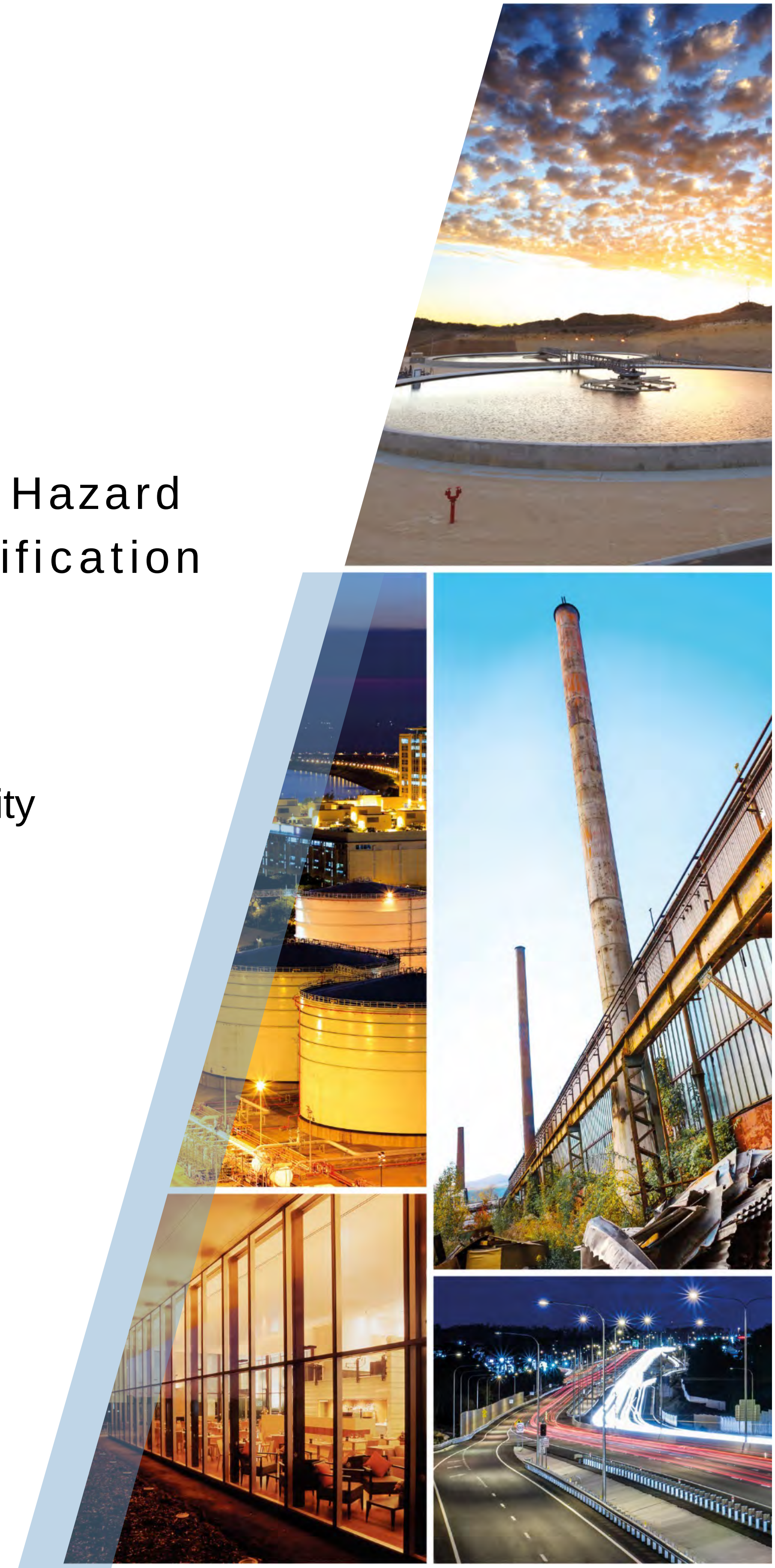
Dalewood Dam Hazard Potential Classification Report

Kettle Creek Conservation Authority



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1. Introduction and Background

Dalewood Dam was originally constructed in 1921 as a water supply system for the City of St. Thomas, Ontario. In the late 1920's, piers and stop logs were added to the structure to allow for greater storage of water in the reservoir. Since the construction of the Lake Erie pipeline for water supply for cities of London, St. Thomas and the surrounding region use of the Dalewood Reservoir for water supply ceased. Presently the dam provides environmental benefit for areas upstream of the Dalewood Road (i.e., habitat to a number of bird, fish and a number of native terrestrial and aquatic plant species). Due to the flashy nature (floods come and go quickly) of Kettle Creek, and the heavy sediment accumulation in the Dalewood Reservoir, the dam presently has limited capacity for flood control.

1.1 Study Objective

Based on the historical document review it was recommended that a review of the Hazard Potential Classification (HPC) be carried out as a best management practice. The scope of the present work is thus to undertake the necessary investigations and identify the HPC of Dalewood Dam according to the Province of Ontario's Best Management Practices for Dam Safety (MNRF, 2019).

1.2 Site and Dam Description

Dalewood Dam is located within the northern limits of the City of St. Thomas, south of the intersection of Water Tower Line and Dalewood Road (see Figure 1-1). The dam is about 6 m high and consists of an earthen berm structure with a central concrete gravity spillway. On either side of the spillway are embankments having a total length in excess of 100 m, with a 0.3 m thick concrete cut-off wall running through the center of the embankments. The central concrete gravity structure consists of an ogee-crested spillway and is approximately 25 m long measuring from left to right bank. A concrete deck exists above the central concrete gravity structure, and is supported by three concrete piers. The concrete deck measures 28 m between the wingwalls on either side. There are four bays at the control structure that accommodate stop logs which are presently placed on top of the main concrete spillway with the purpose of increasing water level in the upstream head pond. Each stop log is 6.3 m long. Each bay presently supports two stoplogs stacked on top of each other, each being nominally 0.25 m (10 inches). The total vertical height of stop logs is thus 0.50 m. The deck supports a stop log lifting device that runs on rail tracks, and is used to remove stop logs from each bay.

The storage capacity of the Dalewood Reservoir is reduced due to sediment build up behind the dam over the last century. There is a low flow control device at the control structure that allows the reservoir to be drained during maintenance operations.

Approximately 40 m upstream of Dalewood Dam is the recently re-constructed Dalewood Bridge (completed during the summer of 2019). The bridge abutments are fully within the limits of the existing reservoir.

There is a weir in the river approximately 1.6 km downstream of Dalewood Dam, known as the Waterworks Weir, which was originally used to divert and store water for the historic St. Thomas water treatment facility. The Waterworks Weir is estimated as being 1.5 m above the upstream river bed, and creates a nominal (in-channel) impoundment that extends some distance upstream. Adjacent to the weir is a pond and a wetland area, presently used for recreational purposes.

Along the reach of Kettle Creek within the limits of St. Thomas there are eight existing roadway, and two railway crossings (all of which were surveyed in this assignment). Dodd Creek empties into Kettle Creek about 5.8 km downstream of Dalewood Dam. Within the south most reaches of Kettle Creek within the town limits lies the St. Thomas Waste Water Treatment Plant, along with its flood protection infrastructure. The overall study area is shown in Figure 1-2.

1.3 Definitions and Conventions

In this work physical features (such as banks for the river, wingwalls, spillways, embankments, and other appurtenant structures) are referred to using a descriptor left and right. The convention adopted is one where the observer is standing in the middle of the river and is looking downstream.

Thus, the left embankment is one that an observer sees on the left when standing in the middle of the river and looking downstream. Similarly, the right embankment is one that an observer sees on the right when standing in the middle of the river and looking downstream.

1.4 Horizontal and Vertical Control

In this assignment the horizontal reference plane used is NAD83(CSRS), Epoch 2010. The vertical datum used is the Canadian Geodetic Vertical Datum 2013 (CGVD2013). All topographic and bathymetric surveys, maps, inundation boundaries, flood elevations and all other references are made to the above noted standard. The project uses SI units, with dimensions reported in meters [m], and discharges reported in meters cubed per second [m^3/s].

1.5 Scope of Work

The main objective of this work is the identification of the HPC at Dalewood Dam. Knowing the HPC at a dam site aids in the determination of the Inflow Design Flood (IDF), which is presently unknown. The tasks within the scope of the present undertaking include:

- Data collection and background review
- Topographic and bathymetric surveying
- DEM processing for hydraulic modeling
- Hydrologic analyses
- Hydraulic capacity calculations
- Assessment of optimal dam operations
- Preliminary HPC
- Dam break analysis
- Confirmation of HPC
- Selection of a IDF

- Preliminary geotechnical evaluation
- Public safety assessment
- Preparation of a summary report

In the text that follows pertinent details on the above tasks are provided, and summarized.

2. Background Review and Data Collection

A number of previous investigations were reviewed for relevance to the present undertaking (determination of the HPC at Dalewood Dam). The reports reviewed are listed below.

2.1 Background Review

2.1.1 1981 Dalewood Reservoir Study

The Dalewood Reservoir Study, completed by Ecologistics and Phillips Engineering (1981) addresses the problem of unsatisfactory environmental conditions at the Dalewood Reservoir. The main problem was identified as poor water quality, resulting from the amount of suspended sediment causing high turbidity and nutrient levels. The nutrients were found to originate in upstream runoff from farms. Poor water quality in the reservoir was identified to create several related problems, including: i) adverse impact of fish and wildlife, and ii) low recreational uses.

The 1981 study notes that the reservoir provides little or no flood control protection to downstream areas. The study notes that the spillway capacity without freeboard amounts of 220 m^3/s (Appendix E of the 1981 study).

2.1.2 1987 St. Thomas Flood Damage Centre Study

The St. Thomas Flood Damage Centre Study (Latham, 1987) completed comprehensive hydrotechnical analyses to quantify the risk of flooding and identify possible flood damage reduction measures to areas which are inundated by floods. Ten damage centers were identified, of which seven were noted as being inundated by the Regulatory flood.

The cause of flooding was identified as insufficient capacity of the channel to convey peak flows. The cause of flood damage was traced to indiscriminate development along the river's floodplain.

Latham (1987) completed detailed hydrologic analysis (statistical flood flow frequency analysis, development, calibration and verification of a hydrologic model) for the Kettle Creek at St. Thomas. The same study also completed hydraulic analysis and estimated water surface profiles for various flood flow conditions (ranging from 2-yr to Regional Storm). Latham (1987) completed a detailed flood damage assessment (split by reaches) along Kettle and Dodd Creeks, provided a range of flood damage mitigation options, and carried out an economic cost-benefit analysis for each identified option.

The Latham (1987) study also considered potential damages due to failure of Dalewood Dam. The study used a simplified dam break model where: i) the downstream channel was approximated as prismatic, ii) off-channel storage was neglected, iii) backwater effects from downstream bridges was also neglected, iv) contribution from Dodd Creek was neglected, and v) only five cross sections downstream of the dam were analyzed.

Realizing the simplistic nature of the dam breach modeling carried out, the Latham (1987) noted that should Dalewood Dam deteriorate further, more sophisticated dam breach modeling could be undertaken.

The Latham (1987) study included a section of the environmental effects of dam failure. Downstream of the dam the effects included: i) bank erosion, ii) failure of the waterworks weir, iii) vegetation damage and silt deposition at Waterworks Park, iv) flooding of properties and buildings at Sunset Drive and Parkins Avenue, v) threatening facilities at The St. Thomas Water Pollution Control Plant. Upstream effects of dam failure included: i) draining of the headpond, ii) deposition of silt downstream, iii) adverse effects to fish spawning downstream, iv) reduction of fish population within the reservoir.

2.1.3 1988 Dalewood Dam Assessment Study

The Dalewood Dam Assessment Study (Delcan, 1988) focused on a review of relevant reports for the purpose of assessing costs and benefits of options of abandonment, repair, replacement, or portion of, or the entire Dalewood Dam structure. A total of 14 alternatives were considered in the analysis. The ranking of options were accommodated using five categories (social and natural environment, economic, engineering, and reliability). The outcome of the study was to replace the deck and wing walls.

Replacement of the deck took place in 1989, and was the last time major repair works took place at the Dalewood Dam control structure.

2.1.4 2010 Dalewood Dam Inspection Report

The inspection report was carried out by Riggs Engineering (2010) and focused on the condition of the downstream embankments and the masonry training wall.

The report identified that a portion of the downstream slope of the south (left) embankment slipped, causing roots of a large tree to be exposed. The recommended option included removing existing protection, re-grading the slope to a 2H:1V inclination, and placing new riprap.

The north (right) embankment was noted to be protected by a masonry training wall, which showed signs of overtopping. The area behind the training wall was noted as eroded, but there was no serious damage. The recommended option was for the situation to be further monitored. Probing of the river bed in front of the wall revealed no concerns related to scour/undermining.

2.1.5 2013 Inspection of Low Flow Valves

Watech Diving Services (Watech, 2013) carried out an underwater diving inspection of the low flow valves at Dalewood Dam. Watech noted that due to heavy infilling of sediment in front of the dam, the north (right) chamber is presently several meters under the sediment level.

The south (left) chamber, is below sediment level, and has valves at three different elevations to permit discharges at various water levels. All valves in the south (left) chamber were noted as heavily damaged, and in poor working condition.

In May 2014 Watech was contracted to replace the broken uppermost low flow valve with a new valve. The remaining low flow valves that are below the sediment level were disconnected and closed to prevent any future uncontrolled discharge of sediment downstream.

2.1.6 2018 Inspections of Water Control Structure at Dalewood Dam

Watech Diving Services (Watech, 2018) completed inspection services at the Dalewood Dam following a major flooding event during winter of 2018. The dam was noted to be in fair to good condition. The inspections focused on superstructure, earth embankments, stop logs, low flow valves and vegetation management. The inspection report provided useful life, recommended repairs, and cost estimates.

2.2 Data Collection Efforts

The following data were provided by KCCA for use in this project:

- Water Survey of Canada (WSC) streamflow measurements at gauges i) 02GC002 – Kettle Creek above St. Thomas for years 1969-2018, and ii) 02CG029 – Kettle Creek at St. Thomas, for years 1985-2018, time step 15 minutes.
- Ministry of Natural Resources and Forestry (MNRF), Southwestern Ontario Ortho-rectification Project (SWOOP 2015) aerial photograph for the City of St. Thomas limits.
- 2016-2018 Ministry of Natural Resources and Forestry (MNRF), Lake Erie Lidar Digital Elevation Model, 0.5 m x 0.5 m resolution, for the City of St. Thomas limits.
- Kettle Creek Conservation Authority (KCCA) topographic and bathymetric surveys at Dalewood Dam, collected in 2017 and 2018.

3. Site Visit and Field Data Collection

3.1 Site Inspections

Site inspections took place on July 24, 2019 with Pat Prodanovic, Ph.D., P.Eng. (Hydrotechnical Engineer), and Hassan Gilani, M.Sc., P.Eng. (Geotechnical Engineer) of GHD. Jennifer Dow of KCCA accompanied GHD staff during the site inspections.

The scope of work for inspections focused on carrying out visual inspections of the left and right earthen embankments, and obtaining measurements of key elevations at the concrete spillway for hydraulic capacity calculations.

The upstream water level in the reservoir at the time of our inspections was recorded as 215.15 m; the downstream water level along the left downstream wingwall was measured as 209.00 m.

3.1.1 Left Embankment

The majority of the left upstream embankment was noted as being lined with riprap, 300-600 mm in diameter during the construction activities associated with the Dalewood Bridge replacement project between 2018-2019. A small portion of the left upstream embankment adjacent of the wingwall was left intact, and is covered with smaller (50-100 mm diameter) riprap along with minor vegetation.

Given the heavy deposition of sediment in the head pond, the upstream toe is buried in reservoir sediment.

The crest of the left embankment was noted as being lined with granular material, placed to accommodate machinery used in the recent placement of the riprap, noted above.

A large portion of the left downstream embankment slope is covered with mature trees and brush, and numerous fallen trees running from the Dalewood Road to the left wingwall. An area of standing water was noted on the face of the left downstream embankment, adjacent to the left downstream wingwall. A small test pit was hand dug to determine if active seepage could be observed with depth. The soils at the bottom of the test pit became progressively drier with depth, getting to a state of natural moist conditions at a depth of approximately 0.6 m, thus indicating the observed saturation was likely due to a localized low lying area rather than any seepage from the pond. The area of wetness at the same location was noted in the Watech (2018) inspections, and Riggs Engineering (2010) inspections.

The elevation of the downstream toe was observed to vary, with the toe having the lowest elevation in the proximity of the left wingwall. The lowest elevation of the left downstream embankment toe coincides with the past failure of the slope (Riggs, 2010), and the area with saturation.

3.1.2 Right Embankment

The approach to the right embankment was modified during the Dalewood Bridge construction activities to accommodate installation of a culvert of an un-named tributary that empties in the reservoir upstream of the dam. The noted bridge construction activities did not modify the right embankment itself.

The face of the right upstream embankment is covered with dense brush vegetation, mature dead and fallen trees. Riprap is visible along a small section of the right upstream embankment adjacent the wingall, with the remaining portion heavily vegetated.

The upstream toe is likewise covered with sedimentation that is present in the head pond (similar conditions as the left embankment).

The crest of the right embankment is vegetated with grass, and appears free of cracks, settlement, or other noticeable defects.

Face of the right downstream embankment is likewise overgrown with dense brush, trees, and other vegetation. An existing training wall lies at the toe of the embankment, creating a small flat bench against the native valley slope.

The toe of slope of the embankment has been artificially raised by the construction of the training wall. Evidence of overtopping of the training wall are visible, but no immediate erosion concerns were noted. Evidence of seepage or wetness were not observed in the downstream slope.

3.1.3 Control Structure

The deck at Dalewood Dam is supported by concrete piers added in the late 1980’s. The existing concrete piers accommodate placement of stop logs on top of the main concrete spillway structure, and thus allow manual control of the water levels at the site.

Length of opening in each of the four bays was measured during inspections, along with distances from the deck to top of stop logs, and from the deck to the top of the concrete spillway structure. During inspections, two stop logs rested on the concrete spillway in each bay.

The measurement made during the site visit on July 24, 2019 include the following:

Top of Deck EL	= 217.40 m
Crest EL of Concrete Spillway	= 214.52 m
Top of Stop Logs EL	= 215.04 m (two stop logs on top of spillway at each bay)
Opening of each bay	= 6.3 m

The above measurements allow for computation of the spillway capacity at Dalewood Dam.

3.2 Topographic Survey at Dam Site and Bridge Crossings

For all topographic survey efforts GHD used Leica Geosystem Global Navigation Satellite System GNSS receivers with the SmartCheck+ technology to assure the highest confidence for all field measurements. High-precision Network RTK corrections are provided by SmartNet Network of Virtual Reference Stations for projection of the GNSS positions in NAD83(CSRS), Epoch 2010 coordinate system. Elevation ground proofing has been obtained by physically occupying a local National Resources Canada (NRC) Benchmark LON1-813041, known as London Calibration Pier 1, located 111 m southwest of the Scotland Drive overpass of Highway 401. The said benchmark is classified by NRC as high precision 3D for vertical and horizontal control. Then London Calibration Pier 1 benchmarks provides a vertical datum to CGVD2013, and horizontal reference plane to NAD83(CSRS), Epoch 2010, thus being consistent with MNRF 2018 LIDAR data and the previous KCCA topographic surveys.

For this assignment topographic surveying was carried out for all major bridges and control structure of Kettle and Dodd Creeks within the limits of St. Thomas, Ontario. The limits of the topographic surveying covered the same boundary as in the previous Latham (1987) report. The bridges/control structures included in our surveying scope included the following:

- Waterworks weir (top of weir, and bottom of channel)
- Railway bridge downstream of Waterworks Park
- Highway 3 bridge
- Railway bridge at Athletic Park
- St. George Street Bridge
- Talbot Hill Bridge over Kettle Creek
- Sunset Drive Bridge north of Fingal Line
- Fingal Line Bridge
- Sunset Drive Bridge north of VA Barrie Park
- Sunset Drive Bridge south of VA Barrie Park
- Bush Line Bridge (near St. Thomas Water Pollution Control Plant)
- Talbot Hill Bridge over Dodd Creek

- Wellington Road Bridge over Dodd Creek
- Sunset Drive Bridge over Dodd Creek

The scope of the surveying was defined to stay consistent with the requirement of the hydraulic modeling required for the HPC analysis according to the Provincial guidelines (MNRF, 2011a). A survey crew of two drove to each location and collected the following information:

- Photograph of bridge opening
- Top elevation of the bridge deck
- Measurement from the bridge deck to the underside of the soffit
- Elevations of the creek at water's edge (left bank), toe of slope (left bank), a number of points in the main channel, toe of slope (right bank), and water's edge (right bank)
- Dimensions of structure opening
- Number and size of piers (if present)

3.3 Bathymetric Survey of Dalewood Reservoir

For the purposes of estimating the volume of water in the reservoir during dam breach calculations a bathymetric survey of the Dalewood Reservoir upstream of the Dalewood Bridge was conducted. Bathymetric surveying efforts downstream of the Dalewood Bridge were not required as these were surveyed by KCCA in 2017. The bathymetry survey extended from shore to shore, spanning the entire impoundment. The surveying was carried out by mounting an echo sounder to a small watercraft powered by a small outboard motor. Bathymetric data was collected using a single frequency echo sounder that links to a pole-mounted GNSS receiver. The field staff employed multiple methods and technologies to measure bathymetry at Dalewood Reservoir, including sonar, lead and plumb line, and direct survey using a pole mounted RTK GNSS (when required). The bathymetric surveying was completed on July 24, 2019.

4. Preliminary Hazard Potential Classification and Inflow Design Flood

4.1 Preliminary Hazard Potential Classification

Within the Province of Ontario the Ministry of Natural Resources and Forestry (MNRF), through its Lakes and Rivers Improvement Act (LRIA), administers various aspects related to construction of new and modification of existing dams (MNRF, 2017). Typically, a proponent proposing new (or altering existing) works is required to satisfy a number of criteria set forth by MNRF. The said criteria are laid out in a series of Technical Guides, all to be read in conjunction with MNRF (2017).

The work in this report deals only with the aspects related to determining the dam's Hazard Potential Classification (HPC) and identifying the dam's Inflow Design Flood (IDF). The criteria for such analysis are presented in the Classification and Inflow Design Flood Criteria Technical Bulletin (MNRF, 2011a).

In summary, Ontario's HPC system categorizes each dam (as Low, Moderate, High or Very High) according to the greatest incremental losses that would result should a dam malfunction and spill the impounded water downstream. The incremental losses are evaluated according to human life, property, and environmental and cultural (built heritage) sites impacted or otherwise influenced by the dam.

The Inflow Design Flood (IDF) for a dam is selected based on the results of an incremental hazard evaluation carried out using a dam breach analysis. This evaluation involves simulating a dam failure during the sunny-day and flood flow conditions, and routing the flood wave downstream. The additional downstream threat from the incremental increase in water surface elevation from a dam failure is assessed in each case.

The preliminary HPC for Dalewood Dam has not been previously determined. In the text that follows the preliminary HPC shall be established, and then confirmed via detailed dam breach modeling.

All monetary values in MNRF (2011a) are indexed to Statistics Canada values at year 2000.

MNRF (2011a) outlines the Provincial criteria for determining the HPC for individual dams in Ontario. The criteria are separated according to: i) loss of life, ii) property losses, iii) environmental losses, and iv) cultural-built heritage losses. Each is discussed next.

4.1.1 Loss of Life

The loss of life category is defined as follows in MNRF (2011a):

LOW – No potential loss of life.

MODERATE – No potential loss of life.

HIGH – Potential loss of life of 1-10 persons.

VERY HIGH – Potential loss of life of 11 or more persons.

The City of St. Thomas is located downstream of Dalewood Dam. However, given the high amount of sediment accumulated in the reservoir, the volume of water stored is limited. A dam breach scenario would generate a flood wave downstream, but given the limited volume stored in the reservoir, the incremental flood wave from a breach is anticipated to be small. Furthermore, all existing mapping suggests that most residential dwellings are located outside of the 100-yr floodplain (only one is located within Parkins Avenue). These facts lead to a preliminary conclusion that a breach at Dalewood Dam would likely lead to no potential loss of life.

4.1.2 Property Losses

The property losses category is defined as follows in MNRF (2011a):

LOW - Minimal damage to property with estimated losses not to exceed $300,000.

MODERATE - Moderate damage with estimated losses not to exceed $3 million, to agricultural, forestry, mineral aggregate and mining, and petroleum resource operations, other dams or structures not for human habitation, infrastructure and services including local roads and railway lines. The inundation zone is typically undeveloped or predominantly rural or agricultural, or it is managed so that the land usage is for transient activities such as with day-use facilities. Minimal damage to residential, commercial, and industrial areas, or land identified as designated growth areas as shown in official plans.

HIGH - Appreciable damage with estimated losses not to exceed $30 million, to agricultural, forestry, mineral aggregate and mining, and petroleum resource operations, other dams or residential, commercial, industrial areas, infrastructure and services, or land identified as designated growth areas as shown in official plans. Infrastructure and services includes regional roads, railway lines, or municipal water and wastewater treatment facilities and publicly-owned utilities.

Given the high amount of sedimentation found in the reservoir that could lead to a limited incremental flood wave, some downstream property damage could still be anticipated. These would likely be limited to accessory structures (garages, sheds, etc.), with anticipated damage to be below $300,000 (year 2000 dollars). Note the monetary damage is for third party losses (i.e., the loss of dam or impact to property of the dam owner is excluded).

4.1.3 Environmental Losses

The property losses category is defined as follows in MNRF (2011a):

LOW - Minimal loss of fish and/or wildlife habitat with high capability of natural restoration resulting in a very low likelihood of negatively affecting the status of the population.

MODERATE - Moderate loss or deterioration of fish and/or wildlife habitat with moderate capability of natural restoration resulting in a low likelihood of negatively affecting the status of the population.

HIGH - Appreciable loss of fish and/ or wildlife habitat or significant deterioration of critical fish and/or wildlife habitat with reasonable likelihood of being able to apply natural or assisted recovery activities to promote species recovery to viable population levels. Loss of a portion of the population of a species classified under the Ontario Endangered Species Act as Extirpated, Threatened or Endangered, or reversible damage to the habitat of that species.

The release of sediment during a potential dam breach could pose a potential environmental impact to the downstream aquatic and terrestrial habitat. Some temporary habitat loss within the banks could be expected following a dam breach, however natural restoration within five years or less are anticipated as reasonable with no long term losses. The major reason for no long term habitat losses is that Kettle Creek is very flashy, meaning that deposited sediment from a breach is anticipated to be moved quickly during flood events. The released sediment will be carried downstream, passing through Port Stanley, and ultimately empty into Lake Erie.

The harbour at Port Stanley is nowadays likely at, or close to, equilibrium conditions (where majority of the incoming sediment is either flushed out to Lake Erie each season, or some amount deposited in the harbour). With a potential breach and release of sediment from Dalewood Reservoir this dynamic equilibrium would be disrupted temporarily. The noted disruption could have a temporary negative impact on vessel navigation.

4.1.4 Cultural – Built Heritage Losses

LOW - Reversible damage to municipally designated cultural heritage sites under the Ontario Heritage Act.

MODERATE – reversible damage to municipally designated cultural heritage sites under the Ontario Heritage Act. Reversible damage to provincially designated cultural heritage sites under the Ontario Heritage Act or nationally recognized heritage sites.

HIGH – Irreversible damage to provincially designated cultural heritage sites under the Ontario Heritage Act or damage to nationally recognized heritage sites.

There are no cultural-built heritage sites identified in the area downstream of Dalewood Dam within the City of St. Thomas limits, or areas downstream.

4.1.5 Preliminary HPC

Based on the above reasoning, the preliminary HPC for Dalewood Dam is set as LOW. A dam breach analysis is carried out to confirm the preliminary classification, and is detailed in the subsequent sections of this report.

4.2 Inflow Design Flood

Based on the HPC rating of LOW, the MNRF (2011a) publications suggests that the IDF ranges from 25-yr to 100-yr flood conditions. In this work, the IDF at Dalewood Dam is thus conservatively set as the peak 100-yr flood.

5. Digital Terrain Processing

A large scale Lidar digital terrain model is required for hydraulic modeling as it efficiently captures geometry of the terrain for large areas of land. However, the Lidar sensors are not able to penetrate the water's surface, thus resulting in inaccurate elevations below the water line. Geometry of the terrain under the water's surface is thus not captured using Lidar products, but is required for accurate assessments of river hydraulics.

This section outlines the methodology that combines the Lidar derived Digital Elevation Model (DEM) with digital terrain models and DEMs derived from topographic and bathymetric surveying. The combining of Lidar with the survey derived DEMs are used to construct a hydraulic model ready DEM. The end product thus includes a digital surface accurate for both above and below water portions of the river and reservoir and is used in all subsequent hydraulic modeling in this work.

5.1 Lidar Data

The MNRF 2016-18 Lake Erie Lidar data set was provided by KCCA for use in this assignment (MNRF, 2019). Lidar data provided includes a DEM having a horizontal resolution of 0.5 m x 0.5 m. The horizontal reference system and vertical datum of the provided Lidar are consistent with specifications outlined at the beginning of this report.

The large scale Lidar DEM was cropped appropriately for use in the project. The Lidar DEM provides consistent information for the above water portion of the terrain to sufficient resolution to be used in the present undertaking. The 2019 topographic survey within the study area were used to compare elevations between topographic data collected using an RTK GNSS and the Lidar DEM product. In areas where the two sources of data overlapped, comparisons showed that on the ground measurements of elevations were consistent with the Lidar DEM product, thus providing confidence in use of the Lidar DEM elevations for this project.

The noted limitation of the Lidar DEM is that underwater portions of the terrain are not captured. Incorporating terrain below water's surface to the existing Lidar DEM was required, especially since an accurate estimate of the bed of the river and reservoir is required for assessment of hypothetical dam breach scenarios.

5.2 Merging Topographic and Bathymetric surveys with Lidar

The bathymetric soundings collected in this assignment were used to create a Triangulated Irregular Network (TIN) model of the reservoir, and then converted to a 0.5 m gridded data terrain surface. Next, the 0.5 m gridded surface representing the reservoir bathymetry was "burned into" (or merged with) the Lidar DEM ultimately producing a hydraulic model ready DEM that accurately captures the above and below water terrain of the reservoir (required for dam breach modeling).

Similarly, the below water portions of the riverbed captured with the 2019 topographic survey of bridge crossings in the study area were used to create a 0.5 m grid of the main channel, which was then "burned into" the existing Lidar DEM.

Main channel centerline and surveyed bridge cross sections were used to construct (and interpolate) the bathymetry of the main channel of the Kettle Creek within the study area. Recognizing that bathymetry at existing bridge crossings were used to construct a digital terrain model of a river measuring some 10 km is a limitation, but given the present assignment (flood and dam breach modeling) such an approximation is deemed appropriate. Interpolated digital surfaces of the main channel and reservoir have been merged with the Lidar DEM, thus producing the hydraulic model ready DEM that includes best available topographic and bathymetric data for the study area.

6. Hydrologic Analysis

6.1 Watershed Description

The Kettle Creek watershed drains mainly agricultural lands from London to Port Stanley. The drainage area of the entire watershed is about 520 square kilometers, flowing generally in the south-western direction towards Lake Erie. Within the limits of St. Thomas (and downstream of Dalewood Dam), Dodd Creek (drainage area of 101 square kilometers) empties into Kettle Creek.

The soils found in the Kettle Creek watershed are generally well-drained in the south portions, and relatively poorly drained clays in the north. The northern portion of the watershed is flat, with the creek following the natural slope of the natural slope of the land. In the southern portion of the watershed (starting at St. Thomas) the creek is deeply incised into its surrounding land forming steep valley channels (in the order of 30 m +/-) above the creek bed. The dominant land use in the Kettle Creek watershed is agricultural. The largest urban center is the City of St. Thomas, followed by the Village of Port Stanley to the south. Isolated pockets of forested land still remain in the watershed.

The study area in the present assignment includes the lands adjacent to Kettle Creek within the limits of St. Thomas (extending from the Dalewood Reservoir to the southwestern portion of the town limits). Pockets of residential development exist within the limits of the natural floodplain of the Kettle Creek, and include a recreational area at Waterworks Park, residential area at Parkins Avenue, Sunset Drive in the vicinity of Fingal Line, the St. Thomas Water Pollution Control Plan, along with municipal infrastructure (twelve bridges crossing Kettle Creek).

6.2 Previous Hydrologic Assessments

The most comprehensive hydrologic assessment was previously completed in the Latham Study (Latham, 1987). The said study carried out a hydrologic flood frequency analysis using an existing gauge and developed a hydrologic model of the watershed for the purposes of establishing flood flows.

At the time of their study (Latham, 1987) the authors had access to stream flow data from the Water Survey of Canada 02GC002 gauge Kettle Creek at St. Thomas for 19 years. The stream flow data was used in a hydrologic flood flow frequency analysis to establish flow characteristics ranging from 2-yr to 500-yr return period flows.

Detailed hydrologic modeling was carried out to establish flows resulting from the Regional Storm, defined as the Hurricane Hazel within the KCCA's administrative boundary. The hydrologic model was divided into nine sub-catchments, and five river reaches. The modeling work included calibration and verification of the model, and was ultimately used to establish Regional Flow Estimates for the Latham (1987) study.

6.3 Hydrologic Data

Since the original Latham (1987) study, Water Survey of Canada has installed an additional stream flow gauge above St. Thomas (02GC029), in addition to maintaining the Kettle Creek at St. Thomas (02GC002) gauge. For the 02GC002 gauge there is presently 50 years of available data. The Latham (1987) study had 19 years of data for the 02GC002 gauge. Given the additional stream flow data it is prudent to complete an updated flood flow frequency analysis.

The following stream flow gauges are used in this work:

WSC Id	Name	Years	Drainage Area
02GC002	Kettle Creek at St. Thomas	1968-2018	335.34 km^2
02GC029	Kettle Creek above St. Thomas	1985-2018	134.64 km^2
02GC031	Dodd Creek above Paynes Mills	1988-2018	101.37 km^2

Water Survey of Canada (WSC) 15 minute resolution stream flow data were obtained for gauges 02GC002 and 02GC029, while annual maximum stream flow data for 02GC031 gauge was extracted from the WSC HYDAT database. The raw 15 minute data was processed, and used in flood flow frequency analysis.

6.4 Flow Characterization

For the purposes of flow characterization above stream flow data was used to carry out a single station flood flow frequency analysis for the purposes of estimating recurrence intervals (or quantiles) of flow data at the project site.

The general steps in the analysis included i) extracting extreme events from the time history for each station (i.e., annual maximum flood), ii) carrying out stationarity and homogeneity tests, iii) fitting a theoretical probability distribution to the sample set, iv) and making inferences about the quantiles (return periods) of the underlying populations.

The Mann-Kendall test was used to test whether the historic data exhibited increasing or decreasing trends (i.e., stationarity) of the extracted annual maximum data. The Mann-Kendall test results for gauges 02GC002, 02GC029, and 02GC031 revealed that annual maximums were indeed stationary, meaning that extracted data could safely be used in classical statistical analysis.

The Pettit test was used to test the homogeneity statistic of the annual extreme data. The homogeneity test is used for detecting data variability and identifying if data has been collected using homogeneous or heterogeneous means. When a hydrological data set is identified as homogeneous, it means that data was recorded using similar instruments, techniques, and environments. Applying the Pettiti test for gauges 02GC002, 02GC029, and 02GC031 confirmed that annual maximum data were homogeneous.

6.4.1 Flow frequency Analysis

After confirming that annual extreme data extracted from the historic record were stationary and homogeneous, the US Army Corps of Engineers Hydrologic Engineering Center’s Statistical Software Package (HEC-SSP) was used to fit a number of different statistical distributions to the observed data and estimate return periods. In comparing a number of different statistical methods within HEC-SSP (17C EMA, LP3, LN, and GENEX L-MOMENTS) the 17C EMA method was identified as one for use in this assignment. Details on the 17C EMA method are provided in England et al. (2018).

The results of the statistical flood frequency analysis are shown in Table 6-1, Table 6-2 and Table 6-3 for various methods of determining flow quanties. All units of flow are reported in m^3/s.

Table 6-1 Flood Frequency analysis, 02GC029 Kettle Creek above St. Thomas

		Flow [m³/s]			
% Exceedance	Return Period [yrs]	17C EMA	LP3	LN	GENEX L-MOMENTS
0.2	500	219.7	219.7	244.4	208.4
0.5	200	190.0	190.0	206.4	181.4
1	100	168.3	168.3	179.6	161.5
2	50	147.1	147.0	154.3	142.1
4	25	126.3	126.3	130.3	123.0
10	10	99.2	99.2	100.2	98.0
20	5	78.7	78.7	78.4	78.8
50	2	49.8	49.8	49.0	50.8

Table 6-2 Flood Frequency Analysis, 02GC002 Kettle Creek at St. Thomas

		Flow [m³/s]			
% Exceedance	Return Period [yrs]	17C EMA	LP3	LN	GENEX L-MOMENTS
0.2	500	268.0	255.4	350.8	243.5
0.5	200	249.1	239.7	310.3	232.1
1	100	233.8	226.7	280.4	221.8
2	50	217.6	212.6	251.0	210.0
4	25	200.0	196.9	222.0	196.2
10	10	174.2	173.3	183.5	174.1
20	5	151.6	151.9	153.5	153.3
50	2	113.4	114.5	109.0	115.1

Table 6-3 Flood Frequency Analysis, 02GC031 Dodd Creek above Paynes Mills

		Flow [m³/s]			
% Exceedance	Return Period [yrs]	17C EMA	LP3	LN	GENEX L-MOMENTS
0.2	500	112.9	112.9	134.2	116.1
0.5	200	102.5	102.5	117.3	104.7
1	100	94.4	94.4	105.0	95.9
2	50	85.9	85.9	93.0	86.8
4	25	77.2	77.2	81.3	77.5
10	10	64.8	64.8	66.0	64.7
20	5	54.5	54.5	54.3	54.3
50	2	38.2	38.2	37.3	38.1

For the purposes of this assignment, the flow frequency results from the 17C EMA method (England et al. 2018) is used as the basis for flows in the rest of this report.

The flow quanties above were pro-rated at key locations of interest using the following relationship:

Q2 = Q1 x (A1 / A1)^0.75

where:

Q1[m^3/s] = flow at known gauge location
Q2[m^3/s] = flow at un-guaged location
A1 [km^2] = drainage area at gauge location
A2[km^2] = drainage area at un-guaged location

Flood flow characteristics at Dalewood Dam were established by pro-rating flows from the gauge 02GC029 Kettle Creek above St. Thomas. The gauge 02GC002 Kettle Creek at St. Thomas was used for the reach below the confluence with Dodd Creek. The same gauge was also used to pro-rate the flow below the St. Thomas Water Pollution Control Plant, as there exists a tributary (21.1 km^2) that empties into Kettle Creek near the treatment plant. Details of the locations for which stream flow was pro-rated is shown in Figure 6-4.

Table 6-5 shows the results of the pro-rating of flow, and summarizes the flood characteristics ranging from 2-yr to 100-yr, as well as the Regional Storm.

Based on the analysis in this work, the 100-yr flow at Dalewood Dam is estimated as 212.69 m^3/s.

Table 6-4 Key Locations of Interest

Location Name	Drainage Area [km^2]	WSC Gauge Id
Kettle Creek above St. Thomas	134.64	02GC029
Kettle Creek at Dalewood Dam	183.96	-
Kettle Creek at St. Thomas	335.34	02GC002
Kettle Creek below WPCP	356.44	-

Table 6-5 Flood Frequency Analysis

Location Name	2-yr [m^3/s]	10-yr [m^3/s]	25-yr [m^3/s]	50-yr [m^3/s]	100-yr [m^3/s]	Regional [m^3/s]
Kettle Creek above St. Thomas	49.80	99.20	126.30	147.10	168.30	N/A
Kettle Creek at Dalewood Dam	62.93	125.36	159.61	185.90	212.69	558.3
Kettle Creek at St. Thomas	113.40	174.20	200.00	217.60	233.80	930.2
Kettle Creek below WPCP	118.71	182.36	209.37	227.79	244.75	962.2

6.4.2 Regional Flow Estimates

Regional flow estimates within St. Thomas available from the hydraulic modeling undertaken for the purposes of regulation mapping were used in this assignment. The peak regional flows were extracted from the hydraulic model, and used in the present work (shown in Table 6-5).

7. Hydraulic Analysis

River hydraulic analyses completed in this assignment are summarized. The analyses include the reservoir storage elevation relationship (derived from the hydraulic model ready DEM), the hydraulic capacity of the control structure, an assessment of dam operations, dam breach analysis, and hydraulic modeling and inundation analysis. Each component is summarized below:

7.1 Reservoir Storage Elevation Relationship

The bathymetric survey and subsequent generation of the digital terrain model of the Dalewood reservoir, along with the Lidar topography allow derivation of the storage elevation relationship. Such relationships are used to quantify (and monitor) how much storage volume the reservoir holds at various elevations.

The storage elevation relationship was derived by determining the volume of water below a range of water surface elevations, ranging from the deepest portion in the reservoir to top of the valley slopes. Note that crest elevation of left and right embankments are 217.3 m and 217.0 m, respectively. Reported storage volumes having elevations above the crest of embankments have to be interpreted with caution, as the dam would be overtopped in such conditions. The storage elevation relationship for Dalewood Reservoir is presented in Figure 7-1.

7.2 Hydraulic Capacity of Dalewood Dam

Estimating hydraulic capacity answers the question whether the dam can safely pass its design flood without overtopping its control structures (i.e., deck and embankments) during design flood events. The necessary data for hydraulic capacity calculation is a topographic survey supplemented by direct measurements. The end product of the exercise is a rating curve that show a plot of river stage versus flow at the dam site. The calculations are made for a range of water levels up to the highest point on the dam (i.e., the top of the earthen embankment berms). The safety of the dam site is then evaluated (in part) by demonstrating the ability of the dam to safely pass the design flood without overtopping or otherwise causing damage to its structures.

For the Dalewood Dam hydraulic capacity calculations the following measurements were made during the site visit and topographic survey efforts:

Length of spillway	= 25.2 m (6.3 m/bay x 4 bays)
Crest EL of Concrete Spillway	= 214.52 m
Top of Deck EL	= 217.40 m
Top of Stop Logs EL	= 215.04 m (two stop logs on top of spillway at each bay)
Crest EL Left Embankment	= 217.3 m
Crest EL of Right Embankment	= 217.0 m

For the calculation of spillway capacity at Dalewood the broad crested weir equation, taking the following form, was used (Mays, 1999):

Q = C0 x L x H^1.5

where:

Q [m^3/s]	= Discharge over the weir
C0 [-]	= 2.1804, weir coefficient, metric units
H [m]	= Head of water over weir, measured from spillway crest to upstream water level

The discharge capacity calculations at Dalewood Dam were carried out for two configurations: i) two 10 inch stop logs placed on top of the concrete spillway, and ii) all stop logs removed from the concrete spillway. Figure 7-2 shows spillway capacity curves for conditions i) and ii) above.

The main findings of the spillway capacity analysis are as follows:

1. With both stop logs removed, the water surface elevation at the Dalewood Reservoir during peak 100-yr flood conditions is estimated as 217.0 m.
2. With both stop logs in place, overtopping of the deck and embankments is anticipated to take place during peak 100-yr flood conditions.
3. With both stop logs in place, the crest elevation of the existing embankment is reached during approximately a 10-yr flood event.

Freeboard, or the vertical distance from the design water level to the crest of the structure, is an important factor when evaluating hydraulic capacity at a dam site. According to the MNRF (2011a), default freeboard estimates are based on the length of fetch of the reservoir (i.e., open water over which wind can blow and generate waves). The larger the fetch, the larger the freeboard requirement. Minimum freeboard for all dams in MNRF (2011a) is 0.3 m, irrespective of fetch.

For Dalewood Dam, the maximum fetch is estimated at 700 m. For the fetch of 700 m, the default MNRF (2011a) freeboard is 0.5 m.

However, the default freeboard can be reduced based on a site specific evaluation of wind generated waves at the site of interest that could reasonably be anticipated. For the calculation of wind generated waves, the following conditions were used:

Length of reservoir fetch	= 700 m
Depth of reservoir during 100-yr flood	= 2.8 m
Design wind speed during 100-yr flood	= 23.4 m/s

The design wind speed of 23.4 m/s was taken as the 10-yr design wind speed for St. Thomas, as published in CAN/CSA S6-14, Canadian Highway Bridge Design Code 2014. Use of 10-yr wind speed jointly on top of the 100-yr flood is deemed appropriate for the present undertaking. A possible refinement in the design wind speed value is possible, by analyzing the observation at the St. Thomas Airport, and carrying a directional wind frequency analysis. Such refinement is believed un-necessary at this time.

The application of the above parameters to the standard SMB equations for wind generated waves (CEM, 2002) produce the following:

Significant wave height, Hm0	= 0.44 m
Peak wave period, Tm	= 1.73 s

Maximum wave run-up on a 2H:1V slope = 0.42 m above still water level

Based on the above calculations, the freeboard at Dalewood Dam can safely be reduced from 0.5 m (default by MNRF, 2011a) to 0.42 m (site specific analysis).

7.3 Assessment of Dam Operations

Dalewood Dam Operation, Maintenance and Surveillance (OMS) Manual (KCCA, 2009) suggests that that the two stop logs are left in place (in each bay) year round. There is an existing stop log lifting device that is presently operational (KCCA, personal communications).

KCCA confirms that the stop log lifting device has not been operated in recent times (approximately a decade). There is a high degree of confidence that the existing stop logs can be removed, but much less confidence that they can be put back in place. Watech (2018) inspection report also notes that placing the stop logs back in position would be challenging.

Presently, two stop logs are placed on top of the existing concrete spillway structure that raise the reservoir water level by some 0.5 m above the crest of the spillway. Two stop logs are permanently kept in place on top of the spillway.

The regional hydrology suggests that Kettle Creek is a flashy catchment that responds fairly quickly (meaning the flood water come quickly, but also recede quickly). Bathymetric survey of the headpond confirms that Dalewood reservoir is fully silted with sediment, and thus has limited practical ability to attenuate (i.e., reduce) peak flows. Given the quick response of the catchment with a reservoir without active flood control storage, changes in stop log operations cannot amount to significant reduction of adverse effects of flooding downstream of the dam.

Existing stop log lifter can be operated to lower or raise the water level in the reservoir during normal (sunny-day) conditions only.

7.4 Dam Breach Analysis and Incremental Analysis

As per the MNRF (2011a) guideline, dam breach analyses are carried out for sunny day (non-flood conditions, reservoir at normal levels), and peak flood breach (peak flood conditions, reservoir at its maximum level). In the evaluations, the dam structure is assumed to be breached by considering various failure mechanisms. In this assignment, the breach failure mechanisms considered are i) embankment breach, and ii) breach of a portion of the concrete spillway. Breaching a dam releases the impounded water downstream, which is then routed through the downstream study area, and its impacts assessed. The consequence of the hypothetical dam breach is evaluated according to the MNRF (2011a) rules. The breach consequence analysis is used as a proxy in determining the Hazard Potential Classification.

The Inflow Design Flood (IDF) is defined as the most severe inflow peak flood (peak, volume, shape, duration, timing) for which a dam and its associated facilities are designed. The IDF for a dam site could be determined from a table published in MNRF (2011a), or by assessing the most severe flood above which there would be no further incremental consequences.

In this report, the HPC and the IDF are determined through a procedure known as incremental analysis (MNRF 2011a). In such analysis a number of flow scenarios are assessed, with and without dam failure, which are then used to identify the flow where there is no longer any significant threat to loss of life, property, environmental and cultural – built heritage.

The incremental analysis methodology is carried out in this work, using the following general procedure:

1. Select a peak flood magnitude for the dam site.
2. Carry out hydraulic modeling using the selected peak flow magnitude and establish its inundation limits.
3. Develop a dam-breach hydrograph during i) peak flood, and ii) normal (sunny day) flows.
4. Carry out hydraulic modeling assuming dam-break occurs during i) peak flood conditions, and ii) normal (sunny day) flows, and establish corresponding inundation limits for each.
5. Compare inundation limits from 2) to inundation limits from 4); differences in losses from to 2) are defined as incremental losses.
6. Increase the peak flood magnitude and repeat steps 2) through 5), terminating once the incremental losses 5) vanish.
7. Select peak IDF as the lowest discharge for which incremental losses are eliminated.

Given the size of Dalewood dam, the minimum peak flow considered shall be the 100-yr flood.

The subsequent analyses determine the flow hydrographs resulting from the release of water during a hypothetical dam breach. The analyses consider dam breach during flood and sunny-day conditions. The generation of the breach hydrograph is achieved by entering the appropriate breach geometry within the HEC-RAS hydraulic model, and numerically simulating the release of stored water from the breaching process. The breach geometries considered are detailed next.

7.4.1 Breach of Concrete Spillway

A hypothetical breach of the concrete spillway is considered by assuming that a 6.8 m portion of the concrete spillway suddenly fails. This dimension is selected as being approximately one quarter of the length of the concrete spillway as measured from left to right of the downstream wingwall. Note that detailed drawings of the concrete spillway (or its internal joint details) are not available, so the 6.8 m breach width represents a best guess estimate.

The final bottom level of the breach is set at EL 213 m, as this is the elevation of sediment in front of the spillway (spillway toe extends below this elevation, but is entirely covered with deposited sediment). In other words, water available to be released downstream is only above EL. 213 m, thus coinciding with the bottom elevation of the breach. The breach formation time of 0.3 hrs is standard for breach application for concrete structures.

Final bottom width	= 6.8 m
Final bottom elevation	= 213 m
Breach formation time	= 0.3 hrs
Breach side slope x, xH:1V	= 0 (vertical)

Flood and sunny-day breach scenarios are simulated using the above breach properties.

7.4.2 Breach of Embankment Structure

Breach of the embankment structures is likewise considered in this work. Given the presence of the concrete cutoff wall running through the center of each embankment, the piping mode of failure is therefore not considered feasible. Consequently, the governing mode of failure considered at Dalewood Dam is overtopping of the embankments.

Different empirical relationships were considered in estimating the breach characteristics from a hypothetical overtopping failure. The empirical relationships from Froehlich (1995, 2008) were used to select breach characteristics, which depend on the crest elevation of the embankments, final bottom elevation of the breach, side slopes of the breach, and the pool level (and volume) at anticipated failure. For the embankment geometry at Dalewood, the following breach configuration was selected:

Final bottom width	= 28 m
Final bottom elevation	= 216.6 m
Breach formation time	= {2.0, 1.5, 1.0} hrs
Breach side slope x, xH:1V	= 1

Note that breach characteristics were reviewed for consistency by a geotechnical engineer on file responsible for preliminary slope stability assessment of the embankments (documented in the next section).

A sensitivity analysis was also carried out using different breach formation times, ranging from 1.0 to 2.0 hrs (meaning time from initial embankment overtopping to the formation of complete breach). Analysis suggests that the breach formation time is an extremely sensitive parameter and significantly affects the peak flow generated by the breach (the shorter the breach formation time, the higher the peak flow). In this work, the breach formation time is conservatively selected as 1.0 hrs.

7.5 Hydraulic Modeling and Dam Breach Analysis

This section of the report focuses on hydraulic modeling, and provides details on data and analytical tools used in the assessment. Hydraulic models are analytical tools that evaluate characteristics of movement of water over time and space. Simply speaking, hydraulic models determine water surface profiles for river reaches within a given study area.

7.5.1 Model Setup and Description

The Hydrologic Engineering Center’s River Analysis System (HEC-RAS) hydraulic model is used in this work. Use of the HEC-RAS model is considered standard engineering practice when carrying out hydraulic assessments within floodplain. In this work a 1D steady and un-stready variants of the HEC-RAS hydraulic model (v5.0.7) are used. The model is developed and maintained by the U.S. Army Corps of Engineers.

The HEC-RAS modeling system in this work is used for analysis of gradually varied flow and water surface profiles during flood and dam breach conditions. The HEC-RAS model captures sub-critical, super-critical and mixed flow regimes. Manning’s equation is used for calculating friction losses in the channel and the floodplain. The representation of the channel is facilitated by specification of cross sections along the reach under study, which is divided into three sections: left and right overbank areas, and the main channel. Manning's roughness coefficients are usually different in the overbank areas than in the main channel due to presence of vegetation (i.e., floodplain that is forested will have different friction characteristics than the main channel having a clay bottom).

Data inputs to HEC-RAS model include: i) cross sections with reach lengths between the sections, ii) hydraulic structures, such as the bridges, culverts, dams, etc., iii) Manning's roughness coefficients for the main channel and the overbank areas, iv) upstream inflows to the reach, and v) downstream water surface elevation as the boundary condition.

The hydraulic model ready DEM (previously documented) was used to extract model cross sections of the 1D hydraulic model (Figure 7-2). The spacing of cross sections in the model is in the order of 100 m, and extends from upstream of Dalewood Dam (covering the entire reservoir) to downstream of the St. Thomas Water Pollution Control Plant (the study area).

The geometry of the eight bridges crossing Kettle Creek within the study area were coded into the hydraulic model, along with the Waterworks Weir, and Dalewood Dam. The Dodd Creek tributary was not included in the hydraulic model, although the flows from the Dodd were included in the analysis.

Different variants of the HEC-RAS model were used (referred to as Model 1, Model 2 and Model 3), defined as follows:

1. 1D steady state model, used for the delineation of flood lines for 2, 10, 25, 50, 100, and Regional Storm conditions.
2. 1D un-stready model, used for generation of the breach hydrograph, and propagating it through downstream areas (slow computation times).
3. 1D un-steady model, using the reservoir, dam and few cross sections downstream of dam (used for rapid generation of dam breach hydrographs).

The Model 1 is used for mapping and delineation of the flood lines based on latest topographic surveys and Lidar data. Model 2 represents the comprehensive 1D un-steady model capturing all relevant dam breach processes (generation and propagation of the breach hydrograph). Model 3 is a trimmed version of Model 2 that is quick to execute, and able to rapidly generate breach hydrographs for a variety of scenarios.

In the analyses that were carried out in this assignment, all three variants of the HEC-RAS were used. The Model 1 variant is most conservative as it assumes that peak flows last indefinitely, and thus tends to overestimate flood elevations. The Model 2 variant does not suffer from the same conservatism, as it is able to propagate the flood hydrograph through the downstream floodplain.

Completing various sensitivities using above models revealed the following: Generating dam breach peak flows using Model 3, and simply adding the computed peak breach flows to steady state flows of Model 1, produced reliable, and conservative results. The results of combination of Model 3 and Model 1 compared favourably with results generated with Model 2. As results of Model 3 and Model 1 were more conservative, they were used in the remaining analysis.

7.5.2 Dam Breach Assessment

For generating dam breach hydrographs Model 3 (described above) is used. Results of the breach modeling were as follows:

Breach of Concrete Spillway

Sunny-day flow = 1 m^3/s (estimated)
Dam breach during sunny-day = 26.9 m^3/s (more than half of 2-yr flows)

100-yr flow = 212.69 m^3/s
Dam breach during 100-yr flow = 212.69 m^3/s

Note that as the geometry of the breach opening (6.8 m wide below the top of the spillway) is simply too small to allow for a much greater release of water from the reservoir during flood conditions. The opening of the breach would have to be in the order of 20-30 m wide (i.e., the entire concrete spillway would have to give way) for a substantial breach flow to be generated during flood conditions. Given that the existing concrete spillway consists of mass concrete, breaching of the entire concrete spillway is unlikely. Follow up structural assessments (to be carried out in subsequent steps) may alter/refine this finding.

From the above results it is readily observed that the breach of the concrete spillway (with the assumption that one quarter of the spillway gives way in 0.3 hrs) will likely not govern.

Breach of Embankments

Sunny-day flow = 1 m^3/s (estimated)
Dam breach during sunny-day {1.0, 1.5, 2.0 hr} = {11.4, 9.9, 9.1} m^3/s

100-yr flow = 212.69 m3/s
Dam breach during 100-yr flow {1.0, 1.5, 2.0 hr} = {274.82, 259.39, 251.31} m^3/s

Where the values {1.0, 1.5, 2.0 hr} represent different breach formation times. For the purposes of this report, the breach hydrograph having the breach formation time of 1 hr shall be used.

Based on the analyses carried out, the governing breach case is the embankment breach during flood flow conditions. Given the limited storage in the reservoir (from a century of sediment accumulation) the sunny-day breach is thus excluded from further consideration.

7.5.3 Flood Inundation Mapping

Breach Inundation Mapping

For the evaluation of incremental losses associated from a hypothetical dam breach, inundation limits from a 100-yr flood profile are compared to inundation from an embankment breach on top of a 100-yr flood. The resulting comparisons of inundation limits are shows in Figure 7-3 (entire study area), with close-ups in Figure 7-4 (at Parkins Avenue), and Figure 7-5 (at Sunset Drive).

Based on the inspection of the results it is evident that the incremental inundation (associated with a dam breach) is limited to inundating a nursery (0.25 m increase) and an accessory structure (less than 0.1 m increase) on Parkins Avenue, and one accessory structure on Sunset Drive (0.1 m increase).

Floodplain Mapping for 2-yr to Regional Storm Profiles

The steady state HEC-RAS model (variant Model 1) was used to determine the inundation limits associated with the following profiles: 2, 10, 25, 50, 100-yr and Regional Storm conditions. Figures 7-6 to 7-11 show the inundation limits associated with the above profiles.

In the hydraulic modeling carried out to delineate downstream floodplain, an assumption was made that all stop logs are removed from the dam during flood events. Due to limited flood control storage in the reservoir, flood flows at Dalewood cannot practically be attenuated. In other words, there would be little difference to downstream inundation extent whether stop logs are left in place, or are removed.

7.6 Verification of the Hazard Potential Classification

The computed incremental flood depths, together with the fact that residential structures are not inundated from incremental flooding associated with a dam breach, supports the conclusion that a LOW HPC of Dalewood Dam can be supported.

7.7 Inflow Design Flood

Based on the above HPC ranking of LOW, the Inflow Design Flood at Dalewood Dam is defined as the 100-yr flood, or 212.69 m^3/s.

8. Preliminary Geotechnical Analysis

This section documents the results of the preliminary slope stability analysis of the left and right embankments. In the text that follows, a desktop assessment of the slope stability is made based on observations (summarized in Section 3), review of existing borehole information, and professional judgment.

8.1 Review of Existing Boreholes

KCCA has provided existing borehole information from the recently completed Dalewood Bridge re-construction project. Existing boreholes are located adjacent to the bridge abutments on the north and south side of the crossing. Knowledge of local geology and stratification available from existing boreholes have been assembled, and used in the preliminary assessment of slope stability.

The stratigraphy at the Dalewood Dam site is inferred to comprise of the following:

- Brown Clay (soft), reservoir bottom to EL 212 m
- Grey clay (soft), EL 212 m to EL 210 m
- Sand (loose), EL 210 m to EL 206
- Sand (dense), beyond EL 206 m

Composition of the existing embankments is not presently known, but is inferred to consist of the clayey materials, and likely mixed with construction fill.

8.2 Preliminary Slope Stability Assessment

The preliminary slope stability assessment of the embankments is evaluated using the following assumptions, geometry and river hydraulic information:

- Left embankment: Crest EL at 217.3 m, 2H:1V upstream side slope, 2.5H:1V downstream side slope
- Right embankment: Crest EL at 217.0 m, 1.5H:1V upstream side slope, 2H:1V downstream side slope
- 100-yr flood conditions with reservoir at EL 217.0 m, and tailwater at EL 211.97 m
- Sunny day conditions, with reservoir at EL 215.15 m and tailwater at EL 209.0 m
- Five days to fill the reservoir to EL 217.0 m (100-yr flood),
- One day to lower the reservoir to EL 215.15 m (normal water level with two stop logs in place)
- Embankment constructed of clayey materials,
- Concrete cut-off wall intact, 0.3 m wide, running full height of the embankment

Based on the available borehole information, regional geology, and professional judgment, a preliminary slope stability evaluation for the left and right embankments revealed the following:

- Downstream slopes may have slope stability safety factors less than currently accepted engineering standards of 1.5
- Flashy nature of the Kettle Creek at Dalewood Dam implies that water level in the reservoir can recede to normal operating levels within a day following a flood event
- Such rapid lowering of the reservoir can lead to rapid drawdown conditions for the upstream face of the embankments
- Factors of safety for rapid drawdown conditions are estimated as less than unity, indicating failure conditions (factors of safety are a function of embankment composition and rate of drawdown)
- Given uncertainty in the composition of the embankment, impact of rapid drawdown conditions are unknown (more permeable material than assumed may result in relatively higher safety factors)
- Remedial works at the embankments may be required to restore slope stability to current standards
- Completing borehole investigations through the embankment is required to assess slope stability, and to determine the extent of and design remedial works, should these be required.

9. Public Safety Assessment

In accordance with provincial regulations (MNRF, 2011b), dam owners are responsible for the safe operation and maintenance of their dams. Part of the safe operation of the dams is a responsibility to implement appropriate public safety measures to address potential exposure to hazards created at the dam site.

The present public safety assessment will identify public safety features at the dam site (fencing, signage, etc.), list sanctioned public activities surrounding the dam site (such as fishing, walking, boating, etc.), and evaluate hazards associated with sanctioned public activities. The public safety measures are typically implemented to eliminate or mitigate public's exposure to hazards including physical barriers, operating controls, warning systems, signage and initiatives intended to raise public awareness and understanding of the associate hazards (MNRF, 2011b).

A photographic log of public safety features at Dalewood Dam is provided in Appendix A. Activities sanctioned at the dam site are noted, existing safety measures described, and an initial hazard evaluation completed.

9.1 Activities

The Dalewood Dam is situated at the center of a recreational hub for the City of St. Thomas and surrounding communities, bordered by a campground, conservation area hiking trail, frisbee disc golf course and public day use area. The head pond is used for fishing and a canoe/kayak access was recently installed north of the Dam and the Dalewood Bridge. While not sanctioned by KCCA or the City, the public have created a path from the Dalewood Road Day-Use parking lot to the left downstream side of the dam by way of the lower access road. It would appear the public use this access as a way to connect to Waterworks Park, located about 1 km downstream and possibly the disc golf course to the south of the dam site. The trail is not maintained or sanctioned by KCCA or the City of St. Thomas.

Noted during site inspections were unsanctioned use of the right downstream bank, immediately inland of the existing training wall. The access to the right downstream area is via the crest of the right embankment, then along a small trail running along the downstream face of the right embankment. The said area is bordered by the river on the south, existing valley walls to the north and west, and the downstream embankment to the east. Evidence of past camp fires were visible.

9.2 Existing Safety Measures

Recently installed steel gates were visible on the driveway entrances at left and right embankment, meaning vehicular access to the dam and downstream areas are restricted. The Dalewood Dam control structure is entirely fenced with a 2 m high fence, enclosing the deck, and up- and downstream wingwalls. Safety signage is noted on the main access gates warning residents of the dangers associated with fast flowing water at the dam site.

Access to the deck at the control structure is restricted and fully enclosed with fencing. Handrails are noted on up and downstream sides of the deck.

On the left upstream abutment of Dalewood Road bridge there is a safety sign warning of the dam ahead.

Safety booms are not present on the upstream side of the dam structure.

Historic knowledge suggests there were no previous deaths, accidents or safety related incidents in the vicinity of Dalewood Dam.

9.3 Hazard Evaluation

9.3.1 Reservoir

The recreational activities in the Dalewood Reservoir include fishing, walking, biking, canoeing/kayaking, hiking, and possibly ice fishing. Riprap protection for the left and right abutments at Dalewood Bridge provide easy unsanctioned access to the reservoir. In the area of the bridge abutments the bank consists of large sized riprap and is relatively steep, with high probability of someone slipping on rocks and falling in the reservoir, and could be swept towards the main spillway of the dam.

Access to the riprap slopes at the existing bridge abutments is recommended to be restricted to avoid the above noted hazards. Appropriate signage along the sidewalk in the vicinity of the bridge abutments should be installed to warn users of dangers associated with fall hazards in close proximity of the reservoir and dam.

It is recommended that KCCA works with the City of St. Thomas and restrict access to the existing riprap slopes at the bridge abutments at the Dalewood Reservoir. Restricting access to the riprap slopes at the existing bridge abutments will avoid someone falling on the rocks and/or slipping into the reservoir upstream of the dam.

KCCA should inform City of St. Thomas of the fall hazards identified at the Dalewood Bridge abutments, and work with City staff in addressing the noted hazards.

Ice fishing potentially occurs in the headpond, and should be discouraged due to unknown ice thicknesses. The hazards of ice fishing include falling though the ice and possibly drowning. KCCA should post signs at access points in the head pond noting that ice fishing is strictly prohibited.

9.3.2 Control Structure

The control structure is fully enclosed with fencing, with controlled access provided by KCCA staff. Handrails exist on top of the deck, on both up and downstream sides. Signs are noted at both left and right access points.

There are presently no safety related issues at the control structure.

9.3.3 Embankments and Downstream Areas

There is an unsanctioned trail on the left bank at Dalewood Dam that eventually connects with the trail network at Waterworks Park. The trail is accessed via Dalewood Road, and runs along the left bank. Presently, access to the trail exists along the toe of downstream slope of the left embankment, running past the area downstream of the control structure, and eventually continuing through the forested area downstream.

Presently there are no safety related signs on the trail warning users of dangers from close proximity to the dam structure and its fast flowing water. The unsanctioned trail is not maintained.

It is recommended that safety signage be installed at key entrance points on the unsanctioned trail, to properly warn users of the dangers, particularly during flood events (which can happen quickly with little warning).

An access path was noted to the area downstream of the right embankment, on the flat area inland of the training wall. Unsanctioned activities in this area included camp fires, walking, fishing from the training wall, etc. Since the area downstream of the right embankment is low relative to the water and fully enclosed (by river, valley walls, and embankment), together with the fact that Kettle Creek is a flashy catchment, flood waters can arrive quickly possibly stranding users in a known area of fast flowing water.

The area downstream of the right embankment is recommended to be fully restricted, by installing fencing along the crest of the embankment and placing appropriate safety signage.

10. Key Study Findings

10.1 Scope of Work

The scope of the present report is to undertake necessary investigations to establish the Hazard Potential Classification of Dalewood Dam. The work undertaken included:

- A review of previous reports and data collection efforts
- A site visit by hydrotechnical and geotechnical engineers
- Topographic and bathymetric data collection efforts
- A preliminary hazard potential classification analysis
- Digital terrain analysis using collected and Lidar data sets
- Hydrologic analyses and establishing flow characteristics
- River hydraulic modeling, dam breach analysis, inundation modeling, and floodplain mapping
- A preliminary geotechnical analysis of earthen embankments
- A public safety assessment

10.2 Key Findings

The key findings of this work include the following:

Hydrotechnical:

- Preliminary Hazard Potential Classification analysis suggests that Dalewood Dam can be categorized as LOW according to MNRF (2011a) guidelines

- The Inflow Design Flood at Dalewood Dam is defined as the 100-yr flow
- The reservoir at Dalewood Dam is noted as being fully silted with sediment, with no practical flood control storage
- Incoming river sediment is flushed downstream by flood events on an annual basis, with no accumulations taking place
- The amount of deposited sediment in the reservoir is the same as reported in 1980 (the last time a reservoir study was completed),
- Flood flow frequency estimates were lower than previously reported in Latham (1987) study (resulting from analysis of a more comprehensive stream flow record spanning 50 years)
- Dalewood Dam is operated by placing two 10 inch high stop logs in each of its four bays year round
- The control structure can pass the 100-yr flood without overtopping of the deck if all stop logs are removed in advance of the flood
- With stop logs left in place, the crest elevation of existing embankments is reached during approximately the 10-yr flows
- The left and right embankments do not have adequate freeboard in passing the 100-yr flood
- Breach of the entire concrete spillway is considered unlikely (as it consists of mass concrete)
- Assumed breach of a ¼ of the concrete spillway does not produce significant downstream inundation during sunny-day or flood conditions
- Breach of a 28 m section of earthen embankments is considered during sunny-day and flood conditions
- Embankment breach during flood conditions is identified as the governing case
- Incremental flood extent analysis associated with an embankment breach has identified to inundate an area of former nursery (0.25 m depth increase beyond 100-yr flood elevation), an accessory structure at Parkins Avenue (less than 0.1 m depth increase on top of 100-yr flood), and one accessory on Sunset Drive (0.1 m depth increase on top of 100-yr flood)
- The incremental analysis supports that Hazard Potential Classification rating of Dalewood Dam be LOW

Geotechnical (preliminary):

- Preliminary geotechnical slope stability analysis suggests downstream embankment slopes may have safety factors less than current standards
- Available borehole information from previous projects does not provide details on composition of the Dalewood embankments
- The flashy nature of Kettle Creek (rapid rise and fall of water levels) may lead to rapid drawdown conditions, potentially putting upstream embankments at risk
- Future remedial works of the embankments will be required

- Borehole investigations at the dam site are required to assess slope stability, and define the scope of future remedial works

Public Safety:

- Access to the riprap slopes at the existing Dalewood Bridge abutments is recommended to be restricted
- Restricting access to the riprap slopes avoids users of the sidewalk along Dalewood Road from falling in the reservoir upstream of the dam
- Ice fishing activities should be discouraged in the reservoir, with clear signs posted at access points
- Access to the paths on the left bank at Dalewood Dam requires safety signage to warn users of dangers during flood events
- Local users are accessing a low lying area downstream of the right embankment that is bounded by river, valley walls and embankment
- Given that Kettle Creek is a flashy catchment (flood waters arrive quickly) a possibility exists that users of the area could become stranded during in an area of fast flowing water
- The area downstream of the right embankment is recommended to be fully restricted, by installing fencing along the crest of the embankment and/or placing appropriate safety signage

11. Recommendations

The text that follows adopts an assumption that KCCA wishes to continue operating the Dalewood Dam for the purposes of achieving the environmental benefit offered by the upstream headpond. In order to realize this benefit, two stop logs are placed on top of the existing concrete spillway structure that raise the reservoir water level by some 0.5 m above the crest of the spillway. Conditions of the present infrastructure at the dam site (i.e., the existing stop log lifter) does not allow KCCA to change position of the stop logs. Two stop logs are permanently kept in place.

Present hydrotechnical investigations identified that present day Dalewood Dam, operating with two permanent stop logs in place, does not meet MNRF guidelines for freeboard and hydraulic capacity (i.e., ability to safely pass the Inflow Design Flood). Changes to dam operations, and/or remedial repairs will be required in the near future.

The scope of the present report was restricted to hydrotechnical (assessment of flows, and evaluation of downstream flood risk from a potential dam breach), preliminary geotechnical slope stability analyses of the embankment berms (desktop analyses only, no intrusive investigations), and public safety analyses (fencing, signage). Given the present scope, a comprehensive action plan for future repairs cannot be defined without the benefit additional investigations, some of which may include those outlined in the following sections.

11.1 Terrestrial and Aquatic Assessment

An appropriately defined study quantifying the environmental benefit of the existing water level in the reservoir (stop logs in place) is recommended to be carried out. The same study should also be tasked with the quantifying environmental conditions in case the stop logs were permanently removed. The impact of the changing water level in the headpond would be quantified.

If it can be demonstrated that removal of the stop logs would not create significant adverse impact to the upstream environmental features (and its terrestrial and aquatic species), then scope of the future maintenance works may be reduced (i.e., if stop logs are not required for environmental benefit, then there would be no need for a functional stop log lifter).

A possible outcome of an environmental study could be that significant environmental benefit exists to maintaining present day water levels (i.e., stop logs left in place). Should significant environmental benefit exist, then an argument can easily be formulated for keeping the stop logs in place. In that case an operating stop log lifter will be required, as would an operating plan stating when the stop logs should be removed, and when they should be placed back in.

Regardless of the outcome of an environmental study, future maintenance will be required to at the Dalewood Dam, regardless of the desired water level in the headpond.

11.2 Geotechnical Investigations and Slope Stability Assessment

A geotechnical investigations at Dalewood Dam is required to assess the composition of its embankments and complete slope stability evaluations. The investigations would require obtaining boreholes through the existing embankments, and obtaining soil stratigraphy into the underlying clay layers below. Interpretation of the boreholes by a geotechnical engineer is required, as are slope stability analyses. Such analyses will establish the safety factors of the embankment slopes, given the results of the hydrotechnical analyses. The obtained safety factors are required to be compared with safety factors required by Provincial guidelines. Key question to be answered in the geotechnical follow-up studies are the following: can the existing earthen embankments at the dam be modified to safely withstand the passage of the Inflow Design Flood (calculated in the present study)

Depending on the results of the investigations, some degree of repairs works should be anticipated for the earthen embankments.

11.3 Structural Stability Assessment

Following geotechnical soil investigations, a comprehensive structural stability assessment of the dam and its components (i.e., spillway, piers, deck, wingwalls) are recommended to be carried out. The soil parameters estimated from geotechnical analysis and investigations will inform soil resistance values required for structural evaluations, along with water loading information obtained from the hydrotechnical analysis. Structural stability safety factors are to be calculated according to standard practice, and compared against Provincial guidelines. Key question to be answered in the structural follow-up studies are the following: can the existing control structure safely withstand the loading during the passage of the Inflow Design Flood.

Following the completion of structural inspections and evaluations of all key features at the dam site, a repair and maintenance schedule (with priority rating of repairs) is anticipated be identified.

11.4 Identification of Repair Alternatives

Following results of the geotechnical and structural investigations, along with the results of the present hydrotechnical analyses, a number of alternative options are anticipated to be developed so that Dalewood Dam can safely pass the Inflow Design Flood while meeting required MNRF guidelines for safety. Each option would require evaluation according to set criteria (to be developed with KCCA staff). Highest ranked alternatives would be subject to detailed design.

11.5 Plans, Specifications, Tendering, Construction

Upon identification of the best ranked alternatives, detailed design would commence, ultimately leading to the preparation of tender ready plans and specifications. Tendering, selection of contractor, and construction would follow.

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All of Which is Respectfully Submitted,

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