 2** above. Full analyses, including potential cascade effects on the existing Lake Geraldine Dam, will be performed once the reservoir and associated structure footprint and design has been finalized.

The maximum water surface elevations obtained from the hydrodynamic dam break simulations were incorporated into a GIS model. Using these results, the maximum inundation limits were calculated and used to develop inundation maps of the flooded areas, showing the extent of inundation downstream of the dam sites. Inundation maps are provided in **Appendix B**.

The inundation maps include the following information at several key locations:

- Distance from Dam (km)
- Peak Flood Level (m)
- Peak Flow Depth (m)
- Time to Peak Flood Level (min)
- Peak Velocity (m/s)
- Time to Peak Velocity (min)
- Peak Flow (m^3/s)
- Time to Peak Flow (min)
- Wave Arrival Time (min)

Note that for the breach of Dam 1, which would flood the city, the distance from the dam is measured in linear kilometers from the dam on Geraldine Lake. For Dyke 8, the distance from the dam is measured in kilometers from Dyke 8 following along the Niaqunguk River.

7 References

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8 Statement of Limitations and Conditions

This Dam Breach and Inundation Analysis has been prepared as a preliminary and partial version of the complete report that is expected to be issued at a later stage of the design process. The analysis, findings, figures, tables, inundation limits, hydraulic parameters, and related interpretations presented herein are intended to support ongoing design review, planning, and discussion only. They should not be considered final, issued-for-construction, or suitable as the sole basis for emergency planning, regulatory decisions, detailed infrastructure design, land-use decisions, or public communication without further review and confirmation. The report is based on the information available at the time of preparation and will be subject to revision as the project design advances and additional project information becomes available.

The dam and reservoir referenced in this report are currently at the 50% design stage. At this stage, the general arrangement, structure locations, reservoir geometry, crest elevations, operating levels, spillway configuration, and related design assumptions have been developed sufficiently for preliminary assessment; however, they remain subject to refinement through subsequent design phases, including 90%, 100%, and issued-for-construction submissions. Changes to the reservoir footprint, dam and dyke alignments, embankment sections, foundation treatment, liner systems, spillway details, water-transfer infrastructure, cut/fill geometry, or operating criteria could materially affect the breach openings, available storage, outflow hydrographs, flow paths, inundation depths, velocities, arrival times, and hazard characterization presented in this report. Accordingly, the results should be interpreted as preliminary estimates associated with the 50% design configuration and should be reviewed and updated if the design assumptions are materially revised.

The hydrodynamic modelling undertaken for this study necessarily includes simplifications and assumptions that reflect the preliminary nature of the design and the available supporting data. The modelling results depend on the adopted breach scenarios, reservoir and Lake Geraldine water levels, terrain and bathymetric surfaces, computational mesh resolution, roughness parameters, boundary conditions, treatment of buildings and surface obstructions, and the selected simulation duration and output intervals. The breach scenarios modelled herein assume instantaneous removal of selected dam or dyke sections, which is generally conservative for estimating rapid release conditions but does not represent a time-dependent physical breach formation process. Actual dam-breach behaviour would depend on the future final design, materials, foundation conditions, reservoir operations, initiating failure mechanism, erosion resistance, local hydraulic response, and other factors that cannot be fully characterized at this stage.

The model results are also limited by the accuracy, resolution, and completeness of the underlying spatial and hydrologic data. The terrain model incorporates available topographic, LiDAR, ArcticDEM-derived, and bathymetric information, including project survey and Lake Geraldine bathymetry, but localized grades, drainage features, culverts, road embankments, building thresholds, utility corridors, small structures, debris, snow and ice conditions, and future site changes may not be fully represented. The use of a one-metre terrain surface and a combined three-dimensional and shallow-water modelling approach provides an appropriate basis for preliminary assessment of flood-wave propagation at the scale of this study, but localized water depths, velocities, arrival times, and inundation extents may vary from the mapped or tabulated results, particularly near steep terrain, narrow flow paths, road crossings, structures, hydraulic controls, and the margins of predicted inundation.

The study has relied on other available Iqaluit Long Term Water Program documents and associated project information, including the Hydrology Review technical memorandum, the 50% design drawings and design-development information for the proposed reservoir, the topographic and bathymetric survey information, the geotechnical work planning and investigation information, and relevant background information concerning Lake

Geraldine and the proposed new reservoir. The preliminary breach and inundation assessment should be read in conjunction with those project documents and should not be interpreted as superseding discipline-specific design reports, geotechnical recommendations, hydrologic design criteria, dam safety evaluations, emergency preparedness documentation, regulatory submissions, or final design documentation. Where assumptions in this report differ from or are updated by later project documents, the later approved project information should take precedence.

The inundation maps and extracted hydraulic parameters are intended to identify potential flood-affected areas and relative hydraulic conditions under the modelled scenarios. They are not floodplain regulation maps, cadastral boundary determinations, property-specific risk statements, or operational emergency response maps. The mapped inundation limits should be interpreted with appropriate professional judgement and with recognition that actual flood extents may be influenced by conditions not represented in the model, including extreme meteorological conditions, antecedent water levels, snowpack and ice effects, debris blockage or mobilization, temporary construction works, future development, changes in surface drainage, changes to Lake Geraldine operations, and concurrent failures or operational responses. The absence of mapped inundation at a given location should not be interpreted as confirmation that flooding, erosion, access disruption, or other adverse effects could not occur under different assumptions or future conditions.

This report does not provide an independent final dam safety assessment, a final consequence classification, a final emergency preparedness plan, a final operations plan, or a definitive assessment of life-safety, environmental, cultural, economic, or infrastructure consequences. The results may inform such evaluations, but those evaluations require integration with the final design, dam safety documentation, owner and regulator requirements, emergency management considerations, operations and maintenance procedures, and stakeholder review. Any use of the results for emergency preparedness, public-warning planning, infrastructure protection, or regulatory submission should be subject to further review by appropriately qualified professionals and by the City of Iqaluit and its representatives.

Arcadis has exercised the standard of care ordinarily exercised by qualified professionals undertaking similar work under similar circumstances, using information available at the time this preliminary report was prepared. No warranty, expressed or implied, is made regarding the accuracy or completeness of information supplied by others, the final configuration of the project, or the ultimate performance of the future dam and reservoir system. Arcadis should be notified of any material changes to the design, project data, operating assumptions, or regulatory requirements so that the need for revisions to this preliminary analysis can be assessed. Until such updates are completed, the findings and conclusions presented herein should be considered conditional on the assumptions and limitations described in this section.

Appendix A

A – Hydrology Analysis

TM-Hydrology Analysis

Project:
Long Term Water Program –
Raw Water Supply and
Storage, Nunavut, the City of Iqaluit

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Technical Memorandum: Hydrology Review

ISSUED TO
Design Engineering

DATE
January 8, 2026

OUR REF
30192375

ORIGINAL ISSUED BY
Vali Ghorbanian – Arcadis Canada

UPDATE ISSUED BY
Jonathan Edwards, P.Eng. – Arcadis Canada

ATTACHMENTS:
Attachment A – Modelling Support Information

Introduction

Iqaluit, situated at the southern tip of Baffin Island in Frobisher Bay (63°45'N latitude and 68°31'W longitude), serves as the capital of the Nunavut Territory. Iqaluit has undergone rapid development and expansion. It serves as the administrative hub for the Nunavut Territory and hosts numerous federal and territorial government departments. Additionally, Iqaluit is evolving into a regional center for the territory, attracting various northern businesses and Inuit organizations that choose it as their operational headquarters. A new water reservoir is being constructed in the northeast of the City of Iqaluit, east of Lake Geraldine. This technical memorandum outlines the standards and calculations regarding hydrology for the determination of potential spillway flows, and proposed height of the planned new reservoir.

Meteorology

The weather stations around the project area were reviewed to find out the available historical data for hydrology analysis. **Figure 1** shows the location of the stations respect to the study area. Available data from meteorological stations was collected and summarized in order to select the most appropriate data and is presented in **Table 1** below. Based on the review, it was concluded that the Iqaluit station is closest to the new reservoir location and in addition has the longest period (1946-1996) of continuous daily (24-hour) rainfall, snowfall, and total precipitation time series. Thus, this station was considered for analysis. The climate at the site is northern with the average temperature -8.97 Cº and average precipitation as 1124 mm.

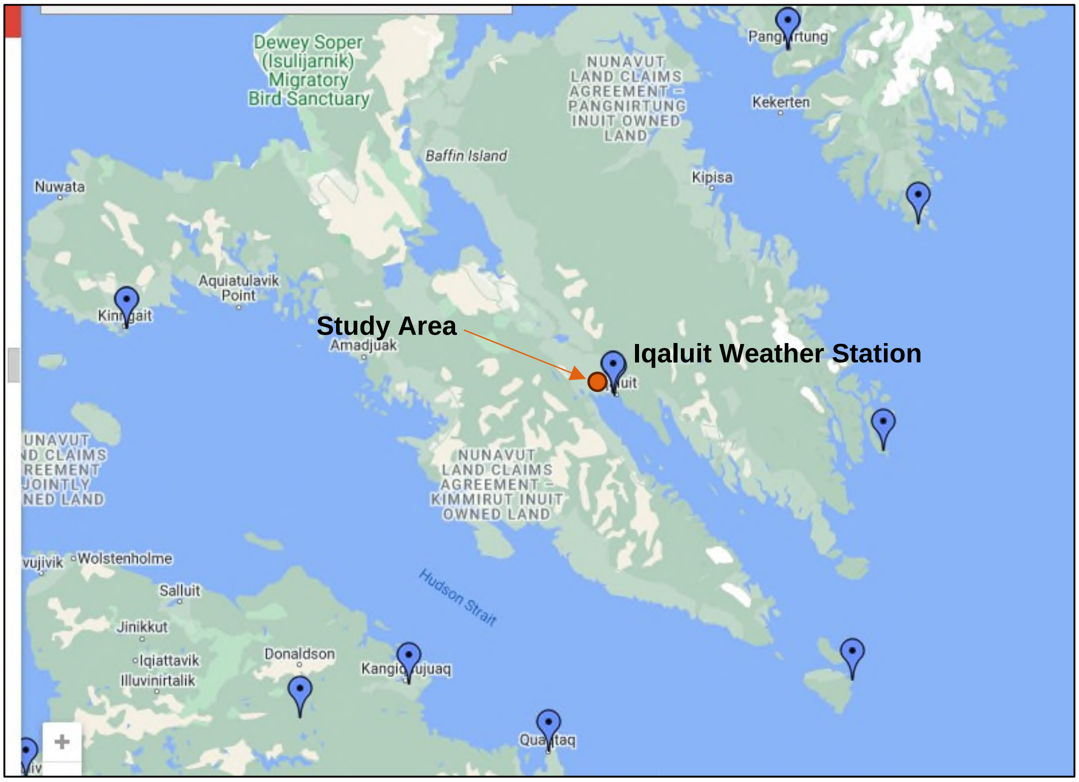


Figure 1. Site and Weather Stations Location

Table 1. Weather Stations

Station Name	Max Temp	Min Temp	Average Temp	Total Precip.	Snow on Ground	Wind Speed	Wind Direction
Iqaluit A (#1)	Jan 1946 - July 2008	Jan 1946 - July 2008	Jan 1946 - July 2008	Jan 1946 - Sept 2007	Jan 1955 - Sept 2007	Jan 1946 - July 2008	Jan 1946 - July 2008
Iqaluit A (#2)	Nov 2018 - Oct 2023	Nov 2018 - Oct 2023	Nov 2018 - Oct 2023	None	None	Dec 2020 - Oct 2023	Dec 2020 - Oct 2023
Iqaluit AWOS	July 2008 - June 2015	July 2008 - June 2015	July 2008 - June 2015	None	None	July 2008 - June 2015	July 2008 - June 2015
Iqaluit Climate	Dec 2004 - Oct 2023	Dec 2004 - Oct 2023	Dec 2004 - Oct 2023	March 2005 - Oct 2023	May 2004 - June 2023	May 2004 - Oct 2023	Dec 2004 - Oct 2023
Iqaluit UA	April 1997 - April 2007	April 1997 - April 2007	April 1997 - April 2007	April 1997 - Feb 2016	Sept 1997 - Feb 2016	None	None
Brevoort Island	Nov 1959 - May 2023	Nov 1959 - May 2023	Nov 1959 - May 2023	Nov 1959 - Sept 2007	May 1960 - Sept 2006	Oct 2007 - Sept 2023	Oct 2007 - Sept 2023
Fort Resolution	Aug 1911 - Jan 1936	Aug 1911 - Jan 1936	Aug 1911 - Jan 1936	Aug 1911 - Jan 1936	None	None	None
Fort Resolution A (#1)	Sept 1930 - Nov 2014	Sept 1930 - Nov 2014	Sept 1930 - Nov 2014	Sept 1930 - Nov 2014	Nov 1980 - Nov 2014	None	None
Fort Resolution A (#2)	Oct 2018 - Oct 2023	Oct 2018 - Oct 2023	Oct 2018 - Oct 2023	None	None	None	None
Resolution Island (#1)	Oct 1929 - Oct 1961	Oct 1929 - Oct 1961	Oct 1929 - Oct 1961	Oct 1929 - Oct 1961	Nov 1954 - Oct 1961	None	None
Resolution Island (#2)	March 1962 - July 2018, Feb 2021 - Feb 2022	March 1962 - July 2018, Feb 2021 - Feb 2022	March 1962 - July 2018, Feb 2021 - Feb 2022	March 1962 - Aug 1975	March 1962 - Aug 1975	July 1996 - May 2014, Feb 2021 - Nov 2022	July 1996 - May 2014, Feb 2021 - Nov 2022

Inflow Design Flood (IDF)

According to the Canadian design standard guideline (DSG)[1], dams shall be designed or evaluated for a maximum flood termed the Inflow Design Flood (IDF). The IDF is selected on the basis of the potential consequences of failure and the probable maximum flood (PMF). The PMF is defined as the most severe flood that may reasonably be expected to occur at a particular location.

The inflow design flood (IDF) selected for the New Reservoir freeboard calculations is based on generalized hazards and responses for embankment dams defined by the Canadian Dam Association (CDA) shown in **Table 2** and specific hazards shown in **Table 3**.

The 100- and 1000-year recurrence interval flood volumes were estimated using Annual Exceedance Probability (AEP) estimates derived from total precipitation (rainfall plus snowfall water equivalent) time series for the Environment Canada Iqaluit Station. Computed Log-Pearson III AEPs and Weibull plotting positions for total precipitation at the Iqaluit station are shown in **Figure 2**.

Table 2: Inflow Design Flood (IDF) and consequence classes (from CDA Technical Bulletin 6)

Consequence Class	IDF
Low	1/100-year
Significant	Between 1/100 and 1/1000-year (Note 1)
High	1/3 between 1/1000-year and PMF (Note 2)
Very High	2/3 between 1/1000-year and PMF Note 2)
Extreme	PMF
Note 1. Selected on basis of incremental flood analysis, exposure and consequence of failure. **Note 2.** Extrapolation of flood statistics beyond 1/1000-year flood (10^{-3} AEP) is generally discouraged. The PMF has no associated AEP. The flood defined as "1/3 between 1/1000-year and PMF" or "2/3 between 1/1000 year and PMF" has no defined AEP.	

[1] Canadian Dam Association (2007). Technical Bulletin 6, Hydrotechnical Considerations for Dam Safety.

Table 3: Iqaluit Dam hazard classification and IDF assessment

Consequence Class	Population at Risk	Loss of Life	Envrionmental and cultural Values	Infrastructure and Economics	Design Flood[1]
Extreme	Permanent where residence would be in the inundation zone should there be a catastrophic failure of the dam	While not expected to be of concern there is the potential as a function of when the breach failure were to occur	Under a worst case scenario the flood wave could remove a significant amount of overburden thus damaging the environment likely a permanent manner with the potential for some loss of habitat down stream	A breach would directly impact Lake Geraldine and potentially comprise the retention dam associated with this reservoir. Furthermore the inundation wave could also impact the existing water treatment plant operations.	IDF = PMF
Very High	Permanent where residence would be in the inundation zone should there be a catastrophic failure of the dam	As compared to Dam 1 it is less likely the inundation wave would be such that the flood would damage homes but depending on the cause of the breach ie flood then issues are potnetially compounded by what happens with the Lake Geraldine dam.	Less likely a breach of this dam would cause significant loss or deterioriation of critical fish or wildlife habitat. There could be compensation in kind but likely impractical	A breach would directly impact Lake Geraldine and potentially comprise the retention dam associated with this reservoir. However unlike Dam 1 the height of this dam is minor and as a result the amount of water from the reservoir that would be lost would be significantly less. Furthermore the inundation wave would lose energy in overland flow as compared to Dam 1 which has next to no overland flow on its downstream side.	IDF = interpolation between 1/1000 year flood and PMF used 2/3 between the two model points noted
Very High	Permanent where there is potential damage to the Apex River and flooding could damage key water supply infrastructure crossing the Apex River	No loss of life anticipated from a breach of this dam	A breach of this dam could cause a significant loss or dterioration of critical fish or wildlife habitat where restoration or compensation in kind may be possible but impractical as the inundation wave could permanently alter the course of the Apex River.	A breach of this dam could impact infrastructure crossing the Apex River including the Road to Nowhere and the proposed conveyance pipe crossing between the pump station and reservoir 2. It is not anticipated infrastructure well down stream of the dam would be impacted but it is possible.	IDF = interpolation between 1/1000 year flood and PMF used 2/3 between the two model points noted
Significant	Temporary only because there is no population immediately upstream of the dam and any flood would dissipate in the environment.	Unspecified however none expected unless the City develops the lands upgradient of the dam.	A breach of the dam is not likely to create any significant loss or deterioration of fish or wildlife habitat where any losses would be of marginal habitat.	Low economic losses and area contains limited infrastructure of services.	IDF = between 1/100 yr and 1/1000 yr flood selected on basis of incremental flood analysis, exposure and consequence of failure.

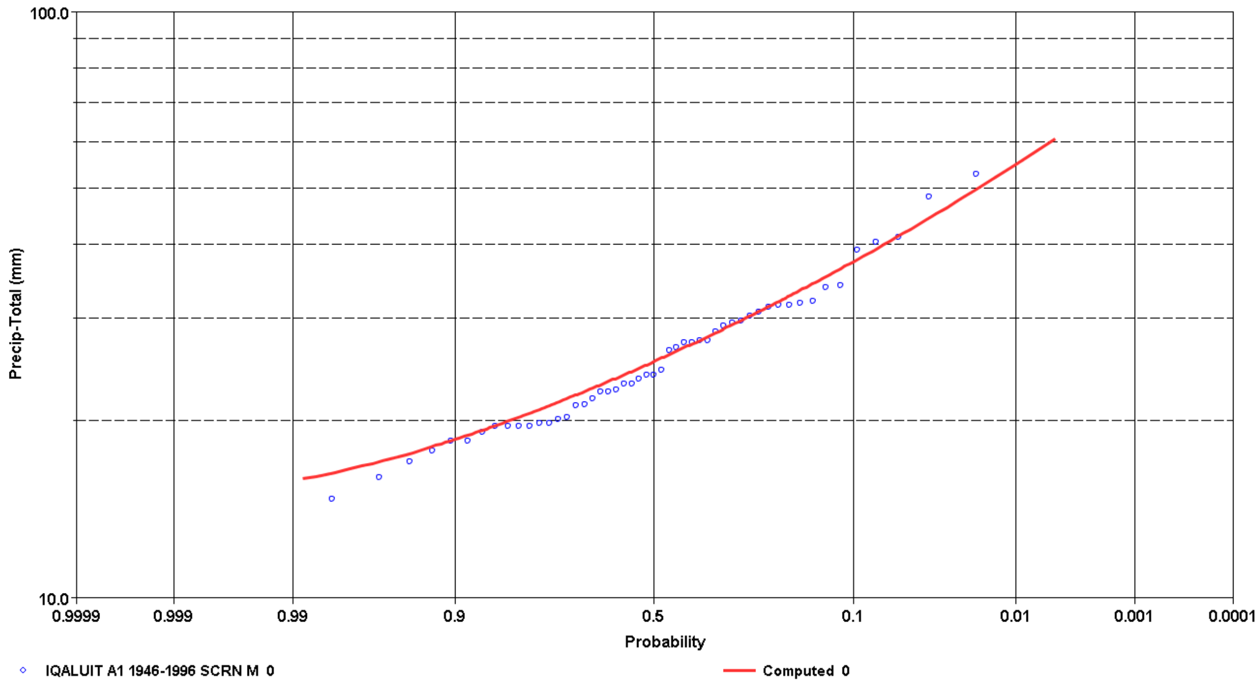


Figure 2: Iqaluit Station total 24-hour precipitation annual exceedance probability (AEP)

Estimated 100-year (0.01 AEP) total daily precipitation is 54 mm, and 1000-year total (0.001 AEP) precipitation was extrapolated to be 75 mm at the station. The 24-hour Probable Maximum Precipitation (PMP) at the site was estimated to be 240 mm, based on supplemental material developed by the American Meteorological Society for comparison of PMP predictions of two Canadian regional climate models[2].

Because the potential hazard classifications of the new reservoir range from Significant to Extreme, the most conservative IDF – the Probable Maximum Flood (PMF) – was assumed as the basis for determination of the maximum water levels and spillway flows.

Catchment Area/Topography

The area around the reservoir is composed mostly of impervious bedrock. The topography in the catchment is moderate to rugged with elevations ranging from 119 meters above sea level (masl) to 144 masl. The available topography (surveyed using LiDAR technology), along with the proposed reservoir boundary and roadway layout was used for the purposes of characterizing catchment topography and delineating the catchment draining to reservoir. The catchment is a confined area with no external surface inflow to the reservoir. The source of water to the new reservoir is precipitation including snow melting. The catchment area is 29.87 hectares and the catchment boundary is shown in **Figure 1.**

[2] M.A. Ben Ayala, F. Zwiers, and X. Zhang (01 Oct. 2019) Evaluation and Comparison of CanRCM4 and CRCM5 to Estimate Probable Maximum Precipitation over North America. *Journal of Hydrometeorology*, Vol. 20, Issue 10. DOI: https://doi.org/10.1175/JHM-D-18-0233.1

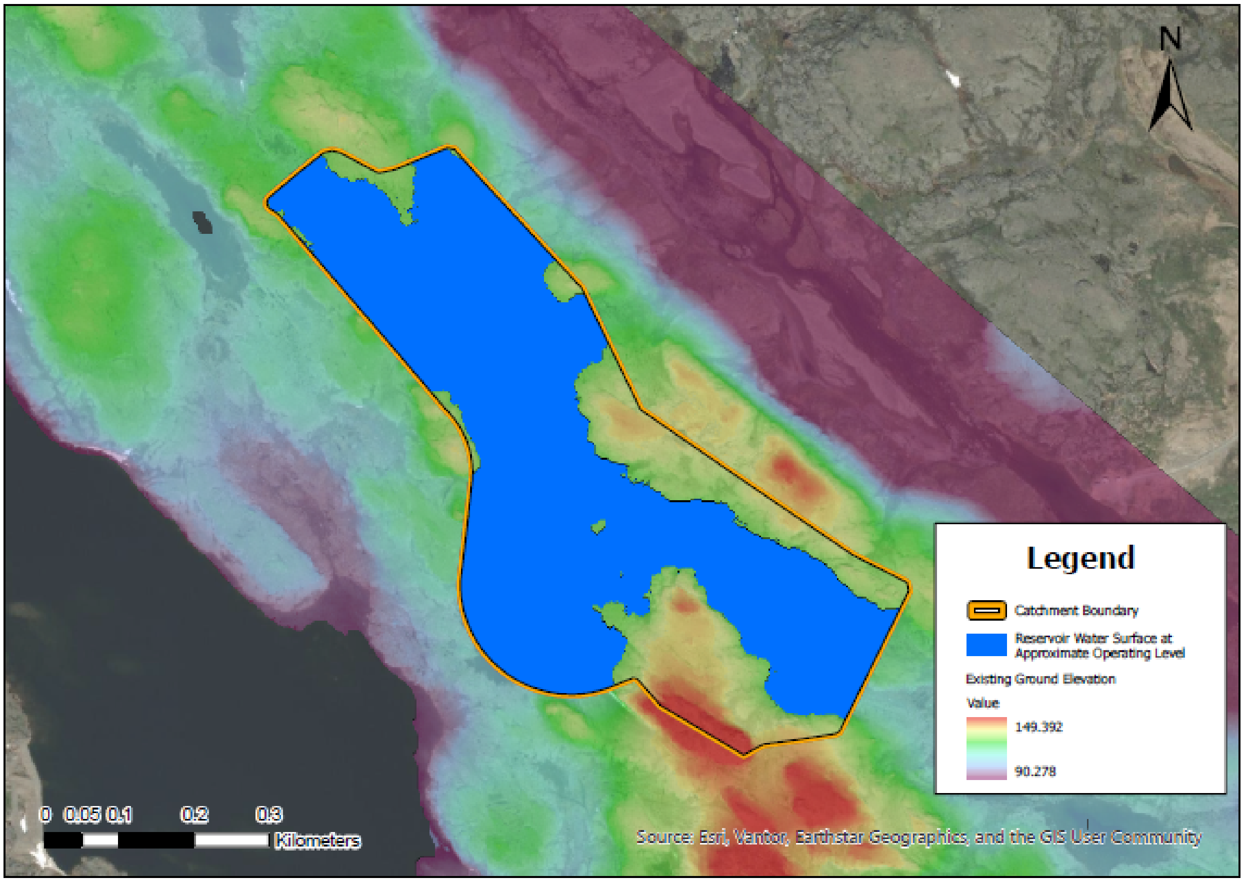


Figure 3. New Reservoir' Catchment Boundary

Reservoir Water Level and Spillway Flows

Inflow to the new reservoir from 100-year, 1000-year, and PMF storms was calculated assuming 100 percent runoff from the reservoir and surrounding drainage area of 0.2987 km2. As shown in **Figure 3**, this assumption is reasonable since the reservoir water surface comprises a large majority of the total catchment area and the remaining surface surrounding the reservoir is steep rocky terrain.

Two potential spillway flows were determined based on the following conditions:

1. **The maximum water level being reached assuming no spill during the event; and,**
2. **24-hour rainfall being routed through a design storm hyetograph (24-hr AES);**

Runoff volumes to the new reservoir were converted to reservoir stage above normal operating level (129.5m) based on the elevation-storage relationship plotted in **Figure 5**. The stage-storage relationship is based on the existing topography, as the majority of the cut-fill will be happening below the operating water level. A maximum water level increase was determined based on the total precipitation runoff volume, assuming no losses from the spillway. Precipitation volumes and the corresponding maximum reservoir water level are listed in **Table 4**, and **Figure 6.**

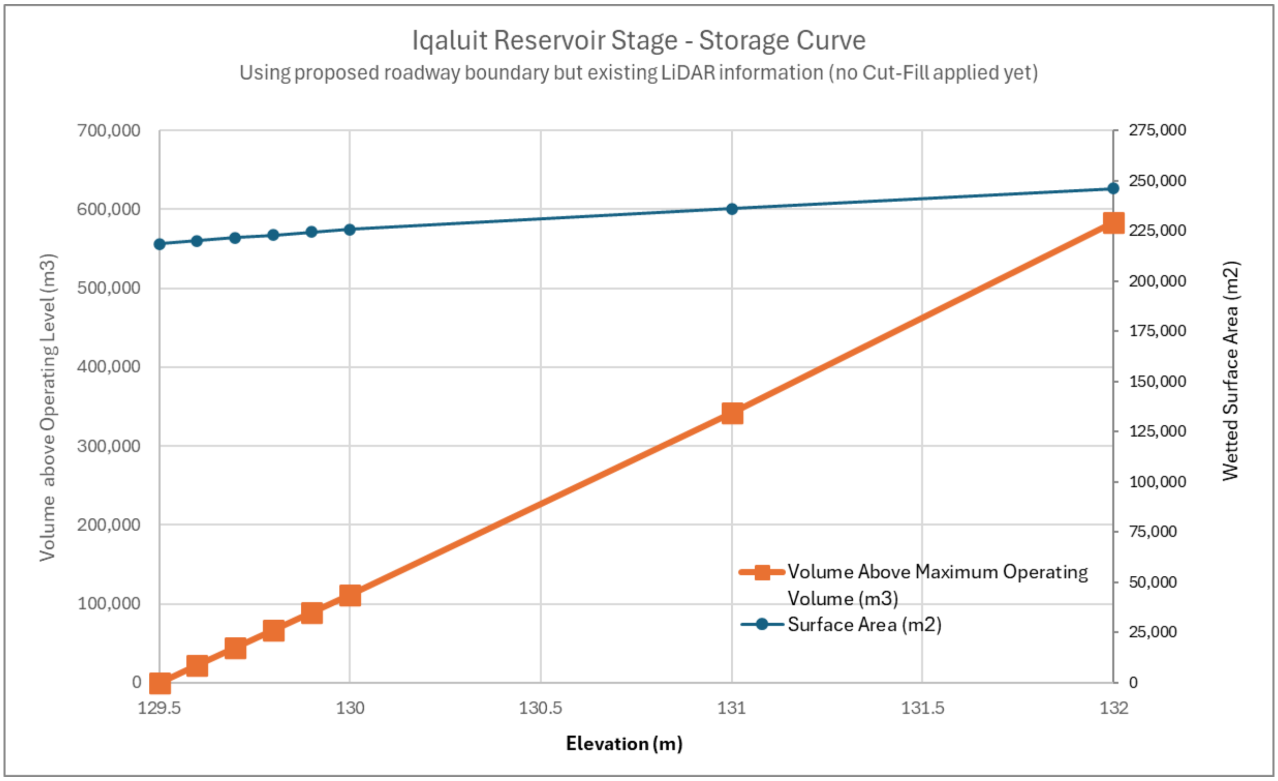


Figure 5. Reservoir elevation-storage relationship above maximum operating level of 129.5m

Table 4: Iqaluit Reservoir Maximum Water Levels under Precipitation Events

Parameter	Precipitation Amount (m)	Volume of Rain (m3)	Maximum Water Level Increase Above 129.5 (m)	Maximum Water Water Surface Elevation (m asl)
100-year 24-hour Event	0.054	16,134	0.074	129.574
1000-year 24-hour Event	0.075	22,408	0.102	129.602
PMF Event	**0.24**	**71,705**	**0.325**	**129.825**

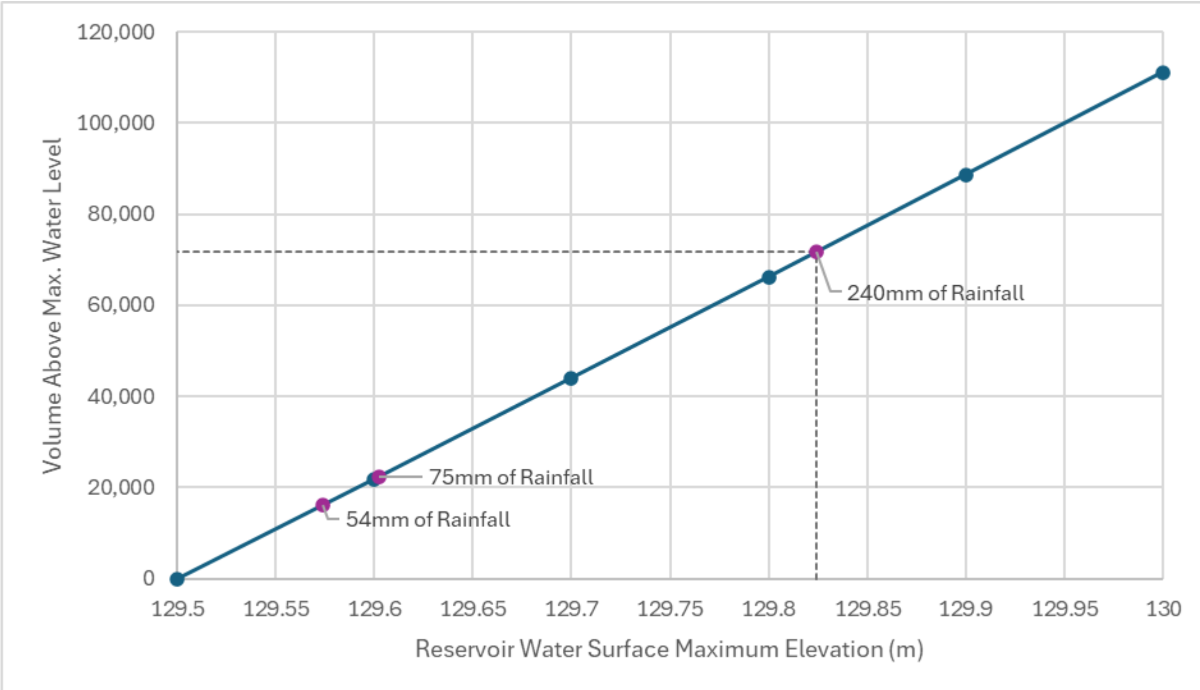


Figure 6. Maximum Water Surface Elevations based on Runoff Volumes for various events

A simple hydraulic model was created using Autodesk's InfoWorks ICM modelling software to evaluate the resulting flows and water depths over the proposed spillway resulting from the two potential conditions listed above. The model setup and components are summarized below:

- The Reservoir was modelled as a storage node, with the stage-storage relationship as shown in **Figure 5**.
- The spillway was modelled as broad crested weir, with an invert elevation matching the maximum operating water level (129.5m), a crest width of 50m and a length of 5m.
- All simulations were for the PMP Event, with a total simulation length of 2 days (48-hours)

For the maximum water level scenario, the following was applied:

- An initial condition water level of 129.825 was applied to the Reservoir Node to reflect the PMP Event maximum water level and corresponding volume.

For the rainfall routing scenario, the following was applied:

- A 24-hour AES shaped design event was developed, with a total depth of 240mm and a maximum precipitation intensity of 55 mm/hr. (Hyetograph included in Attachment A).
- The 29.87 ha catchment area was set up as an impervious surface, with a fixed runoff coefficient of 1, using the Wallingford routing model.

The maximum water level scenario resulted in a maximum depth of flow over the spillway of 0.325m and a peak flow of 17.94 m^3/s at the beginning of the simulation (as the max water level is “released”). The rainfall routing scenario resulted in a maximum flow depth over the spillway of 0.122m, and a peak flow of 3.68 m^3/s. Hydrographs of the resulting depths and flows are provided in **Attachment A**.

Wind setup and wave height

Freeboard is also required above the conservation pool prevent embankment overtopping by wind-generated waves to include wind setup, wave height, and wave runup. Wind setup is the rise in water level caused by wind shear along the effective fetch length, assumed in this case to represent the longest possible horizontal distance in the wind direction. Estimated fetch length for the New Reservoir is 0.99 km in the NNW – SSE direction as shown in **Figure 6**.

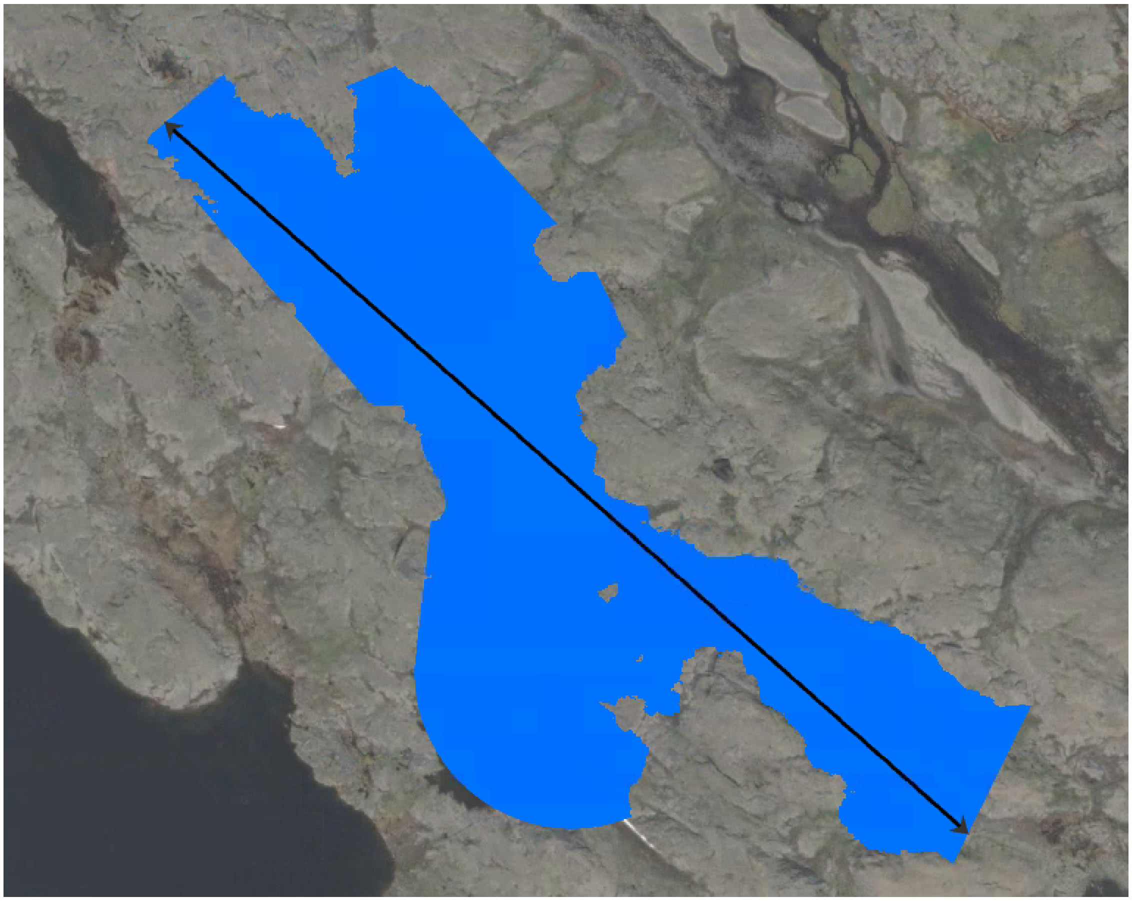


Figure 6: Reservoir Effective Fetch (Black Line)

Two wind speeds were analyzed in wind setup and significant wave height calculations based on information provided in a 2021 dam safety evaluation report for Lake Geraldine Dam[3], located to the immediate west of the New Reservoir. Due to seasonal ice cover, the report recommends wind speeds between 80 and 105 km/hour during the design flood, and between 131 and 216 km/hour during normal operations. For conservatism, the upper limit of each range was applied in this case to determine controlling wind setup and wave runup above the IDF level of 129.825 m (105 km/hour), and above the normal operating level of 129.5 m (216 km/hour).

[3] Concentric (2022). *Lake Geraldine Dam 2021 Dam Safety Review, Iqaluit, Nunavut*. Section 4.5.1 – Hydrotechnical.

Wind setup above stillwater elevation was calculated using the procedure defined in Section 6.3.7 of CDA *Hydrotechnical Considerations for Dam Safety (2007)*, as follows:

$$S = \frac{U^2 F}{Kd}$$

where: S = wind setup (m) above still water level
U = wind speed (m/s)
F = fetch length (km)
d = average reservoir depth (m)
K = constant = 4850 (USBR 1992)

Significant wave height was also calculated assuming fetch-limited conditions using CDA (2007) as follows:

Wave heights and periods may be determined using the following equations, which are valid only for fetch-limited conditions:

$$H_s = 1.616 \times 10^{-2} U_A F^{1/2}$$

$$T = 6.238 \times 10^{-1} (U_A F)^{1/3}$$
$$U_A = 0.71\, U^{1.23}$$

where: H_s = significant wave height (m)
F = fetch length (km)
T = wave period (s)
U = wind speed (m/s)
U_A = wind stress factor

As defined in Section 6 of the CDA Technical Bulletin, wave runup is the vertical height between the maximum elevation attained by wave run-up on a slope and the water elevation at the toe of the slope, excluding wave action. Wave runup on embankments is governed by a variety of factors including slope, the water depth at the toe, the roughness and permeability of the embankment, and incident wave characteristics. Because these parameters are updating at the current stage of design and, without a generic methodology for prediction of wave run-up or relevant empirical data, wave runup in this case was assumed equal to significant wave height Hs above the wind setup elevation.

Uncertainty in estimated wave runup in this case was accommodated by an assumed 0.5 meters added as a contingency for windspeed at flood pool (105 km/hr) and 0.2 meters as a contingency for windspeed at normal pool (216 km/hr). Results of wave runup calculations are shown in **Table 5.**

Table 5. Iqaluit New Reservoir Freeboard Requirements

Stillwater condition	Parameter	Freeboard requirement (m)	Elevation (m)
IDF pool (129.825 m) Wind speed (U) = 105 km/hour	Wind setup	0.01	129.84
	Significant wave height H_s	0.72	130.56
	Wave runup	0.72	130.56
	Contingency	0.50	131.06
Normal pool (129.5 m) Wind speed (U) = 216 km/hour	Wind setup	0.05	129.55
	Significant wave height H_s	1.76	131.31
	Wave runup	1.76	131.31
	Contingency	**0.20**	**131.51**

Summary

Maximum spillway flows were calculated for the Inflow Design Flood (IDF) using InfoWorks ICM modelling software, under two separate conditions: the maximum water level being reached before any spill occurring, and precipitation being routed through a 24-hour AES design storm event with active spill. The flows over the 50m wide berm under each condition were estimated to be 17.94 m3/s and 3.68 m3/s respectively.

The freeboard requirements for the top of the reservoir wall elevations was also calculated based on the Inflow Design Flood (maximum pool level) plus wind setup, wave runup and contingencies. The recommended top of dam elevation is 131 meters, as shown in Table 4.

Inflow design flood calculations and flood-frequency analysis was performed using meteorological time-series data from the Environment Canada Iqaluit station and the USACE HEC-DSSVue (Visual Utility Engine) database software, downloadable from https://www.hec.usace.army.mil/software/hec-dssvue/downloads.aspx.

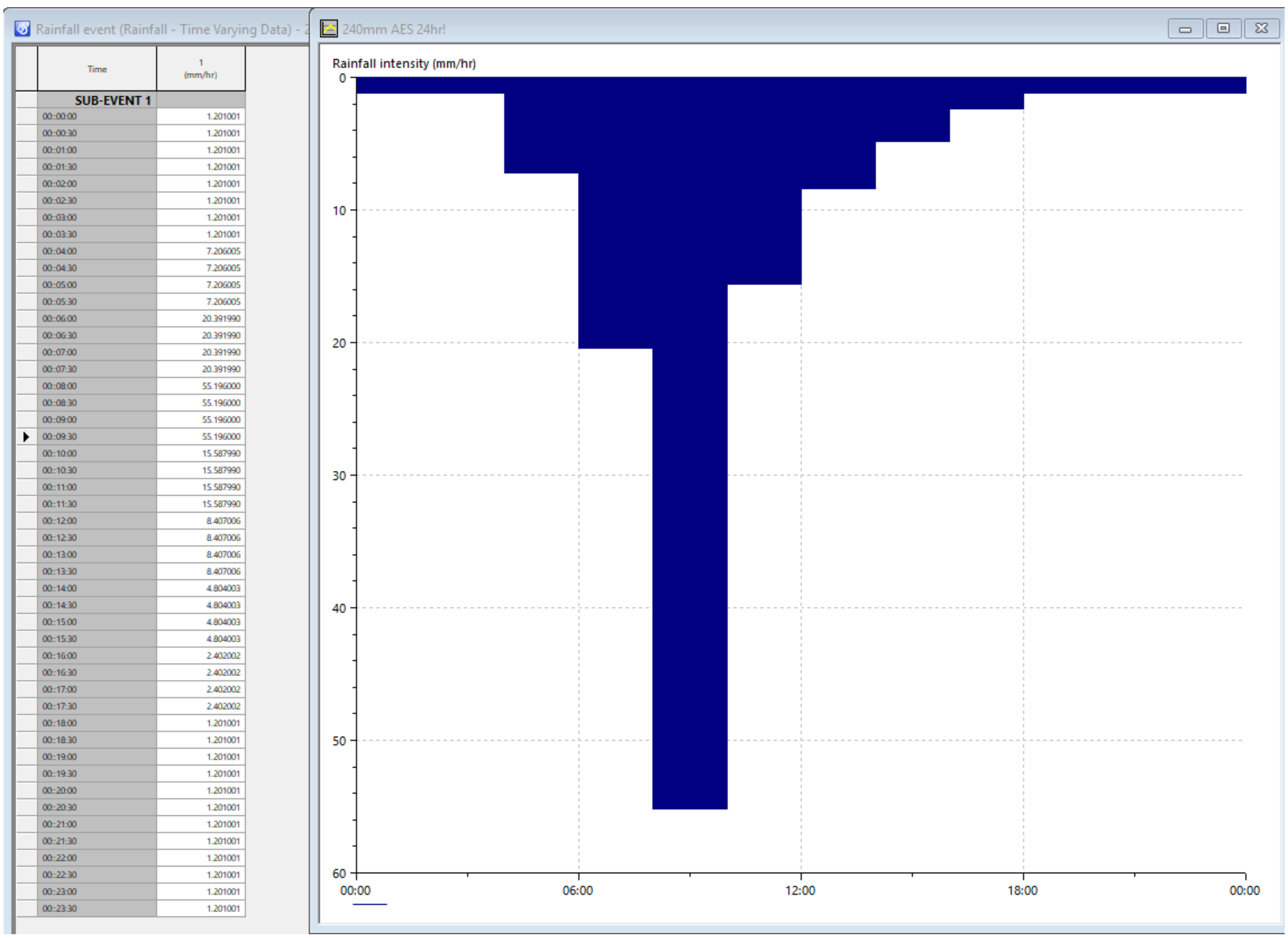
Attachment A

Modelling Support Information

Reservoir Stage-Storage Relationship

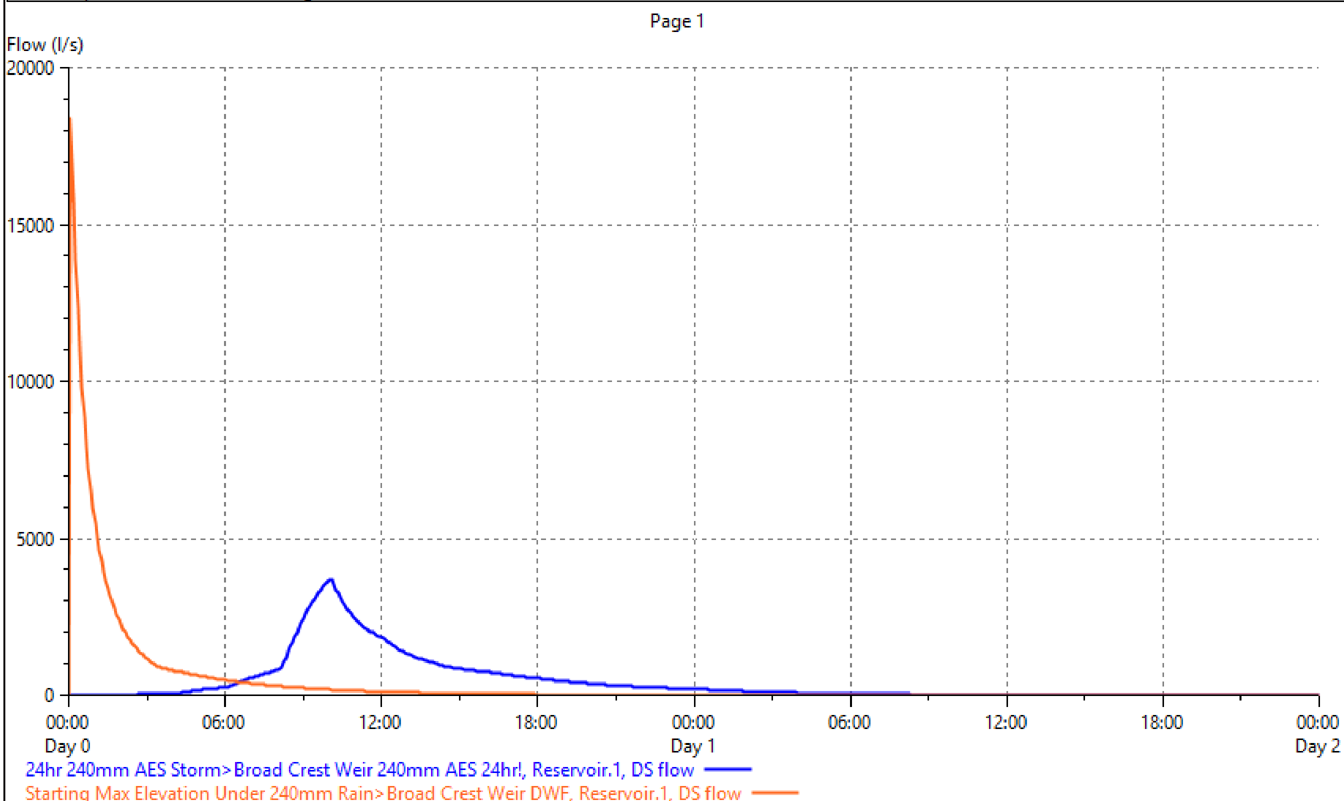
Elevation (m)	Height above Max. Operating Level (m)	Total Wetted Surface Area (m2)	Volume Above Maximum Operating Level (m3)
129.5	0	218437.7	0
129.6	0.1	219971.8	21919.62387
129.7	0.2	221490.1	43995.209
129.8	0.3	222939	66216.18677
129.9	0.4	224363.4	88583.16202
130	**0.5**	225721.1	111088.2178
131	1.5	235984.8	342122.0159
132	2.5	246206.8	582955.769

24-hour AES Rainfall Hyetograph



Flow Hydrographs for Dam Spillway Flows (Broad Crested Weir)

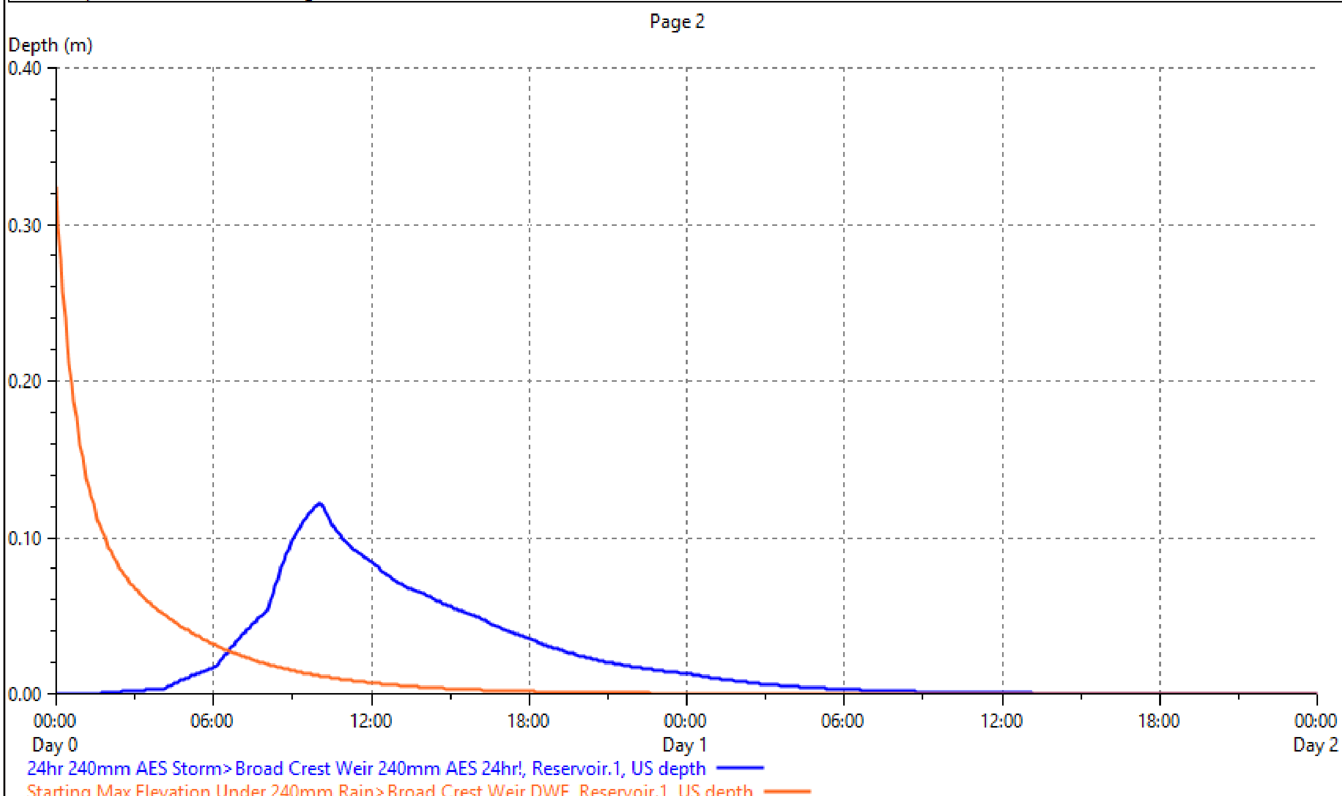
Flexible Report Produced by edwardsj0120 (2026-01-08 3:41:43 PM) Page 1 of 3
Sim: >Iqaluit Model>Runs>24hr 240mm AES Storm>Broad Crest Weir 240mm AES 24hr! (2025-12-10 4:36:32 PM)
Sim: >Iqaluit Model>Runs>Starting Max Elevation Under 240mm Rain>Broad Crest Weir DWF (2025-12-10 4:14:00 PM)



	DS flow		
	Min (l/s)	Max (l/s)	Volume (m3)
...est Weir 240mm AES 24hr!, Reservoir.1	0.000	3677.780	71221.658
...in>Broad Crest Weir DWF, Reservoir.1	0.000	17938.784	69380.503

Depth Plots for Damn Spillway (Broad Crested Weir)

Flexible Report Produced by edwardsj0120 (2026-01-08 3:43:19 PM) Page 2 of 3
Sim: >Iqaluit Model>Runs>24hr 240mm AES Storm>Broad Crest Weir 240mm AES 24hr! (2025-12-10 4:36:32 PM)
Sim: >Iqaluit Model>Runs>Starting Max Elevation Under 240mm Rain>Broad Crest Weir DWF (2025-12-10 4:14:00 PM)



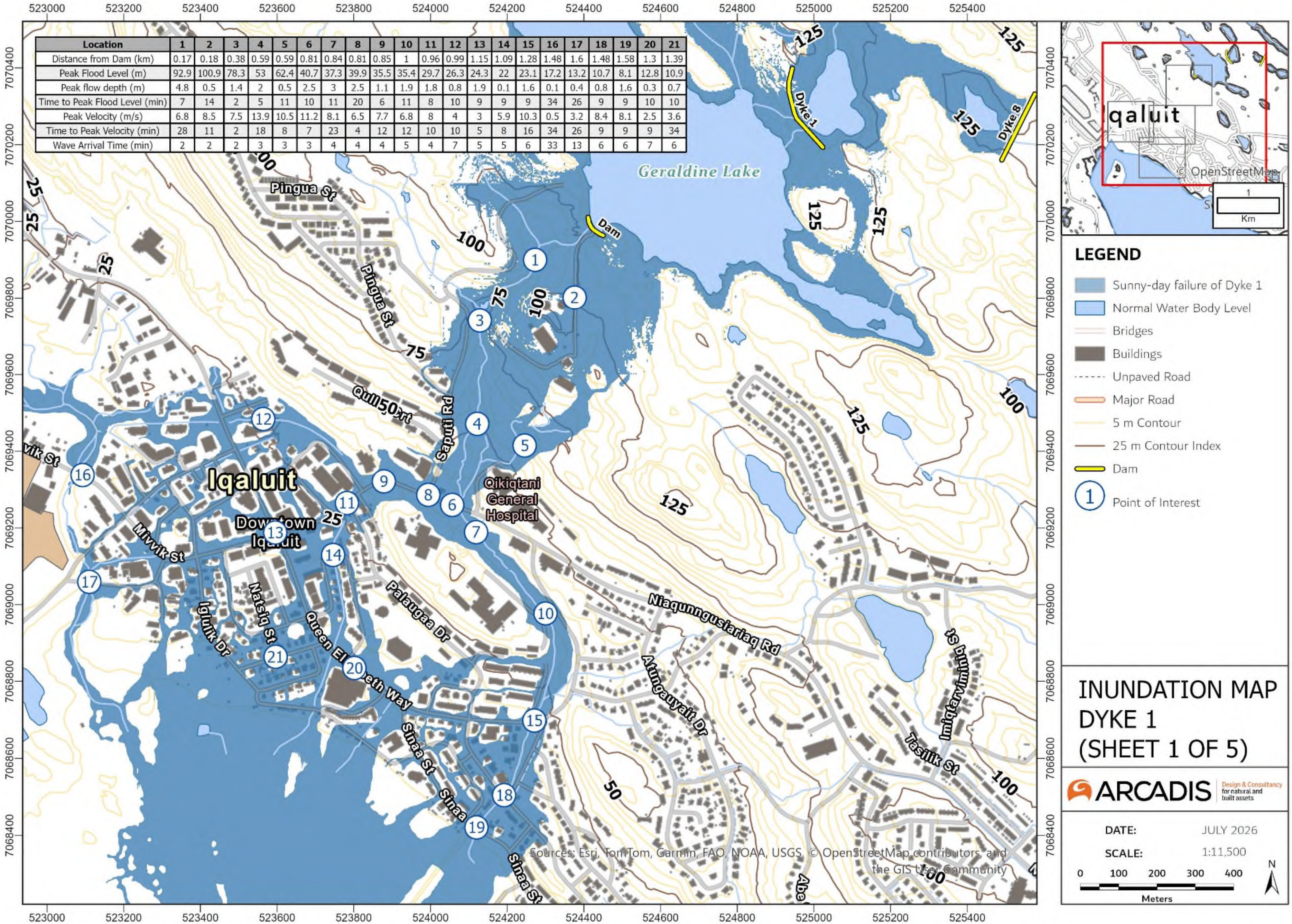
	US depth	
	Min (m)	Max (m)
...rm>Broad Crest Weir 240mm AES 24hr!, Reservoir.1	0.000	0.122
...er 240mm Rain>Broad Crest Weir DWF, Reservoir.1	0.000	0.325

Appendix B

B – Inundation Maps

INUNDATION MAP DYKE 1 (SHEET 1 OF 5)

Location	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Distance from Dam (km)	0.17	0.18	0.38	0.59	0.59	0.81	0.84	0.81	0.85	1	0.96	0.99	1.15	1.09	1.28	1.48	1.6	1.48	1.58	1.3	1.39
Peak Flood Level (m)	92.9	100.9	78.3	53	62.4	40.7	37.3	39.9	35.5	35.4	29.7	26.3	24.3	22	23.1	17.2	13.2	10.7	8.1	12.8	10.9
Peak flow depth (m)	4.8	0.5	1.4	2	0.5	2.5	3	2.5	1.1	1.9	1.8	0.8	1.9	0.1	1.6	0.1	0.4	0.8	1.6	0.3	0.7
Time to Peak Flood Level (min)	7	14	2	5	11	10	11	20	6	11	8	10	9	9	9	34	26	9	9	10	10
Peak Velocity (m/s)	6.8	8.5	7.5	13.9	10.5	11.2	8.1	6.5	7.7	6.8	8	4	3	5.9	10.3	0.5	3.2	8.4	8.1	2.5	3.6
Time to Peak Velocity (min)	28	11	2	18	8	7	23	4	12	12	10	10	5	8	16	34	26	9	9	9	34
Wave Arrival Time (min)	2	2	2	3	3	3	4	4	4	5	4	7	5	5	6	33	13	6	6	7	6



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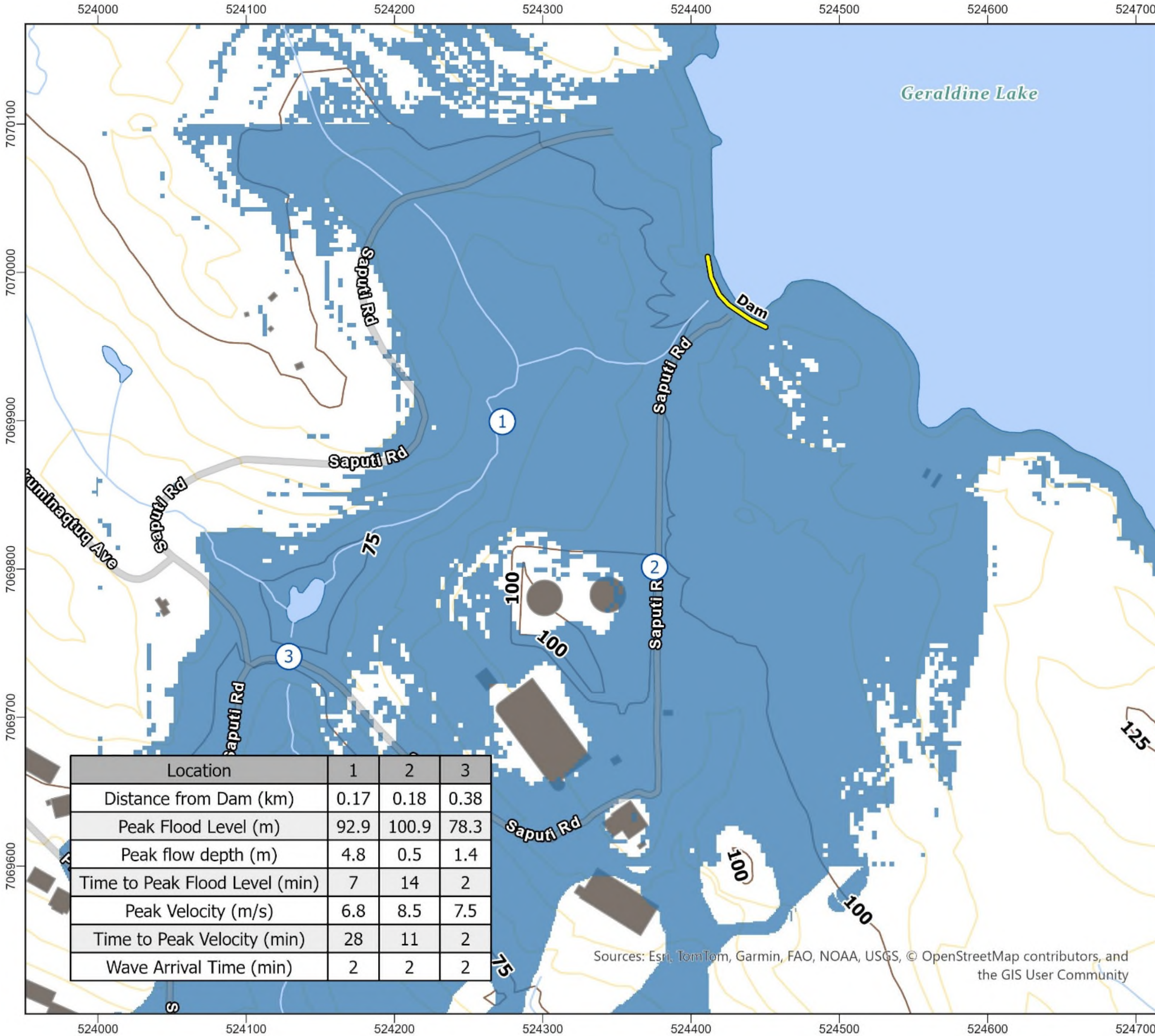
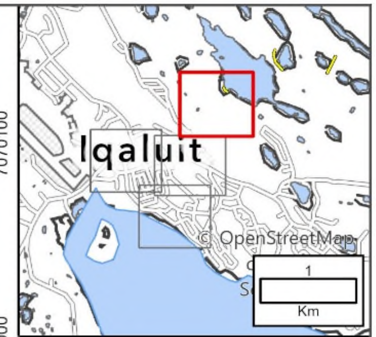


Location	1	2	3
Distance from Dam (km)	0.17	0.18	0.38
Peak Flood Level (m)	92.9	100.9	78.3
Peak flow depth (m)	4.8	0.5	1.4
Time to Peak Flood Level (min)	7	14	2
Peak Velocity (m/s)	6.8	8.5	7.5
Time to Peak Velocity (min)	28	11	2
Wave Arrival Time (min)	2	2	2



LEGEND

- Sunny-day failure of Dyke 1
- Normal Water Body Level
- Bridges
- Buildings
- Unpaved Road
- Major Road
- 5 m Contour
- 25 m Contour Index
- Dam
- Point of Interest

INUNDATION MAP DYKE 1 (SHEET 2 OF 5)

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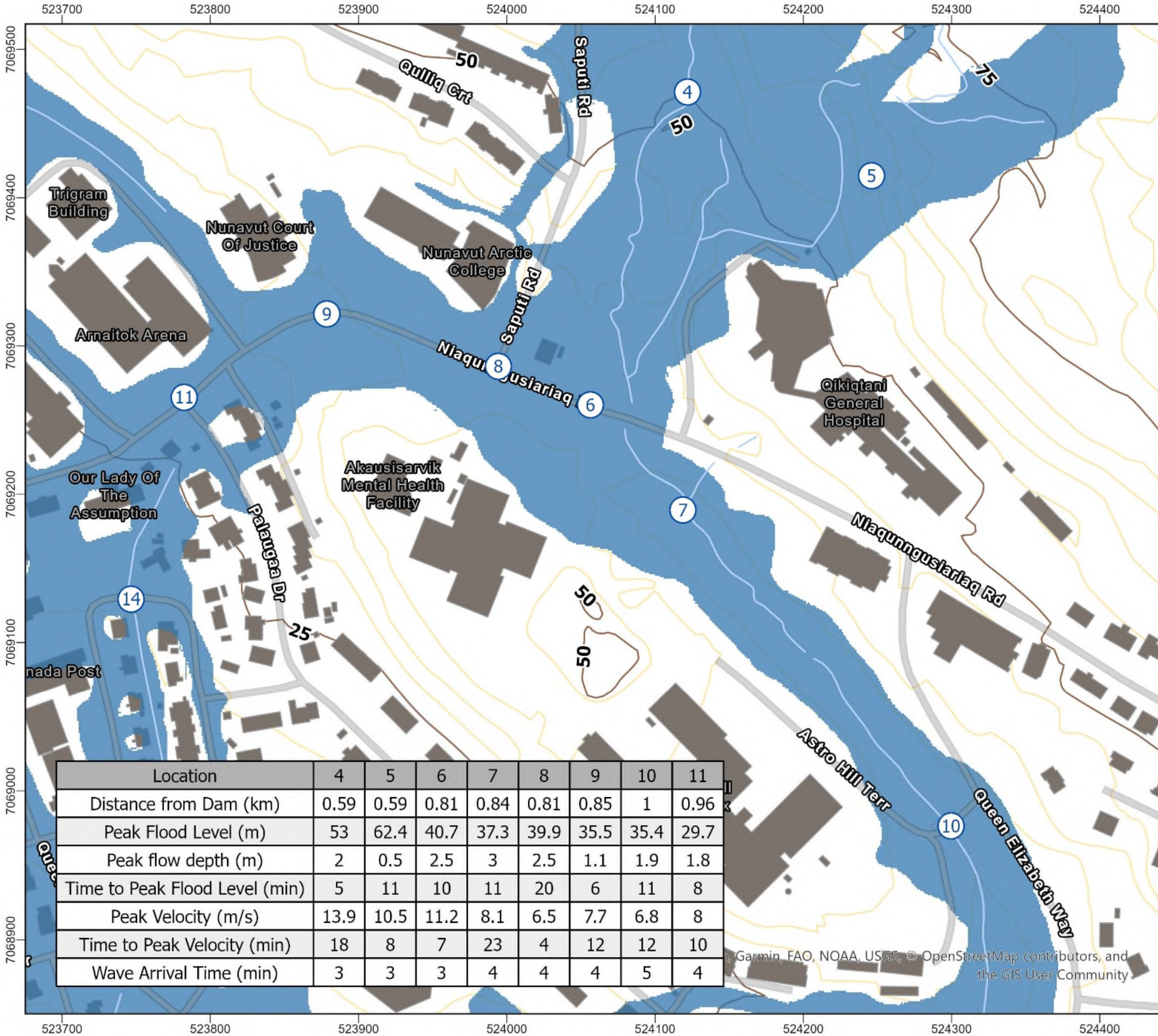
DATE: JULY 2026

SCALE: 1:3,333

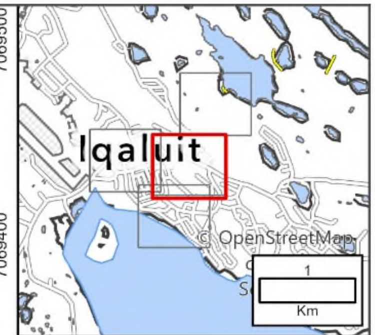
0 30 60 90 120

Meters

N



Location	4	5	6	7	8	9	10	11
Distance from Dam (km)	0.59	0.59	0.81	0.84	0.81	0.85	1	0.96
Peak Flood Level (m)	53	62.4	40.7	37.3	39.9	35.5	35.4	29.7
Peak flow depth (m)	2	0.5	2.5	3	2.5	1.1	1.9	1.8
Time to Peak Flood Level (min)	5	11	10	11	20	6	11	8
Peak Velocity (m/s)	13.9	10.5	11.2	8.1	6.5	7.7	6.8	8
Time to Peak Velocity (min)	18	8	7	23	4	12	12	10
Wave Arrival Time (min)	3	3	3	4	4	4	5	4



LEGEND

- Sunny-day failure of Dyke 1
- Normal Water Body Level
- Bridges
- Buildings
- Unpaved Road
- Major Road
- 5 m Contour
- 25 m Contour Index
- Dam
- Point of Interest

INUNDATION MAP DYKE 1 (SHEET 3 OF 5)

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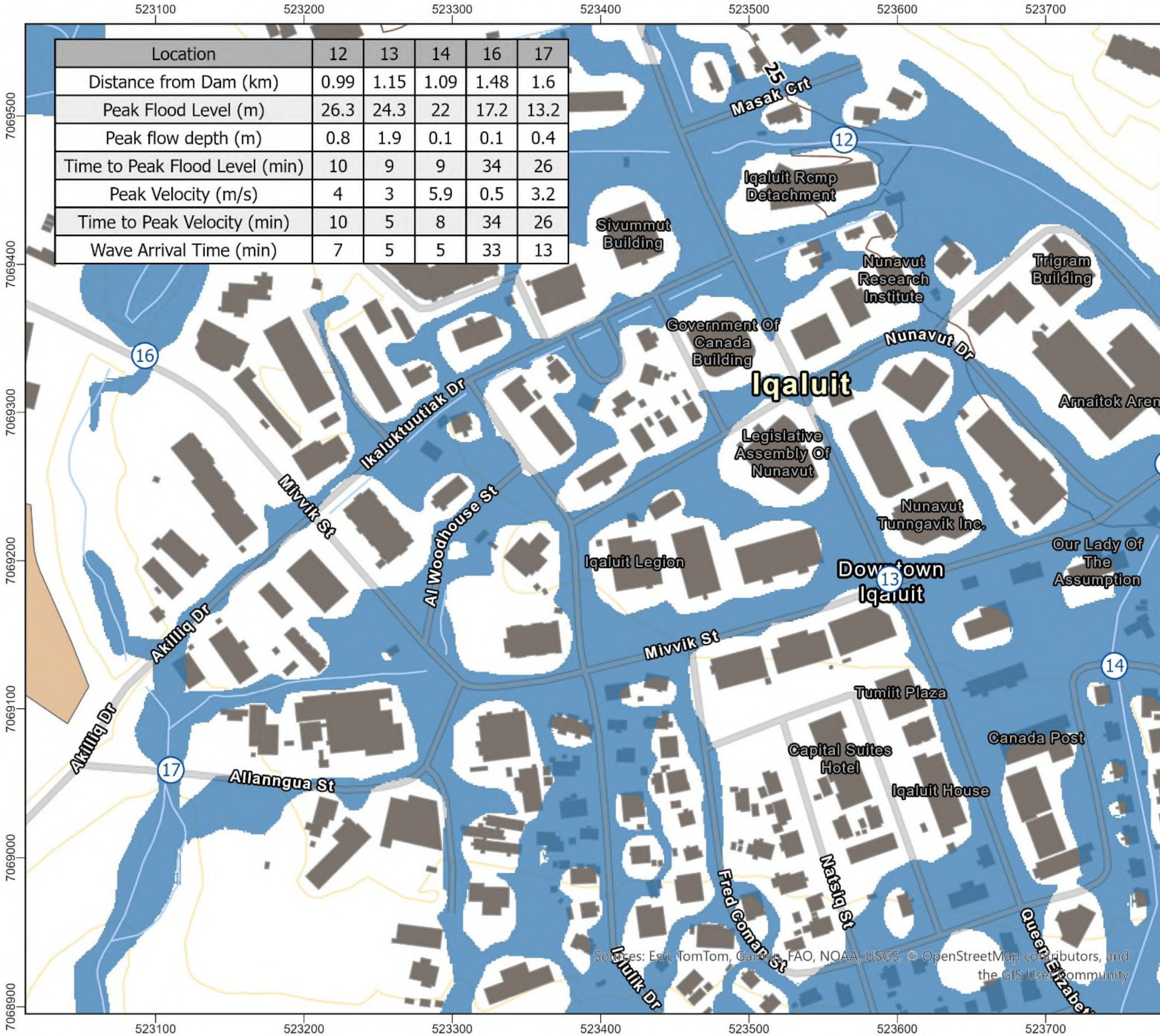
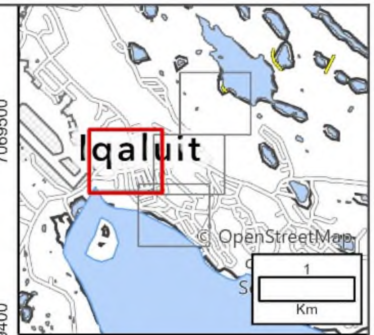
0 30 60 90 120 Meters N

Location	12	13	14	16	17
Distance from Dam (km)	0.99	1.15	1.09	1.48	1.6
Peak Flood Level (m)	26.3	24.3	22	17.2	13.2
Peak flow depth (m)	0.8	1.9	0.1	0.1	0.4
Time to Peak Flood Level (min)	10	9	9	34	26
Peak Velocity (m/s)	4	3	5.9	0.5	3.2
Time to Peak Velocity (min)	10	5	8	34	26
Wave Arrival Time (min)	7	5	5	33	13





LEGEND

- Sunny-day failure of Dyke 1
- Normal Water Body Level
- Bridges
- Buildings
- Unpaved Road
- Major Road
- 5 m Contour
- 25 m Contour Index
- Dam
- 1 Point of Interest

INUNDATION MAP DYKE 1 (SHEET 4 OF 5)

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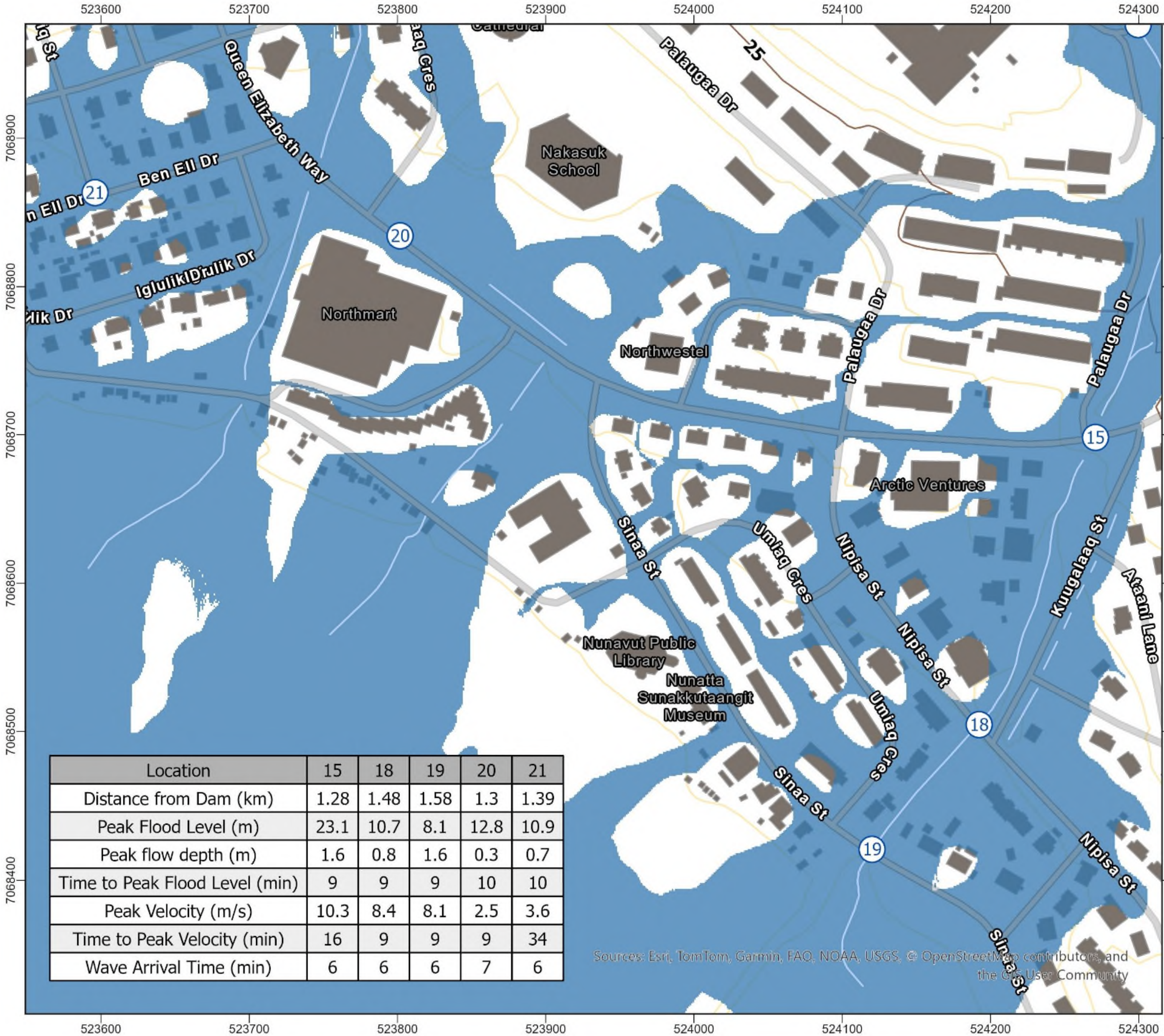
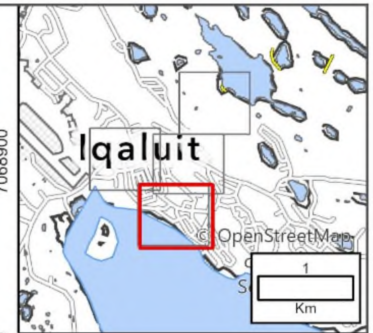
0 30 60 90 120 Meters

N



Location	15	18	19	20	21
Distance from Dam (km)	1.28	1.48	1.58	1.3	1.39
Peak Flood Level (m)	23.1	10.7	8.1	12.8	10.9
Peak flow depth (m)	1.6	0.8	1.6	0.3	0.7
Time to Peak Flood Level (min)	9	9	9	10	10
Peak Velocity (m/s)	10.3	8.4	8.1	2.5	3.6
Time to Peak Velocity (min)	16	9	9	9	34
Wave Arrival Time (min)	6	6	6	7	6



LEGEND

- Sunny-day failure of Dyke 1
- Normal Water Body Level
- Bridges
- Buildings
- Unpaved Road
- Major Road
- 5 m Contour
- 25 m Contour Index
- Dam
- 1 Point of Interest

INUNDATION MAP DYKE 1 (SHEET 5 OF 5)

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0 30 60 90 120

Meters

N

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