Estimating Dam Break Downstream Inundation

Dam Safety Program

Ministry of Forests, Lands and Natural Resource Operations

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Contents

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Introduction

In order to determine the failure consequence classification for dams in BC, a number of things must be considered including: the population at risk, the estimated loss of life, the cultural and environmental consequences, the economic losses and the impact on infrastructure. The first steps are to determine the area that will be inundated, the depth and velocity of the flood waters and the length of time that the area will be inundated. This document outlines a simplified method for assessing the impact of the downstream inundation. This guideline is intended to be used for small dams (under 15 meters in height) but the principles apply to all dams. Please also refer to the Canadian Dam Association (CDA) Technical Bulletin, “Inundation, Consequences and Classification for Dam Safety”, 2007.

If this simplified method provides a clearly defined failure consequence classification then a consequence classification can be assigned. If the results are uncertain, the highest possible consequence classification can be accepted or a more detailed consequence assessment can be conducted. For larger structures or structures with complicated downstream channel conditions more detailed inundation studies are probably required.

Consequence Classification

Schedule 1 of the Dam Safety Regulation outlines a dam failure consequence classification guide for dams in British Columbia. The failure consequence classification (extreme, very high, high, significant or low) identifies the potential for damage and loss in the event of a dam failure. The consequence classification is not a reflection on how safe the dam is or what the probability of failure of the dam is; thus age and condition of the dam are not reflected in the failure consequence classification. A **“Downstream Consequence of Failure Classification Interpretation Guideline for Dam Safety Officers”** has been prepared as a companion document to this guideline.

The prime purpose of the failure consequence classification is to determine the design requirements for a particular dam; dams with higher failure consequence are required to be designed to higher standards. Suggested design requirements for dams falling under the various consequence classifications are identified in the Canadian Dam Association’s “Dam Safety Guidelines”. Regulatory requirements such as maintenance, operations and surveillance are also based on the failure consequence classification.

This document concentrates on the first step in the process, estimating the extent of the downstream inundation. This document also offers a set of hydrograph attenuation curves that were derived for central BC conditions and a graph showing the time between breach and inundation (e.g., the warning time).

The State of Washington Dam Safety Office also has a guidance document, "Dam Safety Guidelines, Technical Note 1: Dam Breach Inundation Analysis and Downstream Hazard Classification" which addresses simplified methods of estimating the inundation and also classifying state dams under their hazard classification system. They also provide associated Microsoft Excel spreadsheets, including calculations for both the dam breach hydrograph and downstream routing of the dam breach flood, in their "Dam Safety Technical Notes" which can be found at: http://www.ecy.wa.gov/programs/wr/dams/GuidanceDocs_ne.html (See Dam Safety Technical Notes: Self-extracting file that contains Technical Note 1 Spreadsheets and Documents). Template spreadsheets and worked examples are provided, in both metric and U.S. customary units, along with a separate set of instructions and tips (Instruct.doc) for using the spreadsheets.

Estimating Inundation Caused by a Dam Breach

Estimating the inundation begins with an estimate of the flood hydrograph; how much water pours through the breach and how fast it pours through. The flood hydrograph is a plot of discharge versus time. The hydrograph resulting from a dam breach is dependent on many factors; primarily the physical characteristics of the dam, the volume of the reservoir, and the mode of failure. Characteristics such as dam geometry, construction materials and mode of failure determine the dimensions and timing of the breach formation. Breach formation (dimensions and timing), volume of reservoir storage, and reservoir inflow at the time of failure determine the peak discharge and the shape and duration of the flood hydrograph. All potential dam failure modes should be considered in the analysis.

The following sections provide a method for estimating breach formation parameters and peak flow discharges for earthfill dams. The focus is on earthfill dams because the majority of small dams in BC are earthfill.

Selection of Reservoir Conditions for Breach Analysis

The selected reservoir storage is an important consideration in dam breach analysis. Normally at least two reservoir conditions are considered, normal pool (normal water level) and maximum water elevation during the design flood.

For smaller unattended structures usually only the case of dam failure during overtopping needs to be considered. Overtopping could result from a debris blockage, or a beaver dam being constructed in the overflow spillway channel. In evaluating the overtopping dam breach it needs to be remembered that the reservoir storage and head on the dam are greater than for normal pool levels. For a sunny day breach scenario, that would result from perhaps an earthquake induced failure or an internal erosion induced failure, it might be more appropriate to use the normal water level. Generally both situations should be reviewed; as although there is less water in the reservoir in the sunny day breach scenario there is potentially significantly less warning of a breach, and therefore potentially a larger loss of life.

Estimation of Dam Breach Parameters for Earthfill Dams

Work by MacDonald and Landridge-Monopolis (MacDonald, 1984) were successful in relating breaching characteristics of earthfill dams to measurable characteristics of the dam and reservoir. Specifically, a relationship exists between the volume of material eroded in the breach and the Breach Formation Factor (BFF):

$$BFF = V_w (H)$$

where:

V_w = Volume of water stored in the reservoir (acre-ft) at the water surface elevation under consideration

H = Height of water (feet) over the base elevation of the breach

Interpretation of data (MacDonald, 1984) suggests that the estimates of material eroded from earthfill dams may be taken to be:

$V_m = 3.75\ (BFF)^{0.77}$ for Cohesionless Embankment Materials; and

$V_m = 2.50\ (BFF)^{0.77}$ for Erosion Resistant Embankment Materials

where:

V_m = Volume of material in breach (yds^3) which is eroded

Using the geometry of the dam and assuming a trapezoidal breach with sideslopes of (Z_b:1) the base width of the breach can be computed (MacDonald, 1984) as a function of the eroded volume of material as:

$$W_b = [27V_m - H^2 (CZ_b + HZ_bZ_3/3)] / [H (C + HZ_3/2)]$$

where:

W_b = Width of breach (feet) at base elevation of breach

C = Crest Width of dam (feet)

$Z_3 = Z_1 + Z_2$

Z_1 = Slope (Z_1:1) of upstream face of dam

Z_2 = Slope (Z_2:1) of downstream face of dam

If the calculated breach width is negative then the reservoir volume is not large enough to fully breach the dam and a partial breach will result. In this case the head of water (H) needs to be adjusted to estimate the breach depth and peak discharge. Mean breach widths for earthfill dams are about 4 times dam height (MGS Engineering, 2007). However, site geometry can limit breach width.

The time of breach development (τ) in hours, has been related to the volume of eroded material (MacDonald, 1984). Interpretation of data suggests that the time for breach development can be estimated by:

$\tau = 0.028\ V_m^{0.36}$ for Cohesionless Embankment Materials; and

$\tau = 0.042\ V_m^{0.36}$ for Erosion Resistant Embankment Materials

There is a large uncertainty in the eyewitness accounts for many of these failures; thus these equations may tend to overestimate breach times. In addition, these equations appear to produce unrealistically short breach development times in the case of small dams. A lower limit for the breach development time of perhaps 10 minutes for dams constructed of cohesionless materials and 15 minutes for dams constructed of erosion resistant materials seems reasonable.

Due to the uncertainties in breach development parameters, a range of values should be used to assess the computed dam break flood peak discharges. There is a range of alternative procedures for estimating dam break parameters. An example is the computer program BREACH, developed by Fread (1987) which is used for larger complex dams.

Estimation of Dam Breach Parameters for Concrete Dams

When estimating concrete gravity dam breach parameters, a complete failure of a discrete number of monoliths is considered to be the most likely scenario, and this defines the range of breach widths. For concrete arch dams a complete dam failure is possible and should be considered. Breach times for concrete gravity dams generally fall between 0.1 and 0.5 hours and for concrete arch dams breach times estimates generally fall between instantaneous and 0.1 hours.

Estimation of Dam Breach Peak Discharge

A number of computer programs, such as DAMBRK (Fread, 1988), have been developed for estimating dam break peak discharge. DAMBRK, and other similar programs utilize unsteady flow conditions in combination with user selected breach parameters to compute the breach flood hydrograph.

Fread (1981) gives an alternative method suitable for many planning purposes. He developed an empirical equation based on numerous simulations with the DAMBRK model. Estimation of the peak discharge from a dam breach is computed as:

$$Q_p = 3.1\ W\ H^{1.5}\ [\ A\ /\ (A + \tau\ H^{0.5}]^3$$

where:

Q_p = Dam breach discharge (cfs)
W = Average breach width (feet) $W = W_b + Z_b H$
H = Initial height of water (feet) over the base elevation of the breach
τ = Elapsed time for breach development (hours)
A = 23.4 S_a / W
S_a = Surface area of reservoir (acres) at level corresponding to depth H

Tables 1 & 2 contain estimates of dam breach peak flows for overtopping induced failures of earthfill dams based on Fread's equation. The values used in developing these estimates are presented after the tables. Values were not entered into the tables for cases in which the calculated breach did not develop to the full depth of the dam. In addition, values were not entered into the tables when breach widths were calculated to be greater than 5 times the dam height.

Tables 1 & 2 – Earthfill Dam Peak Discharge Estimates

(For Cohesionless and Erosion Resistant Materials)

Dam Breach Discharge Estimates
for Earthfill Dams Constructed of Cohesionless Materials

Dam Breach Peak Discharge (m3/s)

Dam Height (meters)	Reservoir Surface Area (hectares) → 1	2	3	4	5	10	15	20	40
1.2	6.9	12*							
2	14	23	31	39			Breach Width > 5xDam Height		
3	23	38	50	61	72				
4		52	69	84	98	160			
5		67	88	106	123	200	267	329	
6			106	128	149	239	318	391	
7				149	173	277	367	450	
8				170	196	313	414	507	832
9					219	347	459	561	918
11						411	542	661	1077
13		Partial Breach				467	616	752	1221
15							682	832	1351

* This discharge value results from a breach width of 5.2 times the dam height

Dam Breach Discharge Estimates
for Earthfill Dams Constructed of Erosion Resistant Materials

Dam Breach Peak Discharge (m3/s)

Dam Height (meters)	Reservoir Surface Area (hectares) → 1	2	3	4	5	10	15	20	40
1.2	4.5	7.8	11						
2	8.8	15	21	26	31		Breach Width > 5xDam Height		
3	15	25	34	41	49	81			
4	22	35	46	57	67	110	148	183	
5		45	59	72	84	138	185	228	
6		56	72	88	102	165	221	272	452
7			85	103	119	192	256	315	521
8			98	118	136	218	290	356	587
9				132	153	244	323	396	651
11					184	292	385	471	771
13		Partial Breach				336	443	541	881
15						376	496	605	983

The tables were computed based on:

Failure by overtopping thus H, S_a, and V_w are for reservoir at crest of dam (they are not values for normal reservoir level)
Storage volume was calculated as ($H\ S_a$ / 3)
Slope of the upstream face is assumed to be 3H: 1V
Slope of the downstream face is assumed to be 2H: 1V
Crest width $C = 2 + 2\ H^{0.5}$ (in feet)
Breach sideslopes (Z_b: 1) are 1.0 for cohesionless embankment material, and 0.5 for erosion resistant embankment material
Minimum breach development times of 10 minutes for cohesionless embankment material, and 15 minutes for erosion resistant embankment material was used.

It should be noted that actual peak discharges could vary greatly from the calculated peak discharges. Differences in site conditions, dam materials, and reservoir inflow could greatly influence the results. For example a dam increasing storage on an existing lake could result is greater peak breach flows due to a greater reservoir volume than modelled.

Downstream Routing of Dam Breach Flood

As the dam breach flood wave travels downstream there is usually a reduction in the peak flow. Occasionally this is not the case; please see the note at the end of this section.

This effect is governed by factors such as:

the channel bedslope,
the cross-sectional area and geometry of the channel and overbank areas,
the roughness of the main channel and overbank,
the existage of storage for floodwaters in off-channel areas, and
the shape of the flood hydrograph.

Small attenuation is associated with:

large reservoir volume,
small confining channel,
steep channel slopes, and
little frictional resistance in channel and overbank areas.

Large attenuation is associated with:

small reservoir volume,
broad floodplain and/or off-channel storage areas,
mild channel slopes, and
large frictional resistance in channel and overbank areas.

There are a number of methods for modelling the attenuation of peak flow as the breach flood wave travels downstream. The HEC-RAS program is one method of modelling the attenuation. The following sets of curves were generated by the Kamloops Dam Safety Office using HEC-RAS to show how the peak flood flow changes with distance downstream from the dam, see note in References. The four sets of curves are all based on a typical stream in central BC, but have assumed different average bed slopes. The curves were generated by assuming that the stream was a “U”-shaped stream with average side slopes of 2V:1H, with a Manning roughness of 0.08 for the channel and 0.10 for the overbanks. This is similar to a Type 2 channel as shown in Table 3. This roughness assumes that some of the flow is significantly affected by vegetation growing above the normal high-water level.

The curves in Figures 1 to 4 are arranged in terms of reservoir storage. They show flood attenuation in terms of peak dam breach discharge (Q_p) at the dam site and peak discharge (Q_x) at some distance downstream.

Flood routing should be continued to a point downstream where the dam break flood no longer poses a risk to life and there is limited potential for further property damage. Flood routing is often terminated when the dam break flood enters a large body of water that could accommodate the floodwaters without a significant increase in water level or when the flood has attenuated to a level that is within the 200-year floodplain for the receiving stream. In the latter case, flood plain mapping for designated flood plains in BC is available on the MFLNRO, Water Management web site at:

http://www.env.gov.bc.ca/wsd/public_safety/flood/fhm-2012/landuse_floodplain_maps.html

Note: Under some geological conditions, a sudden dam breach flood can trigger a debris flow in the creek bed below the dam. Under these conditions, the inundation flow may be much larger than the flood released by the dam breach. An example of this rare occurrence is the Testalinden Dam failure near the town of Oliver in 2010. Contact the Dam Safety Officer if there is a question about this issue.

Figure 1: Hydrograph Attenuation Curves – 5% Average Channel Slopes

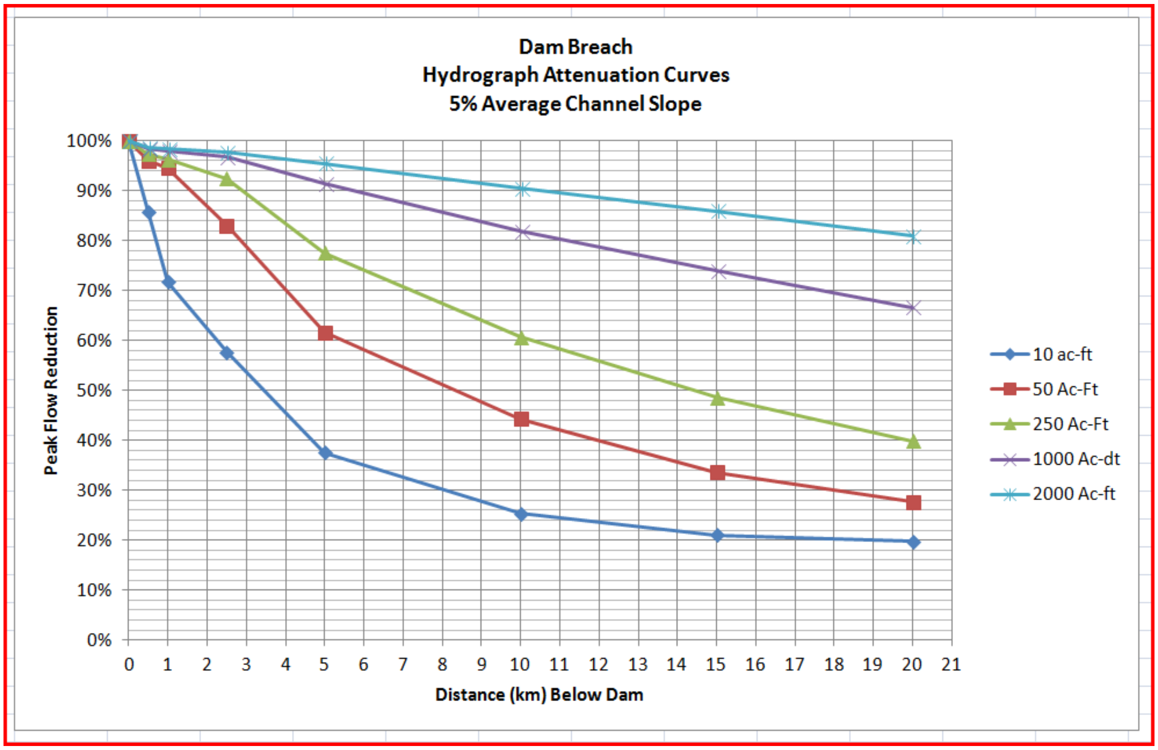


Figure 2: Hydrograph Attenuation Curves – 1% Average Channel Slopes

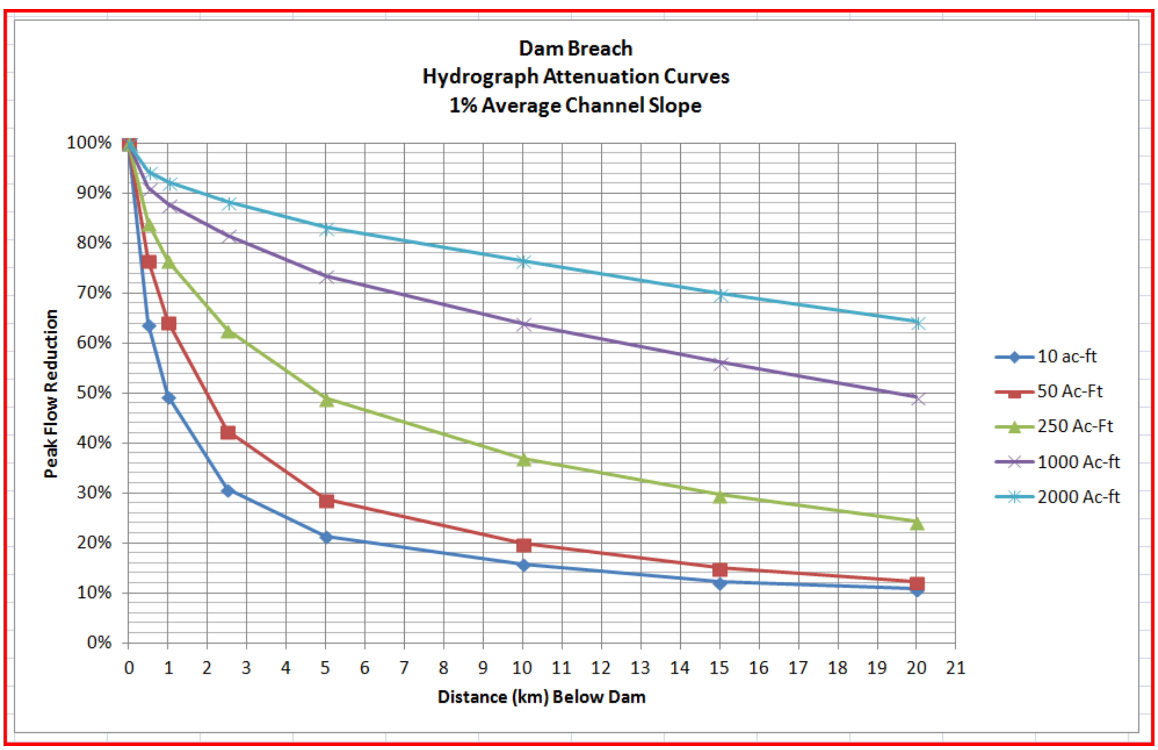


Figure 3: Hydrograph Attenuation Curves – 0.5% Average Channel Slopes

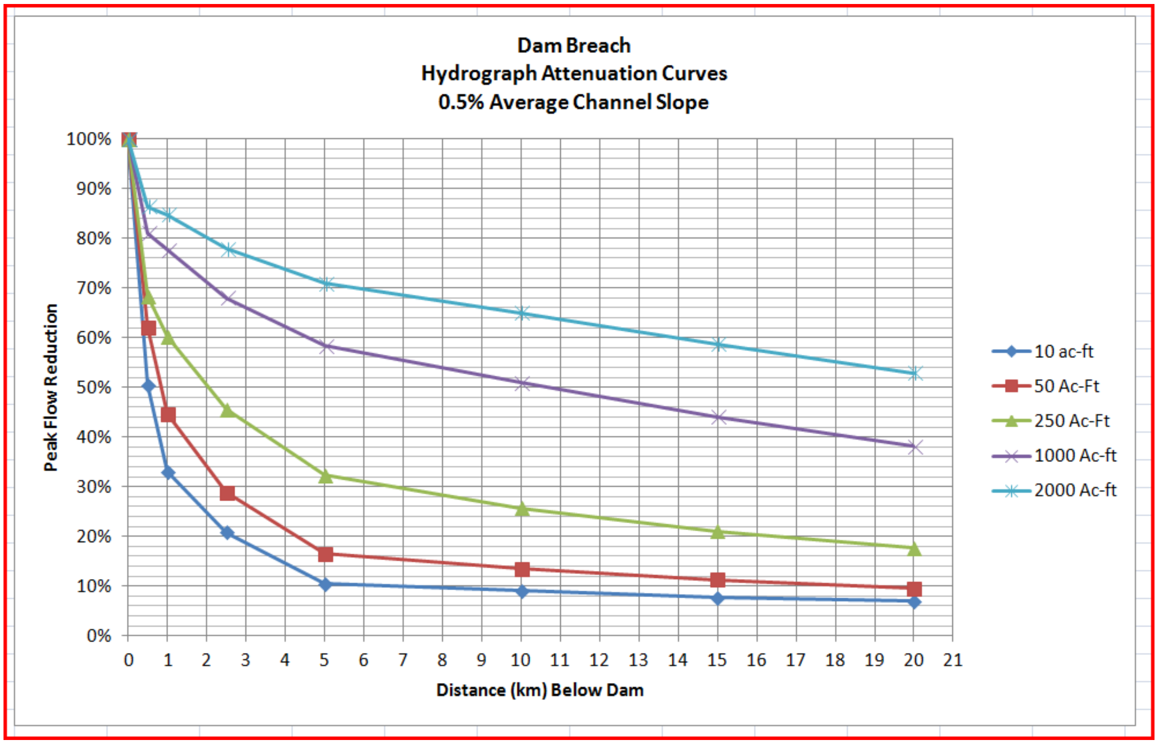
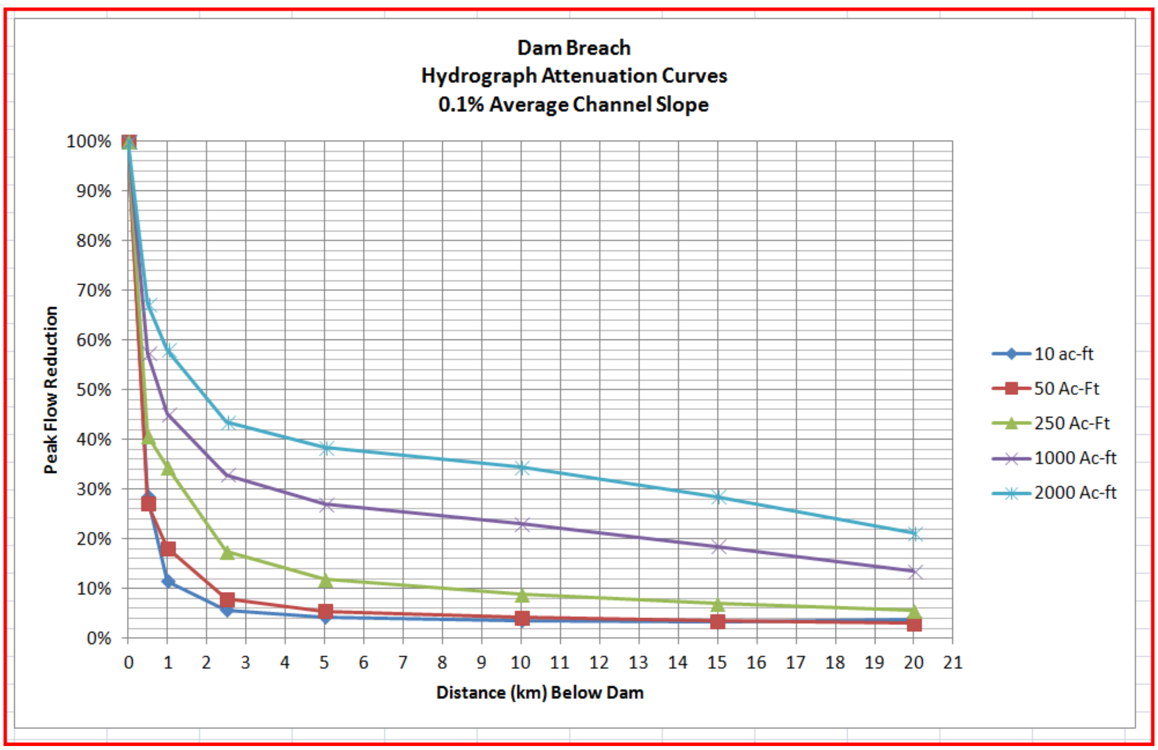


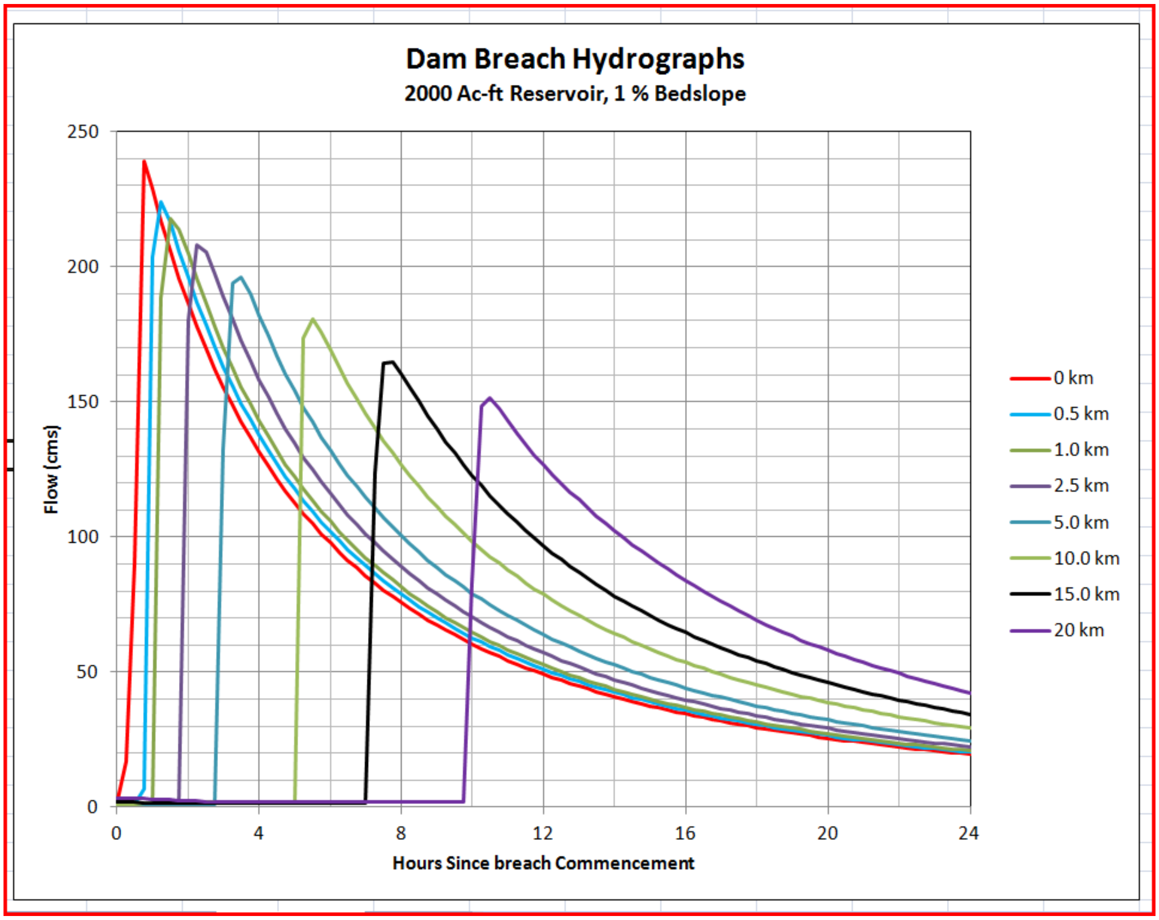
Figure 4: Hydrograph Attenuation Curves – 0.1% Average Channel Slopes



Downstream Inundation

Another critical piece of information when estimating consequences is how much warning time will there be before the flood arrives, and perhaps how long will the area be under water. Certainly the warning time will affect the estimate of the loss of life. Figure 5 shows a series of hydrographs for a relatively large reservoir flowing down a uniformly medium to steep stream (1% average bedslope).

Figure 5: Dam Breach Hydrograph - 2000 Acre Feet Reservoir



For many planning purposes a reasonable approximation of the inundation at a given location can be made using peak dam breach discharge from Tables 1 & 2, the attenuation curves in Figures 1-4, site specific channel cross-section data and representative flow velocities from Table 3.

Table 3 - Representative Flow Velocities for use in Estimating Inundation

(from Dam Break Floods)

Channel Type	1		2		3	
Main Channel	Gravel		Gravel, cobbles		Gravel, cobbles & boulders	
Overbanks	Grass, pasture		Irregular brush & scattered shrubs		Wooded	
	Bedslope %	Velocity m/s	Bedslope %	Velocity m/s	Bedslope %	Velocity m/s
	0.1%	0.7	0.1%	0.5	0.1%	0.4
	0.2%	1.0	0.2%	0.7	0.2%	0.6
	0.3%	1.2	0.3%	0.9	0.3%	0.7
	0.4%	1.5	0.4%	1.1	0.4%	0.8
	0.6%	1.8	0.6%	1.3	0.6%	1.0
	0.8%	2.0	0.8%	1.5	0.8%	1.2
	1.1%	2.5	1.1%	1.8	1.1%	1.4
	1.5%	2.9	1.5%	2.1	1.5%	1.6
	1.9%	3.2	1.9%	2.3	1.9%	1.9
	3.8%	3.7	3.8%	3.3	3.8%	2.6
	5.7%	3.7	5.7%	3.7	5.7%	3.2
	7.6%	3.7	7.6%	3.7	7.6%	3.7

The cross sectional channel area required to pass the flood would be:

$$A = Q_x / V$$

where:

A = Cross-sectional area of channel and overbank (m2)

Q_x = Peak flood discharge (m3/s)

V = Representative average velocity (m/s) at the cross-section

The resulting inundation mapping should represent a conservative estimate of the consequences of a dam failure.

Downstream Consequence of Failure Classification

Once the dam breach flood inundation path has been determined, the resulting consequence of failure classification can be determined. For BC, the classification system is outlined in Schedule 1 “Dam Failure Consequences Classification” of the Dam Safety Regulation. A **“Downstream Consequence of Failure Classification Interpretation Guideline for Dam Safety Officers”** has been prepared as a companion document to this guideline.

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Note (October 2014): Brian Nuttall, Senior Dam Safety Officer and Jeptha Ball, Hydrotechnical Engineer from the Thompson Okanagan Regional Operations Division undertook numerous on-site dam break studies of the downstream impact of the failures of beaver dams. Information on the results of these studies is available from the Thompson Okanagan Regional Operations Division. The intent is to compile the results of this work into a summary report.