

# Chase River Dam Breach Flood Inundation Study



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# **Executive Summary**

# 1 OVERVIEW

The Middle and Lower Chase River Dams are located on the Chase River in the southern half of the City of Nanaimo. Both dams were constructed circa 1911, to provide reservoirs for the storage and withdrawal of wash water for the expanding coal mining industry in the Nanaimo area. After 1945, both dams were no longer used for water supply due to the cessation of coal mining in the area. In 1975, the two dams came under the ownership and control of the City of Nanaimo. Colliery Dams Park was established in the area around the dams, and the reservoirs and park are used for recreation, including hiking, some sport fishing, and passive activities.

Downstream of the Lower Dam, the Chase River ravine opens out into a generally flat floodplain, where the river channel is only slightly incised. This area extends roughly from Sixth Street to Eighth Street, north to south, and Howard Avenue to Park Avenue, west to east. Since the construction of the dams this area has urbanized and is now largely occupied by single family residential homes, but a large school (John Barsby Community High School) and a private daycare operation (Little Ferns Daycare) are also present only a short distance from the Chase River. Refer to Figure 1-1 (all figures are included in Appendix A) for an overview map of the study area. Crossings of the Chase River are located on, from upstream to downstream, Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue.

Currently, both structures are classified as "Very High Consequence" under BC's Dam Safety Regulation, due to the population present in the downstream floodplain areas. Previous reports have identified that neither dam meets the expected level of performance for flood or seismic safety, and both are considered to be at a level of risk outside the envelope of acceptable risk using generally applied standards (EBA, 2010).

Three general approaches are available to the City to address these issues:

- Complete removal of the dams.
- Partial mitigation of risk factors and acceptance of a higher degree of risk by the City's stakeholders.
- Extensive mitigation of risk factors to bring them into compliance with the BC Dam Safety Regulation and Canadian Dam Association's Dam Safety Guidelines.

As part of the decision process to plan the future of the dams, this flood inundation study is required to estimate the areas that are at risk under various credible dam failure scenarios, both flood and seismic. Estimates of the damage value and potential for loss of life are necessary to inform the City's stakeholders (primarily residents of the floodplain) and arrive at an acceptable plan for the City to achieve a desired level of safety, balanced with the recreational and aesthetic values provided by the dams.



This report has considered several different events that are expected to result in the failure of the dams. Two main failure modes exist, flood induced dam failure, and failure during a seismic event (i.e. earthquake). Risk factors and potential modes for dam failure are discussed in Sections 2 and 3 respectively.

### 2 FLOOD INDUCED DAM FAILURE

Both dams have limited spillway capacity, as assessed and reported by Water Management Consultants (WMC) in their "Middle and Lower Chase River Dams Spillway Hydrology Study" of April 2002. During floods exceeding the respective spillway capacities, overtopping of the dam crest and flow down the outer surface of the dams would occur. Overtopping would lead to erosion of the dam faces, which if sustained, would lead to loss of support to their internal concrete cores and their likely collapse.

The Lower Dam spillway capacities are considerably less than the estimated design flows for the probable maximum flood (PMF), and the 1000-year and 100-year return period floods. However, the Middle Dam spillway capacity is sufficient for the estimated 100-year return period flood event, but insufficient for the 1000-year return period flood failure scenarios were assessed:

- .1 100-year Return Period Flood Event Overtopping failure of the Lower Dam only.
- .2 1000-year Return Period Flood Event Overtopping failure of both the Middle and Lower Dams in a cascade sequence, where the Middle Dam fails first leading to the immediate failure of the Lower Dam. The Middle Dam breach and Lower Dam breach hydrograph peaks are assumed to coincide.
- .3 PMF Event Overtopping failure of both dams in a cascade sequence, where the Middle Dam fails first, leading to the immediate failure of the Lower Dam. The Middle Dam breach and Lower Dam reach hydrograph peaks are assumed to coincide.

Based on our floodplain modelling and mapping, presented in Appendix A, very little time is available to notify and evacuate the public at risk once overtopping of one or both of the dams begins. For many areas, significant flooding or increased flood depth occurs within 15 to 30 minutes after the start of dam failure. We note that the dams may be overtopping for some time prior to failure. However, we believe that it may be difficult to effectively notify all potentially affected residents and allow sufficient time for evacuation.

A reliable flood-forecasting and monitoring process would be essential to identify the potential occurrence of a flood related dam failure before it occurs in order to provide adequate time for orderly evacuation of the floodplain. As with any forecasting tool, there is a possibility of generating false alarms, which may reduce its effectiveness with the public over time, and lead to a delayed response should a genuine dam breach situation develop. More notably, such a forecasting tool would provide no benefit in increasing the warning time with respect to a seismically induced dam failure.

## 3 SEISMIC EVENT INDUCED DAM FAILURE

Based on EBA's 2010 Seismic Assessment, we consider the most relevant seismic (or "sunny day") scenario to be an earthquake of sufficient magnitude to cause the Middle Dam concrete central core to break and topple upstream into the reservoir. The Middle Dam then rapidly breaches due to the resulting loss of crest height and overtopping flow. The breach of the Middle Dam leads directly to the overtopping and breach of the Lower Dam.

A seismically generated dam failure has particular importance, as EBA was clear that an event considerably smaller than the design event could lead to a failure in this manner. Given the uncertainties regarding the stability of the dam fill and concrete core wall of the Middle Dam in the face of a seismic event, EBA could not conclusively define the seismic event that would initiate this failure mode. EBA estimated that a seismic event as small as a 100-year return event (probability of 40% in a 50 year period) ranging to a 475-year event (probability of 15% in a 50 year period) could be sufficient to initiate a failure of the Middle Dam. The possibility of a dam failure associated with these relatively small and frequently occurring seismic events is not a certainty, but represents a significant cause for concern.

### 4 CONSEQUENCES

The potential consequences of each of the dam failure scenarios are discussed in detail in Sections 5 and 6.

Without a flood-forecasting tool, we estimate that the number of fatalities resulting from a flood event induced failure of one or both of the dams may be in the range of 30 to 60 (refer to Section 6). This number will be influenced by the timing of the dam failure after the onset of dam overtopping, the amount of warning provided to residents prior to flooding and the amount of time available for evacuation. Incremental (i.e., above and beyond "reference" non-dam breach flooding scenarios) economic damages in the range of \$33 to \$36 million dollars were estimated for the more significant flood induced dam failure scenarios (refer to Table 5-2)

We estimate that the seismically induced dam failure could have total fatalities in the range of 80 (daytime) to 150 (nighttime) people (refer to Section 6). This number will be influenced by the severity of the earthquake, the time of day, degree of structural damage resulting from the earthquake and the number of people trapped by structural damage. We note that the accurate estimation of casualties is particularly difficult as a result of the many factors that influence the outcome. Economic damages in the range of \$38 million dollars were estimated for the seismic event induced dam failure scenario (refer to Table 5-2)

Many variables affect the population at risk including time of day and time of year (season). With respect to the potential number of causalities the single most important factor is the amount of warning provided to the public prior to flooding, as well as the time required to evacuate the area.



Therefore, in our opinion, the event of greatest concern is the seismic failure of the dams as there would be virtually no warning or time for self-initiated or directed evacuation of the public within the affected inundation areas. This would likely result in many casualties.

### 5 CONCLUSIONS

The Chase River Dams present a significant risk to downstream areas in the event of a dam failure. The most significant risk results from a seismically induced dam failure. We note that a 100-year return period seismic event is estimated to result in significantly higher number of causalities than the Probable Maximum Flood (PMF), which has an approximately probability of a 10,000-year return period event. However, economic damages for the PMF event are about 25% higher than the seismic event.

Various approaches are available for reducing the consequences of failure resulting from a dam breach. These include flood-forecasting tools, educational programs for residents within the floodplain area, evacuation plans, and evacuation notification systems. These approaches would be expected to reduce the potential casualties for the events considered. However, these approaches would not reduce economic losses or significantly reduce the estimated casualties resulting from a high-probability seismically induced dam failure. As a result, we conclude that other means of risk mitigation must be utilized.

The probability of failure combined with the consequences of failure present a compelling case for modifying the dams. The possible modifications, assuming that the 'do-nothing' option is unacceptable, are as follows:

- The upgrade or replacement of the existing dams.
- The removal of the existing dams.

We note that this report does not account for the significance or value of existing environmental, social and cultural assets within the floodplain area.

The economic losses identified in this report are based on the values of assessed structures within the floodplain area as provided by the City of Nanaimo. An allowance was made for building contents which was based on a percentage of the structure value. The contents allowance includes one vehicle.

The economic losses exclude damage to public or privately owned utilities, consequential losses, and clean-up of the floodplain area after a dam failure.

Based on the foregoing, we conclude that:

• The estimated number of casualties resulting from the seismic event requires the consequence classification of the Lower and Middle Dams along the Chase River to be increased from 'Very High' to 'Extreme' based on the British Columbia Dam Safety Regulation. The 'Extreme' classification is the highest consequence classification under the British Columbia Dam Safety Regulation.

- As a consequence of the uprating of the dams from 'Very High' consequence structures to "Extreme" consequence structures, under the Provincial Dam Safety regulations the appropriate inflow design flood (IDF) for these dams is now the probable maximum flood (PMF). As discussed in Section 2, neither dam has adequate spillway capacity for the PMF. In addition, the difficulties in providing sufficient spillway capacity for the PMF were identified in WMC's report of April 2002.
- A flood related dam failure, as a result of overtopping, would require a flood-forecasting tool and/or a surveillance system in order to provide adequate time for notification and evacuation of residents. Evacuation alerts and orders would have to be issued prior to the onset of overtopping in order to provide adequate time for evacuation. Depending upon the time of day, this could be problematic. Also, a seismically driven dam failure, which we believe is the greatest risk, would not be covered by such a system.
- A seismic event leading to the failure of the dams could have as high as a 40% probability of occurrence during a 50-year time period.
- A seismic event leading to the failure of the dams is a major concern as notice cannot be provided to the public and inadequate time would be available for self-initiated or directed evacuation of most residents. The estimated number of fatalities is in the range of 80 (daytime) to 150 (nighttime) people.
- Based on current development and estimated population distribution, the estimated direct economic damages are \$38 million for a seismically induced dam failure.
- Incremental direct economic losses to structures and contents resulting from a flood driven failure of both dams, based on assessment building values, are estimated in the \$33 million to \$36 million range depending on the scenario. We note that replacement costs may be higher.
- Based on current development and estimated population distribution, the estimated number of casualties as a result of a flood related dam failure is in the range of 30 to 60 people.

Based on the above conclusions, we recommend that the City of Nanaimo select one of the following two options:

- Rehabilitate or replace the existing Lower and Middle Dams to meet the current Dam Safety requirements.
- Remove the existing Lower and Middle Dams.



# **Table of Contents**

SEC	TION	PAGE NO.	
<b>F</b>			
Exec	utive St	ummary	I
Table	e of Cor	ntents	i
1	Intro	duction	1-1
2	Dam	Configuration and Risk Factors	2-1
	2.1	Seismic Assessment	2-2
	2.2	Flood Event Risks	2-2
3	Pote	ntial Failure Modes	3-1
	3.1	Flood Events	3-1
	3.2	ldf Hydrographs	3-2
	3.3	Reservoir Routing and Dam Breach Assessment	3-2
	3.4	Seismic Event	3-6
4	Floo	dplain Model Development and Setup	4-1
	4.1	Software Selection	4-1
	4.2	Chase River Channel	4-2
	4.3	Chase River Floodplain	4-3
	4.4	Coupled Model	4-3
5	Failu	5-1	
	5.1	Flooding	5-2
	5.2	Property Damage Estimation	5-4
	5.3	Flood Warning	5-10
6	Sum	mary	6-1
7	Cond	clusion	7-1
8	Glos	sary of Terms	8-1



9 Certification

Appendix A - Figures

Appendix B - Upper Chase River Dam

# Introduction

The Middle and Lower Chase River Dams are located on the Chase River in the southern half of the City of Nanaimo. Both dams were constructed by Western Fuel Company, circa 1911, to provide reservoirs for the storage and withdrawal of wash water for the expanding coal mining industry in the Nanaimo area. Wash water was piped from the dams to the coal docks located in south Nanaimo.

After 1945, both dams were no longer used for water supply due to the cessation of coal mining in the area. In 1975, the two dams came under the ownership and control of the City of Nanaimo. Colliery Dams Park was established in the area around the dams, and the reservoirs and park are used for recreation, including hiking, some sport fishing, and passive activities. Originally the dams were managed by the Parks and Recreation Department but responsibility was later transferred to the City's Engineering and Public Works Department.

Downstream of the Lower Dam, the Chase River ravine opens out into a generally flat floodplain, where the river channel is only slightly incised. This area extends roughly from Sixth Street to Eighth Street, north to south, and Howard Avenue to Park Avenue, west to east. Since the construction of the dams this area has urbanized and is now largely occupied by single family residential homes, but a large school (John Barsby Community High School) and a private daycare operation (Little Ferns Daycare) are also present only a short distance from the Chase River. Refer to Figure 1-1 (all figures are included in Appendix A) for an overview map of the study area. Crossings of the Chase River are located on, from upstream to downstream, Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue.

The Chase River re-enters a ravine setting downstream of Park Avenue. The river then passes under the Esquimalt and Nanaimo (E&N) railway embankment via a concrete arch culvert, and then a culvert at Aebig Road. A large concrete box structure is encountered at the Island Highway, and a small trestle on the E&N spur line to Wellcox. The Chase River then discharges to the Nanaimo/Chase River Estuary at the south end of Nanaimo Harbour.

The City has undertaken regular reviews of the dams' condition, safety and operations. In 2003, Golder Associates completed the most recent dam safety reviews of these dams for the City. These reviews highlighted specific concerns with the capacity of the dams to withstand seismic events and safely convey flood flows. Flood flows were previously defined by a Water Management Consultants hydrology study in 2002, which indicated inadequate spillway capacity for both dams. Seismic concerns were investigated in EBA's seismic safety investigation of 2010. The identified concerns reflect the age of the dams, their design, and the practices and materials used in their construction.

Currently, both structures are classified as Very High Consequence, due to the population present in the downstream floodplain areas. Neither dam meets the expected level of performance for flood or seismic safety, and both are considered to be at a level of risk outside the envelope of acceptable risk using generally applied standards (EBA, 2010).



Three general approaches are available to the City to address these issues:

- Complete removal of the dams,
- Partial mitigation of risk factors and acceptance of a higher degree of risk by the City's stakeholders, or
- Extensive mitigation of risk factors to bring them into compliance with the BC Dam Safety Regulation and Canadian Dam Association's Dam Safety Guidelines.

As part of the decision process to plan the future of the dams, this flood inundation study is required to estimate the areas that are at risk under various credible dam failure scenarios, both flood and seismic. Estimates of the damage value and potential for loss of life are necessary to inform the City's stakeholders (primarily residents of the floodplain) and arrive at an acceptable plan for the City to achieve a desired level of safety, balanced with the recreational and aesthetic values provided by the dams.

A secondary application of the flood inundation mapping is to support the development of emergency response plans to address the scenario of one or more of the Chase River dams failing.

2

# **Dam Configuration and Risk Factors**

Both the Middle and Lower Chase River Dams were constructed circa 1910/1911, making them 100 years old. Construction practices of that period are one of the major factors in the degree of concern regarding the safety of these structures from both a seismic and flood risk perspective.

Each dam is composed of an internal concrete core, with a shell of fill material placed on either side of the core. The Middle Chase River Dam is 13 m high at the ravine thalweg (deepest point), with a crest span of 50 m from bank to bank and a crest thickness of 5 m. The concrete core is 0.6 m thick. The Lower Chase River Dam is 24 m high at the ravine thalweg, with a span of 77 m and crest thickness of 10 m. The concrete core is 1.2 m thick.

EBA's "Seismic Hazard Assessment Middle and Lower Chase Dams" of April 2010 provides an extensive discussion of each dam's construction details, including identification of a number of uncertainties regarding their structure. EBA considered the following to be probable elements of their construction:

- The concrete cores are almost certainly unreinforced, so the concrete will have limited resistance to shear and bending moment forces.
- Concrete quality and construction of that time were of a generally poor nature compared to current practices. The concrete is potentially honeycombed (where the cement sand mixture is not continuous or uniform in the concrete). Also the concrete core walls are known to contain cold joints, where new concrete is poured in lifts onto previous concrete pours that have already set.
- Original fill materials were end dumped and hand placed, with no mechanical compaction. Where rock fill was used, some natural compaction will occur during placement. However, finer materials known to be present, such as sand, gravel, cinders and ash, will not self-compact and are likely to be relatively loose.
- End dumping of material tends to segregate course material from fine, rather than retaining an even mix. Therefore, there are likely lenses of coarse and fine material in the original dam fills.
- When constructed, both dams had wood stave low level outlets penetrating the fills and concrete cores, over time the wood stave conduits would rot, raising the possibility of seepage failure.

Both dams have been somewhat modified since their original construction. In 1980, the City undertook significant work on both dams to address the low level outlets. The valve chamber and low level outlets on the Lower Dam were filled with concrete, eliminating this source of concern on the Lower Dam. However, the low level outlet was not successfully located on the outside face of the Middle Dam, though its location was found on the inside face (below water level). The lower shell on the Middle Dam was substantially replaced with compacted fill composed of pit run sand and gravel, replacing the questionable original fill material, except on the left abutment (looking downstream) where a portion of the original fill remained.



### 2.1 SEISMIC ASSESSMENT

As noted above, EBA completed an extensive seismic investigation and assessment of both the Middle and Lower Dams in April 2010. Notably, the applicable design seismic event for these two structures is the 3000-year event, unless they are modified significantly, in which case, the more recent and stringent 5000-year event standard would apply. Normally, the structures would be expected to withstand whichever design seismic event is applicable.

EBA identified and assessed several failure scenarios related to seismic events. These included post seismic event piping related failures of the dam shells, leading to catastrophic failure of the concrete cores, catastrophic failure of the concrete cores during the seismic event, etc.

The EBA report highlighted one seismic failure mode in particular, that being the toppling of the upper portion of the inner concrete core of the Middle Dam. As a consequence, the Middle Dam is overtopped and the compacted sand and gravel outer shell is rapidly eroded. The full volume of the Middle Dam reservoir is then released over a short period of time. There is a reasonable likelihood that the Lower Dam would then fail due to overtopping by the Middle Dam flood wave, releasing the total volume of the Lower Dam reservoir.

This seismic failure mode has particular importance, as EBA was clear that an event considerably smaller than the design event could lead to a failure in this manner. Given the uncertainties regarding the stability of the dam fill and concrete core wall of the Middle Dam in the face of a seismic event, EBA could not conclusively define the seismic event that would initiate this failure mode. EBA estimated that an event as small as a 100-year return event (probability of 40% in a 50-year period) ranging to a 475-year event (probability of 15% in a 50-year period) could be sufficient to initiate this failure mode. The possibility of a dam failure associated with these relatively small and frequently occurring seismic events is not a certainty, but represents a significant cause for concern.

## 2.2 FLOOD EVENT RISKS

Both dams have limited spillway capacity, as assessed and reported by Water Management Consultants (WMC) in their "Middle and Lower Chase River Dams Spillway Hydrology Study" of April 2002. During floods exceeding the respective spillway capacities, overtopping of the dam crest and flow down the outer surface of the dams would occur. Overtopping would lead to erosion of the dam faces, which if sustained, would lead to loss of support to their internal concrete cores and their likely collapse.

The Lower Dam spillway capacities are considerably less than the estimated design flows for the probable maximum flood (PMF), and the 1000-year and 100-year return period floods. However, the Middle Dam spillway capacity is sufficient for the estimated 100-year return period flood event, but insufficient for the 1000-year return period and PMF flood events. The peak flows from the flood events, in comparison to spillway capacities are summarized in Table 2-1.

Dam	Spillway Capacity	Peak Flow Rates m3/s				
	m <sup>-</sup> /s	100-year Event	1000-year Event	PMF		
Middle	62	45	71	198		
Lower	25	48	75	205		

Table 2-1Comparison of Flood Events with Spillway Capacity

PMF flow rates are directly from WMC's report. 100-year and 1000-year return period flows are pro-rated from WMC's PMF estimates.

Previously, Associated Engineering completed a review of spillway capacity and flood scenarios for the Upper Chase River Dam ("Upper Chase Dam Flood Safety" technical memo, March 2011), which is located upstream of the two subject dams of this investigation. The earlier assessment estimated that a catastrophic overtopping failure of the Upper Chase River Dam during a flood event was unlikely due to the very wide crest width of that structure and small volume of impounded water. Consequently, the flood hydrographs for the Middle and Lower Chase River dams are considered to be unaffected by the presence of the Upper Chase River Dam. That technical memo is provided in Appendix C for reference.



3

# **Potential Failure Modes**

# 3.1 FLOOD EVENTS

WMC's investigation indicated that the spillways of both the Middle and Lower Dams have inadequate capacity for either the PMF or 1000-year return period event, with overtopping of the dam crests likely. In addition, the Lower Dam spillway has inadequate capacity for the 100-year event. However, the Middle Dam spillway does have adequate capacity for the 100-year return period flood event. Therefore, the following flood induced failures are possible:

- .1 100-year Return Period Flood Event Overtopping failure of the Lower Dam only.
- .2 1000-year Return Period Flood Event Overtopping failure of both the Middle and Lower Dams in a cascade sequence, where the Middle Dam fails first leading to the immediate failure of the Lower Dam. The Middle Dam breach and Lower Dam breach hydrograph peaks are assumed to coincide.
- .3 PMF Event Overtopping failure of both dams in a cascade sequence, where the Middle Dam fails first, leading to the immediate failure of the Lower Dam. The Middle Dam breach and Lower Dam reach hydrograph peaks are assumed to coincide.

There is uncertainty regarding the timing of the dam breaches with respect to the peak of the flood hydrographs. The dams could fail earlier than the flood peak or later if it takes more time for the downstream slopes to erode. It would require detailed knowledge of the dam structures and intensive geotechnical analysis to undertake an assessment of erosion rates to identify when each structure would lose stability and fail, which is beyond the scope of this study. Therefore, we decided to use a conservative approach and assume that the dam breaches and flood hydrographs would coincide in such a way as to produce the maximum peak flows. We note that the total flood volumes would not change regardless of the timing of the hydrographs.

Whether the two dams would fail in sequence is also not certain, as it is possible that the Lower Dam could fail first as spilling will begin over the Lower Dam's crest before it will at the Middle Dam, due to the greater deficiency in spillway capacity at the Lower Dam. However, the Lower Dam retains at least some of its mixed rock fill in the outer shell, which may turn out to be more resistant to erosion than the Middle Dam's modified outer shell. Nevertheless, it is clear that if the Middle Dam fails first, it will almost certainly result in a cascade failure of the Lower Dam.

In addition, for the 1000-year flood event, a failure of the Middle Dam is not certain, given the small difference between spillway capacity and the peak flow rate of the 1000-year event. Even so, given that the outer shell of the Middle Dam is potentially easily eroded, there is still a reasonably possibility that the Middle Dam could fail. If the Middle Dam were not to fail, but the Lower Dam did, the result would be nearly identical to the 100-year flood event scenario.



### 3.2 IDF HYDROGRAPHS

The PMF hydrograph developed by WMC was used as the basis for the inflow design flood (IDF) hydrographs used to estimate the ultimate dam breach hydrographs. As WMC only estimated the PMF hydrograph, we pro-rated the 24-hour PMF hydrograph to match the estimated peak flow rates of the IDFs for the 100-year and 1000-year return period flood events.

WMC's PMF hydrograph starts at time 0 with an initial flow of 0 m<sup>3</sup>/s. Given the likelihood of wet antecedent conditions leading up to a major rainfall driven flood event, we deemed it prudent to add a base flow condition to the various IDF hydrographs. For the Middle Dam, we applied a base flow of 5 m<sup>3</sup>/s to the prorated IDF hydrographs. An additional 2 m<sup>3</sup>/s of base flow was added to the IDFs at the Lower Dam to reflect the additional contributing catchment area between the Middle and Lower Dams.

Both dams are currently classified as "Very High Consequence" structures under current provincial dam regulations. As such, by regulation, the appropriate IDF would be 2/3rds of the difference between the 1000-year flood event and the PMF. Since it is at the very least possible, and more likely probable, that both dams would fail during any flood event significantly exceeding a 100-year return period, the released reservoir volumes play a greater role in determining downstream flooding than does the particular IDF. Therefore, to simplify this assessment, we assessed the 100-year, 1000-year and probable maximum inflow flood events, rather than an interpolated event between the 1000-year return period and PMF.

The basic IDFs generated by this approach represents the raw input hydrographs to the dams prior to any reservoir routing (attenuation) affects and without the addition of the dam breach component. Determination of the outflow hydrographs requires a reservoir routing model analysis, and the inclusion of the dam breach conditions as appropriate for the three flood-induced failure scenarios indicated above, discussed in the following section.

## 3.3 RESERVOIR ROUTING AND DAM BREACH ASSESSMENT

Prior to undertaking the flood inundation modelling discussed in Section 4, we needed to develop the hydrographs representing the various flood events after they have passed through the two reservoirs and are discharged through the dam spillways, and over the dam crests in the case of overtopping. Dam breaches are included in the modelling by allowing each dam to "open" to release the impounded volume.

In order to develop the final flood hydrographs, we developed a hydraulic model representing both dams, their respective reservoirs, spillways and weir flow over the dam crests when overtopping occurs. USEPA's SWMM5 software was used for routing and dam breach hydrograph generation.

The following major features were incorporated in the model.

#### Spillways

WMC provided rating curves for each of the dam spillways. These are repeated below and were incorporated into the routing model.

Reservoir Elevation (m)	Head (m)	Discharge (m³/s)
86.2	0.0	0
86.4	0.2	2
86.8	0.6	10
87.5	1.3	30
88	1.8	50
88.3	2.1	62

Table 3-1Middle Dam Spillway Rating Curve

Table 3-2Lower Dam Spillway Rating Curve

Reservoir Elevation (m)	Head (m)	Discharge (m <sup>3</sup> /s)
71.6	0	0
71.9	0.3	2
72.2	0.6	6
72.4	0.8	10
72.6	1.0	15
72.8	1.2	20
73.0	1.4	25
73.2	1.6	30
73.4	1.8	35

Note that overtopping of the Lower Dam crest only occurs when the water surface elevation of the reservoir exceeds 73.4 m, at a discharge of  $35 \text{ m}^3$ /s. However, WMC's report indicates that the spillway itself will be overtopped downstream of the dam crest when the discharge exceeds approximately  $25 \text{ m}^3$ /s, with the spilling flow impinging on the dam's outer shell. This leads to the nominal capacity limit for the Lower Dam spillway of  $25 \text{ m}^3$ /s.



## Crest (Overtopping) Discharges

Survey drawings were reviewed for both dams to estimate the crest length, width and elevation in order to include overtopping weir flow in the routing model. Each dam had the appropriate dimensions included in the routing model, with the crest modelled using the model software's standard broad crested weir equations.

### Reservoir Stage Storage Relationships

A critical element in the routing assessment, is the ability of the two reservoirs to store and attenuate some of the incoming flood hydrographs, potentially reducing the peak flows that need to be conveyed by the two spillways or discharged over the dam crest. When dam breaks are not considered, the flood hydrographs arriving at the floodplain can be flattened by reservoir routing, reducing the peak flows (but not the total volumes) in the floodplain.

The City of Nanaimo has undertaken both topographic and bathymetric surveys of the Middle and Lower Dam reservoirs. We used this data to develop stage (elevation)-area relationships that are used in the model to estimate storage volumes at any given reservoir elevation. From our data analysis, we determined that the storage volume of the Middle Dam, at the elevation of the spillway crest, is approximately 110,000 m<sup>3</sup>, in comparison with a previously stated volume of 92,600 m<sup>3</sup>. Similarly, we estimated the Lower Dam reservoir contains a storage volume of approximately 112,000 m<sup>3</sup>, at the elevation of the spillway crest, compared to a previously stated value of 173,000 m<sup>3</sup>. However, stored volumes increase as the depth in the reservoir increases in response to inflows (such as the IDF), as governed by the interaction of each reservoir's stage-storage relationship and the corresponding spillway rating curve.

The stored volume in each reservoir will be a major determining factor in the degree of flooding that occurs in the event of a dam breach.

#### Dam Breach Simulation

As discussed in Section 2, both dams have a unique mix of concrete cores with outer shells of fill material. To put this in perspective, earth fill dams generally fail by relatively gradual erosion and collapse of the earth embankment, opening and releasing the stored reservoir volume over an extended period of time (possibly hours). Concrete arch dams generally fail in a quick and catastrophic manner, such that the dam breach opens fully in a short time, with the stored volume released rapidly.

The assumed overtopping failure mode during a large flood event assumes that the overtopping flow erodes the downstream face of each dam. At some point, the respective concrete cores, which likely lack steel reinforcing and therefore have little resistance to bending moments, will fail catastrophically. So these dams are assumed to fail in a fracture critical mode after erosion has undermined support for the concrete cores rather than in a more gradual manner as with a true earth fill structure.

Developing a precise failure mode model for each dam would be a challenging assignment, requiring a detailed knowledge of the material properties of each dam. This would include erosion susceptibility of the outer shells and the bending moment resistance of the concrete cores, including spatial variations. Given the uncertainties and variables with each of these structures, this approach does not appear feasible. Instead, a common approach is to assume that the breach opens over a period of time without specific simulation of the mechanisms.

Accordingly, in our routing and dam breach model assessment, we modelled each dam as a large gate, which will open over a period of time according to the control instructions that we give the model. Though conservative, we assumed that the Middle Dam would begin to breach at the peak in the IDF; similarly, the Lower Dam breaches when the combined peak of the IDF and the dam breach reach it. Each "gate" was given the dimensions of the downstream side of the dam in the ravine, i.e. the span, depth and ravine side slopes on the downstream face. Notably, the Lower Dam has a much larger potential "gate" than does the Middle Dam, since it has a greater span and depth. While there is no mechanism to suggest that the Lower Dam would necessarily breach over its entire span, we believe this is a reasonable approach since a breach of the Lower Dam would be magnified by the impact of the flood wave from the Middle Dam, as assumed in the 1000-year and PMF flood events. This approach is also conservative for the purposes of considering impacts on the downstream floodplain.

We assumed that the dam breaches open over a period of minutes. For both dams we assumed the breach opens over a time span of 3 minutes, 20 seconds upon the arrival of the peak of the respective flood waves. Table 3-3 indicates the peak volume reached in each reservoir immediately prior to the simulated dam breach, and the elapsed time for each reservoir to drain to 10% of its normal stored volume.

	100-yea	r Event	1000-year Event		nt PMF	
Dam	Peak Volume (m³)	Time to Drain H:MM	Peak Volume (m³)	Time to Drain H:MM	Peak Volume (m³)	Time to Drain H:MM
Middle	147,000	No Breach	159,000	1:03	205,000	6:00
Lower	165,000	0:09	204,000	0:09	238,000	0:14
Peak Flow	537	m³/s	1140	m <sup>3</sup> /s	1168 n	า <sup>3</sup> /ร

## Table 3-3 Dam Failure Details

Due to the larger potential opening in the Lower Dam, it actually loses its impounded volume before the Middle Dam has completely drawn down. The peak stored volumes are greater than the nominal storage volumes of each reservoir due to surcharging produced by the flood events.



Using our reservoir routing model, we briefly assessed the effect of varying the time for each dam breach to fully open on the flood volumes entering the floodplain below the dams. While peak flow rates diminished by approximately 40% when the duration of breach opening was doubled, within ten minutes of the onset of breaching of the Lower Dam, the extended opening case lagged the rapid opening case by only 1.3% of total flood volumes. At longer elapsed times the two hydrographs converge. Accordingly, we anticipate that the extent of flooding in the Chase River floodplain will be largely dominated by the total flood volume released, rather than the duration over which the dam breaches propagate, unless the collapse of each dam was drawn out over a significantly greater period of time. Given the identified concerns and uncertainties with each dam's structure, as discussed in Section 2, a longer time frame for dam collapse does not appear to be justifiable.

The final IDF hydrographs for each of the three events, at each of the Middle and Lower Dams are plotted on Figures 3-1 and 3-2, respectively. The IDF hydrographs for the Lower Dam include whatever attenuation affects are provided by the Middle Dam reservoir, prior to breaching of the Middle Dam. Note that these IDF hydrographs are not a component of the seismic scenarios discussed below; only the respective standalone dam breach hydrograph for the Middle and Lower Dams are relevant for a seismic event.

The resulting flood hydrographs for input to the flood routing model (Mike Flood) for the 100-year, 1000-year and PMF events, including dam breach contributions, are presented on Figure 3-3.

## 3.4 SEISMIC EVENT

Based on EBA's 2010 Seismic Assessment, we consider the most relevant seismic (or "sunny day") scenario to be an earthquake of sufficient magnitude to cause the Middle Dam concrete central core to break and topple upstream into the reservoir. The Middle Dam then rapidly breaches due to the resulting loss of crest height and overtopping flow.

Under this scenario, the timing of the breaches is not a source of uncertainty as the breach of the Middle Dam leads directly to the overtopping and breach of the Lower Dam.

A key difference between the seismic scenarios and any of the flood scenarios, is that there is no flood induced surcharging of the reservoirs above their normal water levels. For the purposes of this evaluation we assume that the Middle and Lower Dams are impounding 110,000 m<sup>3</sup>, and 112,000 m<sup>3</sup> respectively. Using our routing model, we again generated the dam breaches in sequence, but without the IDF inputs. Each dam was assumed to breach in approximately 3 minutes 20 seconds, with the Lower Dam breaching as the Middle Dam hydrograph peaks.

The modelled seismic event generates a peak flow of just under 1000 m<sup>3</sup>/s.

The estimated flood hydrograph for the seismic scenario is presented as Figure 3-4.

4

# **Floodplain Model Development and Setup**

The development of flood inundation mapping for the Chase River floodplain requires the support of extensive data sources that cover all relevant aspects of the modelling and damage assessments. Accordingly, the City of Nanaimo provided the following data in support of this project:

- Contour data for the City of Nanaimo (no bathymetric data was obtained for the ocean area downstream of the Chase River).
- Orthophotos.
- Bridge and culvert measurements.
- Legal lot outlines.
- Building outlines.
- Assessed property value and property improvement data.
- Road centerlines and road names.
- Full-Feature LiDAR elevation data (i.e. including treetops and building roofs).

### 4.1 SOFTWARE SELECTION

The selection of appropriate software for the analysis of potential dam breach scenarios requires careful consideration. A variety of hydraulic conditions will exist while the flood wave travels throughout the inundation area. These conditions include sub-critical and super-critical flows.

Another important consideration relates to the hydraulic analysis of the Chase River channel, as well as the adjacent floodplain. Immediately below the Lower Dam, the Chase River channel has significant conveyance capacity. However, this conveyance capacity rapidly diminishes upstream of Howard Avenue.

Hydraulic analysis of the Chase River channel, while flows are below the top of bank, can be completed by using a 1-dimensional river modelling program such as HEC-RAS or Mike-11. 1-dimensional river modelling programs calculate water levels and flow rates based on input cross-section data. The direction of flow for these models follows the river alignment, which is perpendicular to the input cross-sections. These models are unable to account for lateral outflows and alternate flow paths above the top of the cross-sections; this inability results in the label '1-dimensional model'.

Once the river channel capacity is significantly exceeded, as would be the case during a dam breach condition, then the conveyance and attenuation of the flood wave becomes an important condition. Non-uniform floodplain areas, such as the Chase River floodplain, downstream of the Lower Dam cannot be completed with confidence using a 1-dimensional modelling program. As a result, a 2-dimensional analysis of the floodplain area is required. 2-dimensional models do not use cross-section data and instead rely on a rectangular grid or, occasionally, a triangular irregular network, where each grid cell or triangular cell has been assigned a specific elevation. Models utilizing a triangular cell system can also account for the slope and aspect (slope direction) of the cells, which can result in computational efficiencies over a rectangular grid network. As a result of the cellular elevation system that describes the topography of the study area,



the model can account for flow in any horizontal direction. This ability earns this type of model the label '2-dimensional'. Notably, these models assume a uniform vertically averaged velocity.

Computational effort within the floodplain using a 2-dimensional solution is significantly affected by the resolution of the grid. Reducing the grid size results in increased computational effort and longer model run times for a fixed duration simulation. For example, a grid resolution of 1 m by 1 m would take approximately 4 times longer to run than a 2 m by 2 m grid. Generally, floodplain areas can be accurately modelled using a grid size that is significantly larger than what would be necessary to model a river channel such as the Chase River. As a result, this project would benefit significantly, in terms of computational effort, from the use of a 1-dimensional river model and a 2-dimensional floodplain model where flow can be exchanged dynamically throughout the simulation.

The Danish Hydraulic Institute (DHI) produces a wide variety of hydraulic modelling programs including 1-dimensional, 2-dimensional and 3-dimensional platforms. Many of their programs are able to dynamically exchange flows within a single simulation. In addition, the software programs are recognized internationally by engineers and scientists, and have been in use for several decades.

As a result, we selected Mike-Flood, produced by DHI, which allows for the dynamic coupling (flow exchange) of the following programs:

- Mike-Urban (a 1-dimensional pipe and minor watercourse program).
- Mike-11 (a 1-dimensional major watercourse and river modelling program).
- Mike-21 (a 2-dimensional program capable of modelling flows in floodplains, as well as lakes and oceans).

Note that Mike-Urban was not used in the coupled model for this project.

## 4.2 CHASE RIVER CHANNEL

The main channel of the Chase River was modelled using Mike-11. Representative cross-sections for the Chase River were created from City-supplied contour data. The City-supplied bare-earth contour data was compared to unprocessed (i.e. trees and buildings were included in the surface) LiDAR data. Areas within the LiDAR data, without tree coverage, were compared to the City contour data and the vertical accuracy was generally within 10 cm or better. The contour data within the tree covered area of the river channel, at least from a visual perspective, appears to present a reasonable representation of the river channel.

The dam breach scenario hydrographs contain peak flows that are orders of magnitude higher than the 100-year design flow rate for the Chase River. As a result, we concluded that potential vertical and horizontal errors in the aerially derived contour data for the Chase River would not materially affect the hydraulic results.

Bridge and culvert openings were field measured by City of Nanaimo staff. The bridges and culverts were included in the Mike-11 model. In addition, weirs were added to the Mike-11 model to allow for overtopping of the bridges during high-flow portions of the simulations.

The Mike-11 model extents and cross-section locations are shown in Figure 4-1.

## 4.3 CHASE RIVER FLOODPLAIN

The City-supplied contour data was converted to a 10 m by 10 m rectangular grid using GIS software. This raster grid was then imported into Mike-21, the 2-dimensional modelling program, where the boundary conditions for the grid were developed.

Closed boundaries were used along all sides of the imported grid with the exception of the downstream ocean area that receives the discharge from the Chase River. This portion of the boundary was left as an open boundary in order to allow flows to leave the model area.

The boundary condition water surface elevation along the eastern ocean boundary was set at 2.02 m GSC, which corresponds to the higher high-tide condition at the Nanaimo tide gauge. We note that the ocean area of the Mike-21 model does not include actual bathymetric data. Both models have the same extents.

# 4.4 COUPLED MODEL

The Mike-11 and Mike-21 models are coupled using Mike-Flood in order to allow for the dynamic exchange for flow between the two models. The coupling of the models is achieved by identifying grid cells within the Mike-21 model which correspond to sections of the Mike-11 model.



# **Failure Analysis**

The results presented in this section are based on the dam breach hydrographs discussed in Section 3. Table 5-1 shows the assumed capacity of the bridges and culverts for each scenario.

We note that the concrete arch culvert beneath the E&N railway embankment, located near 9th Street and Applewood Drive, presents a major impediment to the downstream movement of the flood wave. The railway elevation within the Chase River ravine is approximately 29 m GSC and the invert of the culvert beneath the embankment is estimated at 13 m GSC. These scenarios generate peak water levels at the railway culvert, assuming no loss of conveyance as a result of debris accumulation, of 27 m (Scenario 2) and 29.8 m (Scenario 4). Notably, Scenario 4 results in about 45 minutes of sustained flow over the railway track with a maximum depth of 0.8 m.

Although we have no information with respect the construction of the railway embankment, we believe that it will fail, either by seepage or by overtopping, during some of the events. Given the uncertainty of the failure mode and timing of the railway embankment we have chosen to model some of the events without the railway embankment or the culvert. Inclusion of the railway culvert is noted in Table 5-1 and discussed in the respective events where relevant.

Scenario Name	Hydrologic Event	Bridges and Culverts with Full Capacity	Bridges and Culverts Modelled as 1 m diameter Culverts	Railway Embankment Failure
SC1	1000-year flood with no breach	✓		
SC2	1000-year flood with lower and Middle Dam breach		~	
SC3	100% PMF with no dam breach	✓ (except Hwy. 1 culvert)	✓ (Hwy. 1 culvert only)	<b>~</b>
SC4	100% PMF with lower and Middle Dam breach		✓	<b>*</b>
SC5	100-year with Lower Dam Break		~	
SC6	Earthquake		✓	

# Table 5-1 Flood Event Details



### 5.1 FLOODING

#### Scenario 1 – 1000-year flood (no breach)

This scenario assumes that the 1000-year flood event occurs without the failure of the Lower or the Middle Dams and is a reference event to allow an estimate of incremental damages due to dam failure. The peak of the flood hydrograph passes over the dam crests and continues downstream along the Chase River channel.

Significant debris generation is not expected as part of this scenario. As a result, we believe that loss of conveyance capacity at the bridges and culverts will not occur. The model in this scenario assumes full conveyance capacity for the bridges and culverts. The estimated maximum flood depth and the full extent of flooding are presented in Figures 5-1A and 5-1B.

Note that the figures do not feature an overlapping area between them as the ravine contains the flows for all scenarios. As a result, the figures show areas where flooding occurs beyond the banks of the Chase River. This applies to all figures in this report.

Overbank flooding occurs in a number of locations and impacts several properties.

#### Scenario 2 – 1000-year flood (middle and lower dam breach)

This scenario assumes that the 1000-year flood event occurs with the failure of the Lower and the Middle Dams. The peak of the flood hydrograph passes over the dam crests and erodes the dams until the concrete dam cores fail in sequence. The flood flows, as well as the volume of water impounded by the dams, flows downstream along the Chase River channel.

Failure of the Middle and Lower Dams is expected to generate a large amount of debris. This debris is expected to significantly affect the conveyance capacity of the bridges and culverts. The bridges at Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue are assumed to have a reduced capacity similar to a 1 m diameter culvert in order to simulate the effects of accumulated sediment, gravel, trees and other floatable debris. The estimated maximum flood depth and the full extent of flooding are presented in Figures 5-2A and 5-2B.

This scenario results in a significant increase in flood depths and extents, particularly upstream of Park Avenue. Many properties and streets are affected by overbank flooding.

#### Scenario 3 – PMF (no dam breach)

This scenario assumes that the PMF safely passes over the Lower and Middle Dams without resulting in a breach. This is a reference scenario to determine the incremental effects of a dam failure. The flood hydrograph then flows downstream along the Chase River channel.

Significant debris generation is not expected as part of this scenario. As a result, we believe that loss of conveyance capacity at the bridges and culverts will not occur. The model in this scenario assumes full conveyance capacity for the bridges.

During this scenario the railway embankment near 9<sup>th</sup> Street and Applewood Drive experiences an upstream surcharge depth of over 14 m. The duration of surcharge and the surcharge depth are likely to result in the failure of the earth-fill railway embankment. However, the failure mode is uncertain and a geotechnical investigation is necessary to establish the likely failure mechanism.

This scenario has been modelled without the railway embankment or the culvert. As a result, the flood wave moves downstream without attenuation. It is possible that a catastrophic failure mode of the railway embankment may result in increased downstream flood depths when compared the results of this scenario.

The culvert beneath Highway 1 is likely to experience significant blockage as a result of the expected railway embankment failure. As a result, the Highway 1 culvert has been modelled as a 1 m diameter culvert in order to simulate a debris blockage.

The estimated maximum flood depth and the full extent of flooding are presented in Figures 5-3A and 5-3B.

This scenario results in significant flooding within the floodplain. Numerous properties and streets are affected.

### Scenario 4 – PMF (lower and middle dam breach)

This scenario assumes that the PMF results in the failure of the Lower and Middle Dam as a result of overtopping. The flood flows, as well as the volume of water impounded by the dams, flows downstream along the Chase River channel.

Failure of the Middle and Lower Dams is expected to generate a large amount of debris. This debris is expected to significantly affect the conveyance capacity of the bridges and culverts. The bridges at Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue are assumed to have a reduced capacity similar to a 1 m diameter culvert in order to simulate the effects of accumulated sediment, gravel, trees and other floatable debris.

During this scenario, the railway embankment near Ninth Street and Applewood Drive is overtopped for nearly 1 hour. Maximum water depths at the upstream end of the railway embankment culvert are 16.8 m. The duration of overtopping combined with the duration of surcharge and the surcharge depth are likely to result in the failure of the earth-fill railway embankment. However, the failure mode is uncertain and a geotechnical investigation is necessary to establish the likely failure mechanism.

This scenario has been modelled without the railway embankment or the culvert. As a result, the flood wave moves downstream without attenuation. We note that a failure of the railway embankment as a result of overtopping would dramatically increase the extent of downstream flooding.

The estimated maximum flood depth and the full extent of flooding are presented in Figures 5-4A and 5-4B.



This scenario results in significant flooding within the floodplain. Numerous properties and streets are affected. In comparison to the other scenarios evaluated as part of this study, this scenario represents the most significant flooding.

### Scenario 5 – 100-year Flood (lower dam breach)

This scenario assumes that the 100-year flood only results in the failure of the Lower Dam. The 100-year flood flows, as well as the volume of water impounded by the Lower Dam, flows downstream along the Chase River alignment.

Failure of the Lower Dam is expected to generate a large amount of debris. This debris is expected to significantly affect the conveyance capacity of the bridges and culverts. The bridges at Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue are assumed to have a reduced capacity similar to a 1 m diameter culvert in order to simulate the effects of accumulated sediment, gravel, trees and other floatable debris.

The water levels upstream of the railway embankment reach an elevation of 21.6 m, which represents a depth of 8.6 m. We do not anticipate a failure of the railway embankment as part of this scenario. The estimated maximum flood depth and the full extent of flooding are presented in Figures 5-5A and 5-5B.

#### Scenario 6 – Earthquake (lower and middle dam breach)

This scenario assumes that the Lower and Middle Dams fail as a result of a seismic event. The water impounded by both the Lower and Middle Dams flows downstream along the Chase River channel as a result of the seismically induced failure.

Failure of the Lower and Middle Dams is expected to generate a large amount of debris. This debris is expected to significantly affect the conveyance capacity of the bridges and culverts. The bridges at Howard Avenue, Bruce Avenue, Seventh Street and Park Avenue are assumed to have a reduced capacity similar to a 1 m diameter culvert in order to simulate the effects of accumulated sediment, gravel, trees and other floatable debris.

The water levels upstream of the railway embankment reach an elevation of 21.6 m, which represents a depth of 8.6 m. We do not anticipate a failure of the railway embankment as part of this scenario. The estimated maximum flood depth and the full extent of flooding are presented in **Figure 5-6A** and **5-6B**. This scenario results in extensive flooding upstream of Park Avenue.

#### 5.2 PROPERTY DAMAGE ESTIMATION

#### **Building Values**

We obtained property tax assessment information for properties within the area of interest in a spreadsheet format. This was imported to our GIS database and assigned to individual properties by street address which provides a spatial distribution of property values throughout the area of interest.

The stated value of improvements reflects the condition, or age, of the property. For properties in moderate to poor condition, or those properties that are older, the value of the improvements does not reflect the replacement value.

We set a minimum building improvement value of \$100,000 and adjusted the value of those structures with improvements below this amount. A more detailed assessment of replacement cost would require information on the number of floors, the presence of a basement and total floor area per building.

The damage to contents requires an estimate of their replacement cost. Contents would include appliances, home electronics, furniture, clothing, vehicles and other typical items. As this information is not available, we used 50% of the improvement value to a maximum of \$100,000 for single family residential buildings. This results in a range of the value of contents between \$50,000 and \$100,000.

The above approach provides a relative ranking of the various flood scenarios on the basis of damage. However, the estimated damage may be less than what may actually occur.

### Depth-Damage Curves

US Agencies have spent considerable efforts developing methods and gathering damage-related data to support the estimation of damage resulting from flooding. The 'Multi-hazard Loss Estimating Methodology', published by the Federal Emergency Management Agency (FEMA), provides information and methods for estimating losses in the event of a flood.

The technical manual, titled 'HAZUS-MH MR4', provides guidance on applying the loss estimation software. The software is an add-in for ESRI GIS software and utilizes databases provided by the US Government in relation to various types of infrastructure, including buildings. This database does not include data for Canadian regions; the property assessment data, as well as estimated contents, will be used in lieu of this information.

The technical manual discusses the methods and the damage curves that are used to identify the damage percentage of a building resulting from depth of water as well as a depth-velocity relationship. The depth-damage curve provides an estimate of the percent loss based on water depth. The depth-velocity-damage curve provides an estimate of the buildings ability to withstand the momentum of the flowing water and is used to identify structures which may collapse. A collapse would result in 100% damage. In the event of a building collapse, the building damage would be set to 100% which may represent a higher loss value than the depth-damage relationship.

We used the following three damage relationships which are taken from the 'HAZUS-MH MR4' technical manual in order to estimate the loss percentage. The figure numbers for the three damage relationships correspond to the 'HAZUS-MH MR4' report.





Figure 5.3 FIA-Based Structure Depth-Damage Curve 2 or More Stories, Basement-Modified



Figure 5.5 FIA-based Residential Contents Damage Curves









In summary, the process for estimating the property damage is as follows:

- .1 Adjust assessed building improvement value to a minimum of \$100,000, where necessary.
- .2 Estimate the value of contents at 50% of the building value, but limit maximum contents value to \$100,000.
- .3 Identify the maximum depth and maximum velocity for each building based on the model output.
- .4 Estimate the percent damage based on maximum water depth using depth-damage curves.
- .5 Check for building collapse using velocity and depth information. If necessary, increase building damage to 100%.
- .6 Multiply the building value and the contents value by the damage percentage. Sum total losses by property.

Table 5-2 outlines the number of damaged structures as well as the estimated value of damages.

Scenario	Description	Damaged Structures	Building Damage	Contents Damage	Total Damage	Incremental Damage
1	1000-year flood with no breach	28	\$2,800,000	\$1,400,000	\$4,200,000	\$4,200,000
2	1000-year flood with lower and Middle Dam breach	234	\$25,700,000	\$14,900,000	\$40,600,000	\$36,600,000
3	100% PMF with no dam breach	110	\$8,900,000	\$4,900,000	\$13,800,000	\$13,800,000
4	100% PMF with lower and Middle Dam breach	341	\$32,000,000	\$14,900,000	\$46,900,000	\$33,100,000
5	1000-year with Lower Dam Break	206	\$25,700,000	\$12,200,000	\$37,900,000	\$37,900,000
6	Earthquake	221	\$26,100,000	\$12,400,000	\$38,500,000	\$38,500,000

Table 5-2Property Damage by Scenario

Note: Incremental Damage is the additional damage related to a dam breach coinciding with a flood event, in comparison to a flood event occurring without a dam breach.

Figures 5-7 to 5-12 show the estimated damage to buildings within the floodplain area for each scenario.

We note that the above damage estimates do not include costs related to the following:



- Damage to assets other than privately owned housing:
  - City-owned utilities
  - Third party utilities (i.e., electrical, gas, telecommunication)
  - Bridges
  - Culverts
  - Roads
  - E&N Railway
    - Other non-building assets
- Litigation
- Loss of life
- Economic or consequential losses
- Environmental damage:
  - Damage to Chase River habitat and fish stocks
  - Movement of material to the ocean
  - Cost of clean-up:
    - Trees
    - Floatable debris within the floodplain mobilized by the flood wave
    - Sediment from the dams or scoured areas
    - Contamination
    - Hauling material to a suitable landfill
- Damage to sites having a social or historical significance

# 5.3 FLOOD WARNING

The duration from the onset of a dam breach to flood inundation plays a role in the perceived risk to the public. Without adequate warning, the notification and evacuation of the affected population becomes difficult.

Figures 5-13 to 5-18 show the duration from the onset of the dam failure, or the time of peak flow for those scenarios where no dam failure occurs, to initial flooding. The time intervals used in each of the figures varies based on the rate of flood inundation.

# Scenario 1 – 1000-year flood (no breach)

This scenario does not result in significant flooding and generally does not result in short-notice flooding. However, a number of buildings are flooded along Chase River and most of the flooding seems to occur prior to the peak flow, which suggests a gradual rate of flooding.

# Scenario 2 – 1000-year flood (middle and lower dam breach)

This scenario includes the failure of the Middle and Lower Dam. The majority of the flooding occurs within 20 minutes of the dam failure. Notification and evacuation of residents, as well as the public would be particularly difficult if not initiated prior to the observed onset of the dam failure.
Evacuation and notification could be completed based on observed water levels within the reservoirs, observed inflow rates to each of the reservoirs, and observed rainfall rates within the watershed. Such a surveillance program would require significant amounts of watershed modelling in order to develop a reliable flood-forecasting tool. Data collection efforts would need to be implemented on a permanent basis. However, such flood-forecasting tools are imperfect and may result in notification and evacuation of the public when it is not required.

The presence of a large educational facility presents a risk to students during a daytime flood event combined with a dam breach. Evacuation of students on short notice would be difficult. Young students may find it particularly difficult to evacuate the area or to find a safe area within the flood zone.

A nighttime flood event, without notification or evacuation, would take many people occupying basements or first-floor residences by surprise. Under these circumstances, residents would need to evacuate themselves prior to the arrival of emergency responders, which may prove difficult for many under such conditions.

#### Scenario 3 – PMF (no dam breach)

This scenario results in significant flooding. However, it would likely occur gradually throughout the course of a few hours. The gradual onset of flooding at numerous locations along the Chase River would presumably trigger existing emergency plans and provide time to notify and evacuate most residents.

#### Scenario 4 – PMF (lower and middle dam breach)

Like scenario 3, a significant amount of flooding occurs prior to the breach of the dams. The gradual onset of flooding may trigger existing emergency plans and provide time to notify and evacuate some residents. However, the unanticipated and sudden failure of the Middle and Lower Dams would catch many people, including emergency responders, by surprise.

#### Scenario 5 – 100-year Flood (lower dam breach)

This scenario results in some flooding prior to the failure of the Lower Dam. In this instance the Lower Dam takes longer to fail after the onset of overtopping. However, the majority of the flooding happens very quickly after the failure occurs.

The flooding leading up to the failure may result in some localized evacuations. However, the unanticipated and sudden failure of the Lower Dam would catch many people, including emergency responders, by surprise.

#### Scenario 6 – Earthquake (lower and middle dam breach)

This scenario differs significantly from the other scenarios, as no notice can be provided prior to the seismic event, and is expected to result in the sudden failure of the Lower and Middle Dams. In this scenario, the large majority of the flooding occurs within 15 minutes of the seismic event.



The rapid occurrence of dam breaches and the onset of flooding after a damaging seismic event would create considerable difficulty for many people. Self-evacuation from damaged structures without personal injury would be particularly difficult.

Factors which exacerbate the dangers under this scenario include:

- The triggering seismic event does not have to be as catastrophic as a major seismic event and could be as small as a 100-year return period event. Therefore the population at risk may not realize that a critical situation has developed.
- A nighttime event could result in a sluggish response.
- Immediate emergency response efforts could be divided between several developing situations.
- Residents may be reluctant to leave if rescue or assistance of family or neighbors is required.

6

# Summary

The presence of the Lower and Middle Dams along the Chase River present a potential risk to downstream areas as a result of the impounded volume of water upstream of the dams. This report has considered several different events that are expected to result in the failure of the dams. Each of these events has a probability of occurrence that can be used to estimate the risk of the event occurring during a 50-year period.

Figure 6-1 shows the probability of occurrence during a 50-year period for each of the simulated events as well as the estimated damages. This plot clearly shows the significant variation in probability for the PMF event (0.5% chance during a 50-year period) and the seismic event (39.5% chance during a 50-year period). However, the most informative part of this figure is the clear demonstration that the consequence of failure from a PMF related dam failure or a seismically induced dam failure are very similar despite the significant difference in probability of occurrence. In summary, this figure demonstrates that there is essentially a 40% chance during a 50-year period that the dams will fail as a result of a seismic event and that damages would be in the range of \$40 million (2012 dollars).

The buildings within the floodplain are generally single family residential homes; however, the area does contain a large school, a daycare, apartment buildings, and some commercial buildings.

The flood zone also contains major transportation corridors including Highway 1 and the E&N Railway.

The flood zone will also contain general (at large) members of the public, as well as local residents. We have estimated the population within the flood zone based on the 2011 census data for the area by using the average population per housing unit and the number of dwelling units. The spatial distribution of dwelling units was provided by the City. Our estimate is outlined in Table 6-1.

Area	Daytime Population Estimate	Nighttime Population Estimate
Single Family Homes	1.2 per house	2.2 per house on average
John Barsby High School	650	5
Apartment Buildings	1.2 per apartment unit	2.2 per apartment unit
Daycare	23	0
Travelling Public	100	10
Total Estimated Population	1810	1883

#### Table 6-1 Estimate of Affected Population



Many variables affect the population at risk including time of day and time of year (season). With respect to estimating the potential number of causalities the single most important factor is the amount of warning provided to the public prior to flooding, as well as the time required to evacuate the area. Providing sufficient warning in order to allow the population at risk to evacuate the area will dramatically reduce the number of potential casualities.

#### Seismic Event Induced Dam Failure

A failure of the Lower Dam during a 100-year flood or the failure of the Middle and Lower Dams during a seismic event, presents the highest level of risk. Notably, these events result in damages that are similar to extreme events such as the 1000-year flood or the PMF.

In our opinion, the event of greatest concern is the seismic failure of the dams as there would be virtually no warning or time for self-evacuation of the public within the affected inundation areas. This would likely result in many casualties.

Our assessment of potential causalities for the seismically induced dam failure assumes the following:

- .1 The public is generally unaware that a significant earthquake would result in an immediate dam failure.
- .2 The public is generally unaware that a seismically induced dam failure will result in damaging flows immediately downstream with virtually no warning.
- .3 Residents will not receive notice of the dam failure or subsequent flooding, either by emergency personnel or nearby residents, until at least 30 minutes after the earthquake.
- .4 Ten percent of single family homes have an occupant who normally sleeps in the basement or ground floor.

We note that the number of fatalities is difficult to estimate and there are many factors that will influence the fatality rate.

Approximately 130 buildings will be flooded to a significant depth within 30 minutes.

At the upstream end of the floodplain, 13 buildings are expected to be destroyed or completely inundated by the flood wave within 5-10 minutes of the dam failure. Notably, the dam failure may occur during or immediately after an earthquake. Maximum water levels in this area range from 1.2 m to 11 m, while velocities range from 1 m/s to 5 m/s. Effective self-evacuation of persons in the area from damaged buildings, as well as the floodplain in general, with no prior warning and immediately after a seismic event is unlikely. Therefore we expect a number of causalities to occur in this area. We note that the rapid onset of flooding combined with the depths and velocities would result in individuals being swept away by the flood wave. In this area, we estimate that the daytime number of casualties as being on the order of 20 people, while the number of nighttime casualties may be higher, possibly in the range of 30 to 40 people.

Further downstream, near Park Avenue, a number of larger buildings experience a flooding depth in the range of 2 m to 6 m with less than 30 minutes of warning. Anyone within in the building would not be able



Figure 6-1

to escape after the onset of flooding, as the maximum flood depth in many cases exceeds the height of the building. Also, anyone outside of the buildings would likely be swept away by the flood wave due to the depth and velocity of the flows. In this area, we estimate that the daytime number of casualties may be in the range of 35 to 40, while the number of nighttime casualties may be in the range of 45 to 60.

Elsewhere within the floodplain we estimate that another 40 to 50 fatalities may occur as a result of occupied buildings located near the river, individuals trapped within basement areas which would flood rapidly even if water depths in the floodplain are low, traveling public and individuals being swept away while attempting to self-evacuate across the floodplain.

Accordingly, we estimate that the seismically induced dam failure could have total fatalities in the range of 80 (daytime) to 150 (nighttime) people. This number will be influenced by the severity of the earthquake, the time of day, degree of structural damage resulting from the earthquake, and the corresponding number of people trapped by structural damage. We note that the accurate estimation of casualties is particularly difficult as a result of the many factors that influence the outcome.

An educational program for the affected public with respect to the likelihood of a dam failure following a seismic event combined with designated evacuation areas for each building may significantly reduce the number of casualties. However, it is very unlikely that casualties could be eliminated.

#### Flood Induced Dam Failure

We understand that the City of Nanaimo has an emergency management plan related to the Lower and Middle Dams. An emergency plan would include a strategy for monitoring the dams and notifying the public with respect to evacuation. A key element of such a plan relates to the amount of time required to notify potentially impacted residents as well as the more general public and the amount of time required for evacuation of the endangered area. Based on our analysis, very little time is available to notify and evacuate the public once overtopping of one or both of the dams begins. For many areas, significant flooding or increase in flood depth occurs within 15 to 30 minutes after the start of dam failure. We note that the dams may be overtopping for some time prior to failure. However, we believe that it may be difficult to effectively notify all potentially affected residents and allow sufficient time for evacuation.

The proximity of downstream residential buildings, as well as the educational and childcare facilities, to the Lower and Middle Dams would likely require a flood-forecasting tool in order to provide sufficient time to evacuate the large number of students prior to a daytime dam failure.

Such a tool would be necessary in order to properly evaluate the potential for overtopping of the dams based on observed inflows to the dams, water levels within the reservoirs, observed rainfall rates within the watershed, and weather forecasts. This type of tool would likely provide emergency personnel with sufficient time to notify and evacuate the public from the inundation area in the event of an anticipated flood related dam failure. Notably, such a forecasting tool would provide no benefit in increasing the warning time with respect to a seismically induced dam failure.



Without a flood-forecasting tool, we estimate that the number of fatalities may be in the range of 30 to 60. This number will be influenced by the timing of the dam failure after the onset of dam overtopping, the amount of warning provided to residents prior to flooding and the amount of time available for evacuation.

# Conclusion

The Chase River Dams present a significant risk to downstream areas in the event of a dam failure. The most significant risk results from a seismically induced dam failure. We note that a 100-year return period seismic event is estimated to result in a significantly higher number of causalities than the PMF which has an approximately probability of a 10,000-year return period event. However, economic damages for the PMF event are about 25% higher than the seismic event.

In addition to the high degree of risk to the downstream areas as a result of a dam failure, the probability of an extreme failure is very high. Figure 6-1 shows the probability of occurrence during a 50-year period for each of the simulated events as well as the estimated damages. This figure clearly demonstrates that during the next 50 years there is a 40% chance that the dams will fail as a result of a seismic event and that damages would be in the range of \$40 million (2012 dollars). This degree of risk provides a compelling argument for action.

Various approaches are available for reducing the consequences of failure resulting from a dam breach. These include flood-forecasting tools, educational programs for residents within the floodplain area, evacuation plans, and evacuation notification systems. These approaches would be expected to reduce the potential casualties for the events considered. However, these approaches would not reduce economic losses or significantly reduce the estimated casualties resulting from a high-probability seismically induced dam failure. As a result, we conclude that other means of risk mitigation must be utilized.

The probability of failure combined with the consequences of failure present a compelling case for modifying the dams. The possible modifications, assuming that the 'do-nothing' option is unacceptable, are as follows:

- The upgrade or replacement of the existing dams.
- The removal of the existing dams and their respective impoundments.

We note that this report does not account for the significance or value of existing environmental, social and cultural assets within the floodplain area.

The economic losses identified in this report are based on the values of assessed structures within the floodplain area as provided by the City of Nanaimo. An allowance was made for building contents which was based on a percentage of the structure value. The contents allowance includes one vehicle per dwelling.

The economic losses exclude damage to public or privately owned utilities, consequential losses, and clean-up of the floodplain area after a dam failure.

Based on the foregoing, we conclude that:

- The estimated number of casualties resulting from the seismic event requires the consequence classification of the Lower and Middle Dams along the Chase River to be increased from 'Very High' to 'Extreme' based on the British Columbia Dam Safety Regulation. The 'Extreme' classification is the highest consequence classification under the British Columbia Dam Safety Regulation.
- As a consequence of the uprating of the dams from 'Very High' consequence structures to "Extreme" consequence structures, under the Provincial Dam Safety regulations the appropriate inflow design flood (IDF) for these dams is now the probable maximum flood (PMF). As discussed in Section 2, neither dam has adequate spillway capacity for the PMF. In addition, the difficulties in providing sufficient spillway capacity for the PMF were identified in WMC's report of April 2002.
- A flood related dam failure, as a result of overtopping, would require a flood-forecasting tool and/or a surveillance system in order to provide adequate time for notification and evacuation of residents. Evacuation alerts and orders would have to be issued prior to the onset of overtopping in order to provide adequate time for evacuation. Depending upon the time of day, this could be problematic. Also, a seismically driven dam failure, which we believe is the greatest risk, would not be covered by such a system.
- A seismic event leading to the failure of the dams could have as high as a 40% probability of occurrence during a 50-year time period.
- A seismic event leading to the failure of the dams is a major concern as notice cannot be provided to the public and inadequate time would be available for self-initiated or directed evacuation of most residents. The estimated number of fatalities is in the range of 80 (daytime) to 150 (nighttime) people.
- Based on current development and estimated population distribution, the estimated direct economic damages are \$38 million for a seismically induced dam failure.
- Incremental direct economic losses to structures and contents resulting from a flood driven failure of both dams, based on assessment building values, are estimated in the \$33 million to \$36 million range depending on the scenario. Direct losses range from \$40 million to \$46 million. We note that replacement costs may be higher.
- Based on current development and estimated population distribution, the estimated number of casualties as a result of a flood related dam failure is in the range of 30 to 60 people.

Based on the above conclusions, we recommend that the City of Nanaimo select one of the following two options:

- ,1 Rehabilitate or replace the existing Lower and Middle Dams to meet the current Dam Safety requirements.
- .2 Remove the existing Lower and Middle Dams.

# 8

# **Glossary of Terms**

- DHI– Danish Hydraulic Institute. A software and consulting company.
- EBA EBA Consulting Engineers, A Tetra-Tech Company. A consulting company.
- E&N Esquimalt and Nanaimo Railway.
- FEMA Federal Emergency Management Agency (US Government Agency).
- GIS Geographic Information System. A spatial database software program.
- GSC Geodetic Survey of Canada. A horizontal and vertical survey control system.
- HEC-RAS Hydrologic Engineering Centers River Analysis System. A free-ware river modelling software program.
- IDF Inflow Design Flood.
- LiDAR Light Detection and Ranging. An optical remote sensing technology for aerial survey.
- PMF Probable Maximum Flood.
- USEPA United States Environmental Protection Agency (US Government Agency).
- SWMM5 Storm Water Management Model (version 5). A hydrology model with a onedimensional hydraulic routing model for pipes and open channels.
- WMC Water Management Consultants. A consulting company.

# 9

# Certification

This report presents our findings from the Chase River Dam Breach Flood Inundation Study. Please contact either of the undersigned regarding any questions.

Prepared by:



Andrew Wiens, P.Eng. Water Resources Engineer

AW/MM/lp

Michael MacLatchy, Ph.D., P.Eng.<sup>7</sup> Senior Water Resources Engineer

Reviewed by:

A Appendix A - Figures







### IDF Hydrographs for Middle Chase River Dam



Flood Inundation Study

#### IDF Hydrographs for Lower Chase River Dam



## Flood Event Hydrographs at Floodplain Entrance

City of Nanaimo Chase River Dam Breach Flood Inundation Study



## Seismic Event Hydrograph at Floodplain Entrance





Dam Locations Chase River Mike-11 Cross-Section River Bank

# Figure 4-1

DATE: June 22, 2012



Property Damage







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# Figure 5-12B

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Building Footprint **Time to Inundation** Flooding at start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 40 min after Dam Breach

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## Figure 5-13B

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Building Footprint **Time to Inundation** Flooding at start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach

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## Figure 5-14B

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Building Footprint **Time to Inundation** Flooding at Start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach C c<sup>l</sup>col z D



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Building Footprint **Time to Inundation** Flooding at Start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach

## Figure 5-15B

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Building Footprint **Time to Inundation** Flooding at Start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach 10002






Building Footprint **Time to Inundation** Flooding at Start of Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach

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### Figure 5-16B

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### Figure 5-17B

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Building Footprint **Time to Inundation** 5 min after Dam Breach 10 min after Dam Breach 20 min after Dam Breach 30 min after Dam Breach 60 min after Dam Breach

### Figure 5-18B

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Figure 5-1A DATE: June 22, 2012



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#### Figure 5-1B

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Scenario 2 - 1000-Year Flood with Lower and Middle Dam Breach Maximum Flooding Depth and Extents

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## Figure 5-2B

DATE: June 22, 2012



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Maximum Flooding Depth and Extents



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Scenario 3 - 100% PMF with No Dam Breach Maximum Flooding Depth and Extents

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Maximum Flooding Depth and Extents







Scenario 4 - 100% PMF with Lower and Middle Dam Breach Maximum Flooding Depth and Extents

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# Figure 5-4B

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### Figure 5-5B

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Scenario 6 - Earthquake Maximum Flooding Depth and Extents

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### Figure 5-6B

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Property Damage





### Figure 5-7B

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Property Damage

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#### REPORT

**R** Appendix B - Upper Chase River Dam



Associated	GLOBAL PERSPECTIVE.	Date:	March 8, 2011	File:	20102733.00.E.03.00
Engineering	LOCAL FOCUS.	То:	Scott Pamminger		
		From:	Michael MacLatchy		
		Project:	Chase River Dams	9	
MEMO		Subject:	Upper Chase Dam Flo	ood Safe	ty

# This memo outlines our ongoing assessment of future options for the Upper Chase River Dam in the City of Nanaimo and is intended to support discussion of the range of options available for the Upper Chase River Dam.

#### 1 PREVIOUS DISCUSSION

Previously, our team identified that decommissioning the dam would result in destruction of habitat that was created in the bypass channel, which has functionally become the main channel of the Chase River. We also noted that decommissioning the dam could provide an opportunity to establish additional habitat that could partially or fully compensate for this loss.

Our assessment indicates that should the Upper Chase River Dam remain, flow would have to be maintained in the existing spillway and bypass channel due to the identified habitat value. Also, the spillway capacity of the No. 1. Reservoir Dam is limited. Therefore, it is not advisable to route all of the Chase River flows through this facility, which was designed to be offline from the River.

This suggests that unless both facilities are to be decommissioned, the current flow routing of the Upper Chase River Dam should be maintained and water levels are not likely to be significantly reduced.

Previous studies by Water Management Consultants indicated the hydraulic capacity of the existing spillway system is limited by the existing culverts to a flow rate of approximately 18 m<sup>3</sup>/s. If the hydraulic constriction represented by the culverts were removed then the capacity of the spillway itself would increase to approximately 32 m<sup>3</sup>/s, which is consistent with the estimated 100-year return period flood event. The spillway system is incapable of safely routing either the Q1000 flow rate (60 m<sup>3</sup>/s) or the PMF flow rate (168 m<sup>3</sup>/s) without overtopping the dam structure at one or more locations. This raises the possibility that the structure could be breached by erosion due to overtopping.

#### 2 APPLICABILITY OF BC DAM SAFETY REGULATION

The possibility to declassify the Upper Chase River Dam as a dam for the purposes of BC's Dam Safety Regulation (the Regulation) has been raised. The goal of a possible declassification is to reduce the surveillance and reporting requirements that the City must comply with for this structure. The Regulation states:

#### 2 Application

(1) This regulation applies to all of the following:

(a) a dam 1 metre or more in height that is capable of impounding a volume of water greater than  $1,000,000 \text{ m}^3$ ;





Memo To: Scott Pamminger March 08, 2011 - 2 -

(b) a dam 2.5 metres or more in height that is capable of impounding a volume of water greater than  $30,000 \text{ m}^3$ ;

(c) a dam 7.5 metres or more in height;

(d) a dam that does not meet the criteria under paragraph (a), (b) or (c) but has a downstream consequence classification under Schedule 1 of low, high or very high.

Accordingly, we calculated the elevation-volume relationship for the No. 2 Reservoir to assess the applicability of the Regulation and whether changes to the Upper Chase Dam would allow for declassification.

The City provided an historic drawing that indicates the topography of the No. 2 Reservoir in the period of the 1930s (Plan of No. 1 and 2 Service Reservoirs, City of Nanaimo, November 1933). Using a PDF copy of this drawing, we were able to re-trace the contours into Civil 3D at a known scale and use them to estimate the volume of the reservoir. We note that the drawing indicates the spillway sill has an elevation of 375.54 feet or 114.46 m. Other documentation, including the Water Management Consultants reports, provide a sill elevation of 111.6 m, indicating that a different datum was used for the 1933 drawing or a systematic error is present. The discrepancy appears to be 2.86 m.

The resulting storage curve (using corrected elevations) is attached as Figure 1. Please note that the approximate depth of the reservoir, to the spillway crest is 3 m or greater (the 1933 drawing does not indicate the deepest point immediately in front of the concrete wall), and the estimated storage volume is approximately 30,000 m<sup>3</sup>. The 1933 drawing places the Upper Chase River Dam as being on the threshold of applicability under criteria (b) of the Regulation. A more accurate estimate, using better bathymetry data, could confirm the Upper Chase River Dam's applicability under criteria (b) one way or the other.

Further, aerial photography and site observations indicate that a significant portion of the reservoir volume has been displaced by the accumulation of sediment from Chase River inputs. Using a rough estimate of the area of reservoir lost by superimposing the digitized contours onto aerial photography, we achieved an order-of-magnitude estimate that the reservoir volume has decreased by approximately 10,000 m<sup>3</sup>, to 20,000 m<sup>3</sup> at the spillway sill elevation of 111.6 m. This is a rough estimate; confirmation with up to date bathymetry is again required.

However, the issue of reservoir volume does not solely decide the applicability of the regulation, as criteria (d) would cause the dam to remain classified unless its consequence rating was Very Low. Golder, in their Dam Safety Review of April 2004 recommended the dam's consequence rating be reduced to Low from its rating of High, which has been done by the Dam Safety Branch. Even under this less severe rating of Low, the Upper





Memo To: Scott Pamminger March 08, 2011 - 3 -

Chase River Dam would still remain classified as a dam under the Regulation regardless of the decreased storage volume, but would not have to meet as stringent seismic and flood safety requirements. Only a rating of Very Low would allow complete declassification of the Upper Chase River Dam. Achieving that rating is uncertain at this time.

The volume analysis does confirm that this structure impounds a relatively small volume of water. Golder's 2004 assessment indicates that a catastrophic failure of the structure is not a credible event under seismic conditions. If erosion due to overtopping does cause a release of the impounded volume, it would occur over a period of time as the dam fails under flood conditions. This being the case the stored volume would add a minor increment to an already occurring flood event. This reasoning supports the classification of Low.

In comparison to the consequences of the Middle (92,600 m<sup>3</sup> of storage) and Lower (173,000 m<sup>3</sup> of storage) Chase River Dams failing in sequence under a significant flood event (or due to seismic forces), it is a reasonable argument that the Upper Chase River Dam is of low consequence in the context of an extreme event involving the lower two dams.

#### 3 FLOOD ROUTING

Does this imply that no action is necessary due to the Upper Chase River Dam's limited storage capacity? That depends on the potential impacts and vulnerability of the No. 1 Reservoir Dam (considered a High Consequence structure), immediately downhill from the Upper Chase River Dam.

Currently, the Upper Chase River Dam ensures the diversion of river flows around the No. 1 Reservoir site under frequently occurring flow conditions, including moderate flood events. As previously identified by others, and discussed above, the existing spillway has the capacity to safely discharge approximately 18 m<sup>3</sup>/s to the Chase River (i.e. the bypass channel). This would be increased to 32 m<sup>3</sup>/s if the hydraulic constriction imposed by the culverts under Nanaimo Lakes Road were removed. At best this would allow safe passage of a 100-year return period event.

Assuming that the spillway is not further upgraded, we investigated the potential flow paths of the remaining portion of the flood hydrograph by examining the topography in the immediate vicinity of the Upper Chase Dam, and the elevations of key features such as the concrete dam wall and Nanaimo Lakes Road.

Other than the spillway itself, the next spill point for the Upper Chase Dam is the earth fill dam section to either side of the spillway, with elevations on the order of 114 m. Nanaimo Lakes Road has an average elevation in this area of 113.5 m, so it would not provide a significant impediment to spilling flows. Under a 1000-year return period flood event (60 m<sup>3</sup>/s), with the spillway operating at its capacity of 32 m<sup>3</sup>/s (i.e., assuming no culvert restrictions), the weir flow over the earth dam crest would be approximately 0.60 m deep. No flow would be occurring over the concrete wall section during the 1000-year return period flood event.





Memo To: Scott Pamminger March 08, 2011 - 4 -

During the PMF (168 m<sup>3</sup>/s) the water level in the lake will have risen to approximately 115.65 m, with spilling occurring over both the earth fill and concrete wall sections of the dam.

To illustrate the potential impacts to the No. 1 Reservoir (capacity 60,000 m<sup>3</sup>), we developed Figure 2, a conceptual sketch showing flow routing during these two extreme events. Please note that this sketch does not represent the results of modelling or detailed investigation of overland flow routing, but a qualitative interpretation given the approximate water levels that would occur at the Upper Chase River Dam under extreme flood events. These flow paths also assume that the Upper Chase River Dam remains intact without degradation of the crest elevation. During the 1000-year return period event, at least some (indeterminate) portion of the flood hydrograph will pass down to the No. 1 Dam. A greater flow will pass to the No. 1 Dam during a PMF event.

If the Upper Chase River Dam spillway capacity is increased to the  $Q_{1000}$ , then spilling is confined to the earth dam section for events greater than the 1000-year return period up to the PMF. No spilling of excess flood flows over the concrete wall section will occur. This will likely greatly reduce the flood flows entering the No. 1 Reservoir during extreme flood events, those greater than the Upper Chase River Dam spillway capacity.

At this time, the consequences of excess flood flows entering the No. 1 Reservoir are not known. It is possible that the No.1. Dam could withstand the increased hydrostatic forces as a result of receiving flood flows from the upper Chase River, particularly given the structural reinforcing and anchoring undertaken in 1996. A structural investigation of this dam to determine its capacity to withstand high flood flows would be required. Increasing the potential concern with this structure is that it could fail in a more catastrophic manner versus a gradual process, as with an earthfill structure, and could aggravate the downstream impacts of an extreme flood event. As it is yet to be determined whether this dam and reservoir will remain after the completion of the new water treatment plant and new No. 1 Reservoir, the issue could well be moot.

A significant concern is the erosive power of the flood flows passing over Nanaimo Lakes Road as they spill from the Upper Chase River Dam. Existing and future water mains on this alignment are critical infrastructure to the City of Nanaimo and damage as a result of extreme events should be avoided or reduced as much as practical.

#### 4 MITIGATIVE ALTERNATIVES

Based on the above discussion - what measures can be undertaken to partially or fully address these concerns?

Only two options completely nullify all of the factors discussed above:

1. Completely remove both the Upper Chase Dam and the No. 1 Reservoir Dam. Provide habitat compensation for the abandoned bypass channel in the restored original channel reaches.





Memo To: Scott Pamminger March 08, 2011 - 5 -

2. Provide spillway capacity for the PMF at the Upper Chase River Dam. Channel capacity improvements may also be required.

However, the likely consequences of not being able to pass a PMF relate to the downstream impact of the potential failure of the No. 1 Reservoir, and loss of the major supply water mains in Nanaimo Lakes Road if the Upper Chase River Dam overtops. Catastrophic failure of the Upper Chase River Dam does not appear to be a likely outcome during a PMF.

Incremental options that could reduce, but not eliminate, the likelihood of serious consequences:

- 3. Increase the capacity of the Upper Chase River Dam, consistent with the 1000-year return period flood event, and verify the consequences to the No. 1 Reservoir Dam of the excess spilling resulting from larger events.
- 4. Provide armouring or other protection of Nanaimo Lakes Road and the dam embankment in the vicinity of the earth fill dam to protect water infrastructure for events up to and including the 1000-year return period event. Implicitly, this infrastructure could be lost or damaged during events exceeding the 1000-year return period flood.
- 5. Remove the No. 1 Reservoir Dam and infrastructure (only necessary if the structure will fail with large overtopping flows).

As a minimal measure, and in order to safely pass the 100-year return period event, the culverts under Nanaimo Lakes Road would need to be removed/upgraded. Of course, the do-nothing option remains.

Our assessment is that Options 2 and 3 appear preferable:

Option 2 completely nullifies all PMF related risk factors associated with these two structures. The capital cost of expanding the spillway capacity at Upper Chase River Dam to convey the PMF will be high.

Option 3 assumes that the Upper Chase River Dam will be retained. The spillway would be upgraded to provide capacity for the 1000-year return period event. This will ensure the safety of the Nanaimo Lakes Road infrastructure and the No. 1 Reservoir Dam to a reasonable level of service. This is contingent on the No.1 Reservoir Dam having the ability to pass significant overtopping flows since it is a High consequence structure, or its decommissioning.

However, Option 1 becomes more feasible if it is decided that the No. 1 Reservoir Dam will not be retained for other purposes once its drinking water role ceases, and consequently will be removed. Then decommissioning of the Upper Chase River Dam could follow.





Memo To: Scott Pamminger March 08, 2011 - 6 -

Our recommendations are:

- Verify the ability of the No.1 Dam to resist overtopping flows. Mitigation strategies may be required if this structure cannot resist overtopping during extreme events exceeding the Q<sub>1000</sub>.
- Design and Construct the Upper Chase River Dam spillway for a Q<sub>1000</sub> capacity (60 m<sup>3</sup>/s). This is primarily intended to protect critical water mains in Nanaimo Lakes Road from erosion and severing.

In our judgement, a 1000-year level of service is appropriate for the Upper Chase River Dam spillway.

We trust this memo proves useful in advancing this project, and look forward to discussions with the City of Nanaimo.

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FIGURE 2