EXECUTIVE SUMMARY

Introduction and Background

This study provides assessment of safety and economic geohazard risk, and development of conceptual risk control options, for 35 creeks within the District of North Vancouver (DNV). The creeks assessed are subject to flood, debris-flood or debris-flow processes (debris geohazards) with the potential to cause economic damages and loss, and/or pose risk to life for persons within buildings.

BGC's work was subdivided into "urban creeks" and "Indian Arm creeks" for the purpose of defining the scope of work. The urban creeks included creeks accessible by road and serviced by DNV's stormwater management infrastructure, and the Indian Arm creeks included all creeks on Indian Arm north of and including Sunshine Creek. The urban creeks received the highest level of detail of assessment to support planning decisions for stormwater management infrastructure. Percy Creek also received detailed assessment due to the presence of development on a fan subject to highly destructive debris flows.

For the urban creeks, this study included the following scope of work:

- Assess debris geohazards including their frequency, magnitude, extent, and potential to result in blockage and overflow of DNV stormwater management infrastructure.
- Numerically model debris geohazard scenarios and create maps showing the estimated extent and potential severity of impact to buildings and infrastructure.
- Estimate the risk posed by these hazards to buildings and persons within buildings.
- Prioritize locations for risk reduction planning based on results of the risk assessment.
- Identify possible risk control measures and estimate costs of such measures to inform DNV policymaking deliberations and creation.

For the Indian Arm creeks, this study included the following scope of work:

- Re-evaluate existing estimates of debris-flow or debris-flood frequency and magnitude in light of new advances in debris-flow science, and identify significant (defined as greater or equal to factor of 2) differences in magnitude in comparison to previous work.
- Prioritize the Indian Arm creeks for further risk assessment, but no completion of quantitative risk analyses.
- Provide preliminary risk management options for each creek.

BGC assessed both safety and economic risk to buildings and persons within buildings due to debris geohazards. The safety risk assessment included assessment of risk to individuals and groups (societal risk), and identified cases where the estimated risk level exceeds risk tolerance thresholds defined by DNV. The economic risk assessment was limited to assessment of direct building damages due to debris geohazard events. Following consideration of safety risk, the consideration of economic risk will be helpful in DNV's policymaking deliberations regarding these risks.

BGC quantified safety and economic risk for each creek, and semi-quantitatively estimated economic risk associated with specific culvert blockages. The two approaches used are compatible with the APEGBC's Professional Practice Guidelines for Legislated Landslide Risk Assessments for Proposed Residential Developments in B.C. (2010) and the Guidelines for Legislated Flood Assessments in a Changing Climate in BC (2012). They are also consistent with other Canadian landslide and risk management guidelines[1] in that they provide a transparent, repeatable method to assess risk, define thresholds for risk tolerance, evaluate potential debris-flow mitigation alternatives, and describe uncertainties. Other jurisdictions where risk assessment is an established standard of practice, such as Hong Kong and Australia, use similar frameworks.

BGC assessed risk at two geographic scales: individual assets and watersheds. The asset level assessment supports risk reduction planning for individual assets (e.g., should culvert "X" be higher priority, from a risk perspective, than culvert "Y"). The watershed level assessment supports risk reduction prioritization for the entire creek (e.g., should creek "X" be higher priority, from a risk perspective, than creek "Y"). Together, the site-specific and watershed scale assessments will help DNV in prioritizing mitigation works from a debris hazard perspective.

Risk Assessment Results

Table E-1 lists individual risk results for buildings that exceed DNV's individual risk tolerance thresholds on the urban creeks and Indian Arm creeks. Best-estimate values shown in red exceed DNV's threshold for existing development. Best-estimate values are also shown in orange where they would have exceeded DNV's threshold for new development. Percy Creek was the only urban creek where estimated risk levels fell within the unacceptable range when compared to international risk tolerance standards for societal (group) risk.

Of the creeks listed in Table E-1, Scott-Goldie, Shone/Underhill, Holmden and Friar Creeks were assessed as part of the Indian Arm creeks scope of work. While hazards were re-evaluated with new advances in debris-flow science, the risk estimates shown in the table are from BGC (2009). At Scott-Goldie Creek, this assessment interprets hazard levels as lower than previously assessed (KWL 2003). While not quantified, this implies the actual level of safety risk is lower than that listed in Table E-1. For Shone/Underhill creeks, re-evaluation of hazard characteristics suggests that safety risk due to direct building impact may be lower than previously estimated. However, additional hazard scenarios were identified that had not been considered in previous risk estimates, including bank erosion and long-term channel aggradation, that may affect risk estimates for buildings adjacent to the active channel. For Holmden and Friar creeks, the level of safety risk is still estimated to exceed the DNV risk tolerance threshold for existing development.

[1] e.g., The Draft Provincial Guidelines for Steep Creek Risk Assessments in Alberta (BGC Engineering, 2015), CAN/CSA Q850-97, and the Canadian Landslide Assessment Guidelines.

Table E-1. Number of parcels on the assessed creeks where estimated individual risk exceeds DNV risk tolerance criteria for existing or proposed development.

Creek	Asset ID	Address or Legal Description	Best-Estimate Risk of Fatality Per Year
Cleopatra Creek	BLDG11848	2755 PANORAMA DR	1.5E-05
Gallant Creek	BLDG14092	2150 BADGER RD	4.5E-05
Gavles Creek	BLDG12034	2683 PANORAMA DR	1.0E-04
	BLDG12061	2679 PANORAMA DR	1.0E-04
	BLDG25265	2672 PANORAMA DR	1.5E-05
Mathews Brook	BLDG12267	2603 PANORAMA DR	1.5E-05
Mission Creek	BLDG02481	310 NEWDALE CRT	6.0E-05
Panorama Creek	BLDG12516	2525 PANORAMA DR	9.0E-05
Percy Creek	BLDG22671	326 SASAMAT LANE	3.1E-04
	BLDG22672	327 SASAMAT LANE	6.4E-05
	BLDG22673	328 SASAMAT LANE	6.4E-05
	BLDG22674	330 SASAMAT LANE	6.4E-05
	BLDG22676	332 SASAMAT LANE	7.6E-05
	BLDG22677	333 SASAMAT LANE	8.1E-05
	BLDG22678	334 SASAMAT LANE	8.1E-05
	BLDG22682[1]	338 SASAMAT LANE	1.4E-05
	BLDG24327	335 SASAMAT LANE	7.5E-05
	BGCBLDG00001[2]	-	5.6E-03
Scott-Goldie Creek[3]	BLDG02865	301 SASAMAT LANE	1.7E-04
Shone / Underhill Creeks[3]	BLDG22738	LOT 3 BLOCK B DISTRICT LOT 812 PLAN 10914	1.0E-04
	BLDG22739	LOT 4 BLOCK B DISTRICT LOT 812 PLAN 10914	1.0E-04
	15 Cabins	-	1.0E-04 to 2.0E-04
Holmden Creek[3]	1 Cabin	DISTRICT LOT 871, PLAN 996 AND DISTRICT LOT 871, PLAN 2860	1.2E-03
Friar Creek[3]	BLDG22743	LOT 14 DISTRICT LOT 873 PLAN 3427	4.1E-03

Notes:

1 BLDG23261 is immediately adjacent to Vapour Creek and BLDG22682 is approximately 20 m south of Vapour Creek outlet. The estimated individual risk considers only Percy Creek and does not include some unquantified level of risk from Vapour Creek.

2 This is a cottage at the outlet of Percy Creek, which is not accessible by road. No DNV-assigned Asset ID exists for this building and it has not been 100% confirmed that it is occupied.

3 These creeks were assessed as part of the “Indian Arm creeks” scope of work. The risk estimates are from BGC (2009).

Table E-2 lists estimated annualized direct building damage costs due to debris floods for a given creek, considering all hazard scenarios identified in this assessment. Because the annualized costs consider the hazard frequency, highly damaging but rare events may have a lower annualized cost than more frequent but less damaging events. Thain, Coleman, Canyon, McCartney, Kai, Cove, and Martin Creeks were estimated to have limited to no potential for downstream impacts to infrastructure, and are thus not shown in Table E-2.

Table E-2. Annualized direct damage costs for urban creeks.

Creek	Annualized Damage Cost Estimate
Mackay Creek	$92,000
Gallant Creek	$90,000
Kilmer Creek	$83,000
Thames Creek	$79,000
Mission Creek 3	$18,000
Mosquito Creek	$14,000
Cleopatra Creek	$12,000
Mission Creek	$10,000
Gavles Creek	$10,000
Mission Creek 2	$10,000
Percy Creek	$4,800
Unnamed Creek	$4,800
Mathews Brook	$3,800
Ward Creek	$3,800
Panorama Creek	$2,800
Taylor Creek	$1,900
Allan Creek	$1,600
Ostler Creek	$1,300
Hastings and Dyer	$400
Francis Creek	$100

Risk Control

BGC's risk control assessment developed options to reduce economic and safety risks. This included:

- Risk control design considerations applicable to all creeks.
- General guidance for sediment management and design of culvert inlet debris barriers and trash racks.
- Conceptual-level design options, relative cost, and potential risk reduction that could be achieved at identified sites that do not currently meet DNV's safety risk tolerance criteria or DNV's design standard of passing the 200-year instantaneous flow.

Possible risk control design elements provided in this report for DNV consideration for different creeks employ one of more of the following approaches to risk:

- <u>Debris Control</u> – methods including sediment basins, check dams, and culvert inlet protection to limit the volume of mobilized debris and protect culvert inlets from blocking. Sediment basins or checks dams are proposed at eight creek systems considered in this study.
- <u>Conveyance</u> – channel or culvert upgrades to reduce avulsion potential and methods to increase culvert capacity and allow passage of sediment. Replacement of an existing culvert with a larger diameter culvert is a common risk reduction option proposed at more than 50 sites considered in this study.
- <u>Designated Overflow</u> – methods to direct excess flow to a designated overflow area or channel, and flood protection at individual buildings, to reduce the area of impact and associated consequences. This option tends to incur less capital cost than other options, and is listed as a risk control option at 14 creeks.
- <u>Watershed Drainage Area</u> – methods to reduce the watershed drainage area and resulting peak discharges by diverting water captured in upper areas of the watershed. Frequent inspection of upper watershed drainage elements is also recommended to manage risk water diversion in upper watershed areas.
- <u>Operations and Maintenance</u> – routine maintenance for assuring full flow capacity at culverts, and for preparation of emergency response plans to be implemented during forecasted high runoff events. Routine inspection and maintenance is recommended at all creeks and all assets.

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LIMITATIONS

BGC Engineering Inc. (BGC) prepared this document for the account of District of North Vancouver (DNV). The material in it reflects the judgment of BGC staff in light of the information available to BGC at the time of document preparation. Any use which a third party makes of this document or any reliance on decisions to be based on it is the responsibility of such third parties. BGC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

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1.0 INTRODUCTION

1.1. General

Since the landslide along the Berkley Escarpment on January 19, 2005, BGC has been working with the DNV to develop a risk-based landslide geohazard management system. Through this work the DNV has become the Canadian municipal leader for active geohazard risk management.

Following a damaging November 2014 rainstorm on Kilmer Creek (see Appendix A), DNV requested a district-wide assessment of creeks that could block DNV stormwater management assets and/or damage buildings and infrastructure. Drawings 01 and 02 show the creeks assessed, which include 35 creeks prone to debris flows, debris floods, or floods transporting debris (debris geohazards) with the potential to cause economic damages and loss, and/or that pose risk to life for persons within buildings.

BGC's work was subdivided into "urban creeks" and "Indian Arm creeks". The urban creeks included creeks accessible by road and serviced by DNV's stormwater management infrastructure, and the Indian Arm creeks included all creeks on Indian Arm north of and including Sunshine Creek. The urban creeks, as well as Percy Creek, were assessed in greatest detail.

For the urban creeks, the objectives of this study were to:

- Assess debris geohazards including their frequency, magnitude, extent, and potential to result in blockage and overflow of DNV stormwater management infrastructure.
- Numerically model debris geohazard scenarios and create maps showing the estimated extent and potential severity of impact to buildings and infrastructure.
- Estimate safety and economic risk posed by these hazards to buildings and persons within buildings.
- Prioritize locations for risk reduction planning based on results of the risk assessment.
- Identify possible risk control measures and estimate costs of such measures to inform DNV policymaking deliberations and creation.

For the Indian Arm creeks[2], the objectives of this work were to:

- Re-evaluate existing estimates of debris-flow or debris-flood frequency and magnitude in light of new advances in debris-flow science, and identify significant (defined as greater or equal to factor of 2) differences in magnitude in comparison to previous work.
- Assess whether the differences in hazard characteristics identified in this study could result in changes to the risk profile if new risk assessments were completed.
- Prioritize the Indian Arm creeks for further risk assessment.
- Provide preliminary risk management options for each creek.

[2] This includes all creeks on Indian Arm north of and including Sunshine Creek, except for Percy creek. Percy creek was assessed in detail. These objectives reflect the focus of the work on assessment of DNV's stormwater management infrastructure, as confirmed in a February 16, 2016 email to DNV.

BGC used both quantitative and semi-quantitative approaches to assess economic and safety risk associated with debris geohazards. The approaches used are compatible with the APEGBC Professional Practice Guidelines for Legislated Landslide Risk Assessments for Proposed Residential Developments in B.C. (2010) and the APEGBC Guidelines for Legislated Flood Assessments in a Changing Climate in BC (2012). They are also consistent with the Draft Provincial Guidelines for Steep Creek Risk Assessments in Alberta (BGC Engineering 2015), Canadian and international guidelines for risk management (CAN/CSA Q850-97), and the Canadian Landslide Assessment Guidelines in that they provide a transparent, repeatable method to assess risk, define thresholds for risk tolerance, evaluate potential debris-flow mitigation alternatives, and describe uncertainties. Other jurisdictions where risk assessment is an established standard of practice, such as Hong Kong and Australia, use similar frameworks.

Table 1-1. Risk management framework (adapted from CSA 1997, AGS 2007, ISO 31000:2009).

<table>
<tr><td rowspan="6"><u>Risk Communication and Consultation</u>
By way of maps, reports, signage, warning systems, public meetings, and educational materials</td><td>1. Project Initiation
a. Recognize the potential hazard
b. Define the consultation zone (study area) and level of effort
c. Define roles of the client, regulator, stakeholders, and QRP
d. Determine ‘key’ risks to be considered in the assessment</td><td rowspan="6"><u>Land Management Planning and Regulation</u>
Ongoing review of the risk management process for land use and development permiting</td></tr>
<tr><td>2. Hazard Assessment
a. Identify and characterize the hazard
b. Develop a hazard frequency-magnitude relationship
c. Identify hazard scenarios to be considered in risk estimation
d. Estimate hazard extent and intensity parameters for each scenario</td></tr>
<tr><td>3. Risk Assessment
a. Characterize elements at risk and determine vulnerability criteria
b. Estimate risk: the probability that hazard scenarios will occur, impact elements at risk, and cause particular consequences.</td></tr>
<tr><td>4. Risk Evaluation
a. Compare the estimated risk against tolerance criteria
b. Prioritize risks for risk control and monitoring</td></tr>
<tr><td>5. Risk Control
a. Identify options to reduce risks to levels considered tolerable.
b. Select option(s) providing the greatest risk safety and economic reduction at least cost</td></tr>
<tr><td>6. Action
a. Implement chosen risk control options
b. Define ongoing monitoring and maintenance requirements</td></tr>
</table>

1.2. Terminology

In this report, we use the following general definitions from APEGBC (2012), Canadian Standard Association (CSA) (1997), Australian Geomechanics Society (AGS) (2007), and Hungr et al. (2001). Additional terms are defined where used in the text. Further, more detailed discussion of hydrogeomorphic flood processes is provided in Appendix B.

Assessment data: these are tabular data describing property ownership, usage, and assessed value. Linked to cadastral boundaries, the assessment data provide information about the type and value of building “improvements” on a parcel.

Buildings data: these data show the spatial extent (polygons) of buildings within the DNV. Attributes linked to this geospatial layer and considered in the analysis include the type of building and estimated number of residents.

Cadastral data: these data show the spatial extent of parcels (property boundaries) in GIS polygon shapefile format.

Clear-water flood: flow of water in a channel with a relatively low proportion of debris compared to debris floods. The word “flood” in this term, as used in this report, does not necessarily imply flow outside the normal stream course (i.e., as would be described by “flooding” of properties or buildings).

Debris: sediment (e.g., sand, gravel, boulders) or organic material (e.g., trees or other vegetation). In the context of this assessment, “debris” relates to sediment and wood moved by water in steep creeks.

Debris flow: very rapid to extremely rapid flow of saturated non-plastic debris in a steep channel. Debris flows typically require a channel steeper than about 30% for transport over long distances and have volumetric sediment concentrations typically in excess of 50-60%.

Debris flood: very rapid surging flow of water and debris in a steep channel. Debris floods typically occur on creeks with channel gradients between 3 and 30% and have a lower proportion of debris compared to debris flows.

Debris hazard (geohazard): the continuum of floods, debris-floods and debris-flows (referred to as hydrogeomorphic processes) with their associated phenomena of channel bed scour, bank erosion, avulsion and debris deposition, that have the potential to cause economic damages, injury and potential loss of life.

Economic Risk: measure of asset or business loss risk associated with a geohazard event.

Elements at Risk: persons or infrastructure potentially exposed to hazard.

Fan: landform at the outlet of steep creeks created and modified by the deposition of sediment from the upstream watershed (Figure 1-1). Deposition occurs where the channel gradient decreases and the creek loses confinement. *Alluvial fan*s are formed predominantly by fluvial processes: that is, by flowing water. *Colluvial fans* are formed predominantly by landslide processes, including debris flows. Many fans are *composite fans* in which several hydrogeomorphic processes interact at different time scales and magnitude. Relatively few “classically” shaped fans exist within the DNV, and most of these are located at the outlet of creeks along Indian Arm.

Figure 1-1. Typical low-gradient and steep fans feeding into a broader floodplain. On the left a small watershed prone to debris-flows has created a steep fan that may also be subject to rock fall processes. Residential developments and infrastructures are shown to illustrate their interaction with hydro-geomorphic events. Artwork: Derrill Shuttleworth.

Hazard (geohazard): earth surface process with the potential for causing harm, in terms of human safety, property, the environment, other things of value, or some combination of these.

Hazard Frequency: average annual probability of occurrence of a geohazard. Annual hazard frequency is the inverse of return period for events occurring less than once per year. For example, an event with a return period of 100 years would have an annual frequency of 1:100, or 0.01. This implies a chance of approximately 1% occurrence in any given year.

Hazard Scenario: hypothetical scenarios where flows occur outside the normal creek channel in areas where they can impact development or infrastructure.

Hydrogeomorphic process: steep creek process whose dominant driver is water, but with varying sediment concentrations. This term includes the spectrum of clear-water flood, debris flood, and debris flow process types (Figure 1-2). Many steep creeks are subject to several hydrogeomorphic processes at different time scales and magnitude. Note that clear-water flood processes still transport debris, but at lower concentrations than debris flows or debris floods.

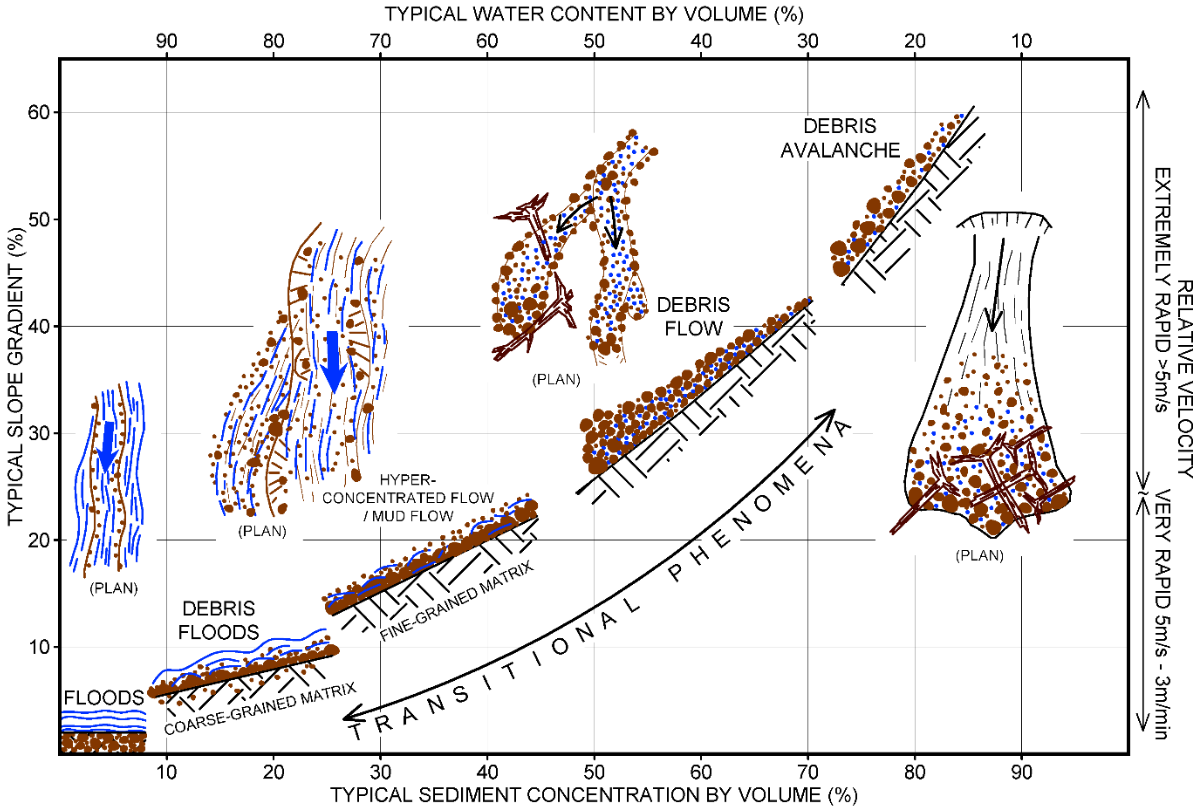

Figure 1-2. Schematic illustration of debris-transporting processes with different water and sediment concentrations.

Safety Risk. measure of risk to life associated with a geohazard event, assessed for individuals ("individual risk") or groups of individuals ("group risk").

Steep creek: creek containing channel gradients equal to or exceeding approximately 5%. Channel gradients are typically lowest on fan slopes and higher near the headwaters. Some sections of a given creek might meet the "steep creek" gradient criterion; others might not.

Steep creek hazard: restricted in this report to include hydrogeomorphic processes on steep creeks that have the potential to result in undesirable consequences.

1.3. Scope of Work

BGC's scope of work was described in a proposal dated April 10, 2014 and is being carried out under the terms of DNV Professional Services Agreement No. 96726 dated July 30, 2015. Table 1-2 lists the creeks assessed.

The creeks assessed are defined as "steep creeks" subject to flood, debris-flood or debris-flow processes, with the potential to cause economic damages and loss, and/or pose risk to life for persons within buildings. Stormwater management infrastructure assessed on these creeks include culverts and stormwater mains owned by DNV that are considered to have credible potential for debris blockage and/or that are located in creek sections subject to debris hazard processes.

BGC also assessed culverts in upper watershed areas that are not owned by DNV, but that affect watershed boundaries and associated creek flows into DNV. However, hazard scenarios and risks associated with blockage of these culverts were not assessed in detail (see Recommendations Section 9.0).

The web application accompanying this report ("DNVHIT", see Section 1.4) identifies culverts as "in scope" if they were assessed for creek blockage, or are located in upper watershed areas and received field inspections.

The creek names have been assigned in accordance with the gazetted creek names listed in BC Geographical Names, where available. Where no gazetted creek name is available, the creek names have been assigned in accordance with those used in previous assessments.

Drawings 01 and 02 show study creek locations. Of the 35 creeks assessed, 20 are classified as prone to debris floods, 6 to debris flows, and 9 to flood processes. All creeks prone to debris flows or debris floods are also prone to floods. Appendix B provides background information on hazard process types.

Table 1-2. Study creeks, listed from west to east.

Creek	Process	Location
Mackay	Debris flow	West of Lynn Creek
Mosquito	Debris flood	
Mission	Debris flood	
Thain	Debris flood	
Hastings[3]	Debris flood	
Dyer	Debris flood	
Kilmer	Debris flood	
Thames	Debris flood	
Canyon	Flood	East of Lynn Creek
McCartney	Flood	
Taylor	Flood	

[3] Hastings Creek has been identified as Dunell Creek in previous assessments. The name has been updated in accordance with the gazetted name listed in BC Geographical Names as requested by DNV.

Creek	Process	Location
Gallant	Debris flood	Deep Cove
Panorama	Flood	
Kai	Flood	
Matthews Brook	Debris flood	
Gavles	Debris flood	
Cove	Flood	
Cleopatra	Debris flood	
Martin	Flood	
Francis	Debris flood	
Unnamed Creek 2	Flood	Indian Arm (road accessible)
Ward	Flood	
Ostler	Debris flood	
Allan	Debris flood	
Sunshine	Debris flood	
Scott Goldie	Debris flood	
Percy	Debris flow	
Vapour	Debris flood /Debris flow [4]	
Gardner Brook	Debris flood	Indian Arm (not road accessible)
Shone	Debris flow	
Underhill	Debris flow	
Ragland	Flood	
Holmden	Debris flow	
Coldwell	Debris flood	
Friar	Debris flow	
Clegg	Debris flow	

The scope of work includes assessment of safety and economic risk, and determination of conceptual risk control options. For each creek, the major tasks are as follows:

[4] The hydrogeomorphic process changes from debris flood to debris flow at higher return periods.

- Hazard assessment: Hazard characterization followed by frequency-magnitude analysis, flow modelling and preparation of hazard intensity maps for each for each creek (Sections 3.0 to 5.0).
- Risk assessment: Estimation of risk due to debris impact for the elements at risk listed in Table 1-3 (Section 6.0).
- Risk control assessment: Identification and prioritization of conceptual options to reduce damages and loss, recommendations regarding preferred options, and description of work to implement the preferred risk control options (Section 7.0).
- Conclusions and recommendations: Summary of major findings and recommendations (Sections 7.5 and 0).
- Presentation to District Council and participation in a public open house.

The assessment is based on a review of previous work and collection of additional field and desktop-based hazard information. The scope of work is primarily focused on the urban creeks serviced by DNV's stormwater management infrastructure, to support risk reduction planning for these areas. As such, the majority of project effort is focused on the urban creeks. Assessment of Indian Arm creeks included review of hazard information collected by KWL (2003b, 2003d, 2003g, 2003h, 2003i, 2003j), BGC (2009) and KWL (2011), desktop-based analyses of regional hazard frequency-magnitude relations, and one day of shoreline observations completed in November 2015.

Other creeks exist within the DNV that are not assessed and that may be subject to risk of inundation from flood processes. However, these creeks fall outside the scope of this report as they are not considered steep creeks. One such example is Parkside Creek located to the south of Gallant Creek. Flood issues were noted on this creek by DNV during the November 2014 storm event and may exist in the future in conjunction with flood processes; however, this type of creek falls outside the defined scope of this report.

It is also not possible to identify and assess every conceivable hazard scenario leading to a loss associated with debris hazards in DNV. Rather, the assessment considers key scenarios and losses that can form the basis to prioritize areas and make practical decisions on risk reduction. The resulting risk reduction measures, once implemented, will reduce risk for a broader range of hazard scenarios than are possible to directly assess. As a baseline estimate, the assessment considers existing conditions and does not consider emergency response measures such as evacuation or emergency mitigation (e.g., sand bags or culvert cleanout).

Table 1-3 lists the types of elements at risk and types of consequences assessed. Specifically, "safety risk" considers risk to life for persons within buildings. "Economic risk" considers direct damages to buildings due to debris impact. Assessment of drainage infrastructure focuses on identifying requirements and costs to upgrade assets as part of debris risk management. Assessment of other types of infrastructure was limited to identifying their location in relation to areas impacted by debris hazard scenarios.

Table 1-3. Elements at risk.

Element at Risk	Type of Risk Assessed
Persons within Buildings	• Quantitative estimation of individual and group risk to life for persons located within buildings.
Buildings	• Quantitative estimation of damage to buildings due to debris impact expressed as a proportion of appraised building value and direct damage cost, and as an annualized damage cost. • Identification of facilities considered critical by DNV for function during an emergency that are located within areas potentially subject to debris geohazard impact. These include schools, police stations, or fire stations.
Roads	• Estimation of spatial extent and intensity of impact by debris geohazard scenarios.
Culverts	• Estimation of event return periods likely to result in blockage by debris for a given culvert.

Concurrently with BGC's scope of work, Northwest Hydraulic Consultants (NHC) is presently developing a drainage model for the DNV. This work includes estimation of peak flows for various return periods (2, 10, 100 and 200) at a majority of culvert locations within the DNV. This work has not been completed at the time of issue of this report. BGC's estimates of peak flows are subject to change once final estimates are available from NHC.

BGC has relied upon third-party sources for base map information (e.g., topographic and hydro network data) and data describing the location and characteristics of elements at risk (e.g., DNV development infrastructure). These data will need to be updated with new development or any terrain alterations. BGC's review of data accuracy and completeness should not be considered exhaustive: errors or omissions in third-party data sources may exist that were not identified.

1.4. DNV Hazard Information Tool

The DNV Hazard Information Tool (DNVHIT) is an online-accessible, interactive map that displays DNV infrastructure, study creeks and the hydrogeomorphic hazards identified and characterized by BGC for DNV.

Additional information regarding the DNVHIT and how to use it can be found in Appendix C.

2.0 DNV INFRASTRUCTURE

BGC considered DNV buildings, drainage management works and transportation infrastructure in the assessment of debris geohazard impact. All of these assets are classified by DNV using a unique identifier (Asset ID). For ease of reference, BGC used the same identifier during inspections, geohazard and risk assessments as well as to report risk control recommendations for a given asset. Other types of infrastructure, such as power infrastructure (transmission lines) or oil/gas infrastructure, were not assessed.

2.1. Buildings

DNV provided BGC with geospatial data of building locations and characteristics throughout the district. The data included building footprints (dated 2010) and attributes by DNV for a NRCAN earthquake risk assessment (Journeay et al. 2015), and data from the BC Assessment Office. All data are associated with a unique Building ID and geospatially related to building footprints in GIS. Table 2-1 outlines the building data incorporated into this study.

BGC used classifications of building type, population (estimated number of occupants), and estimated replacement value across DNV by Journeay et al. (2015) as part of a seismic risk assessment. The building inventory was developed from a combination of DNV information and windshield surveys[5] completed by Carlos Ventura and colleagues at the UBC Earthquake Research Facility. Replacement costs were based on standard methods from the Canadian version of Hazus (earthquake model), a damage estimation tool developed by the U.S. Federal Emergency Management Agency (FEMA).

Table 2-1. Building data incorporated in this study.

Data	Source
Building footprint	DNV[1]
Cadastral parcel (property footprint)	DNV[1]
Building type (general occupancy class), estimated number of occupants, estimated replacement value of building and contents	NRCAN (2015)
2015 BC Assessment land and improvement value	DNV[1]

Note:

1. Supplied in August 2015.

2.2. Drainage Infrastructure

Drainage infrastructure within the current DNV asset inventory includes 371 culverts, 7 natural hazard mitigation structures[6], and approximately 350 kilometres of storm mains with replacement costs totalling $296 million (DNV 2015). An additional 86 culverts also exist within the DNV that are owned by others (e.g., the Province of BC). These form part of a larger array of water

[5] Windshield surveys are systematic observations made from a vehicle.
[6] Mackay Creek debris basin, Mosquito Creek Evergreen Basin, upper Mosquito Creek debris net, Inter River landfill area dike, bank protection at the OC Works Yard and inside William Griffin trail, and the Inter River flood protection berm.

management infrastructure within DNV, associated with wastewater and stormwater management as well as the distribution of potable water.

Table 2-2 lists the number of stormwater mains and culverts assessed on study creeks that received an overflow rating. These include culverts and stormwater mains owned by DNV that are considered to have credible potential for debris blockage, based on the hazard characterization described in Section 5.2.1. The list also includes culverts assessed as having no credible potential for debris blockage, but that are located in upper creek sections subject to debris hazard processes. The list does not include study creek drainage assets located sufficiently far downstream that they do not have credible potential for blockage by debris geohazards. Drainage infrastructure locations are displayed on DNVHIT.

Table 2-2. Drainage infrastructure assessed on study creeks that received an overflow rating.

Creek	Number of Assets Assessed[1]	
	Culverts	Storm Mains
Mackay Creek	9	8
Mosquito Creek	0	0
Mission Creek[2]	10	2
Thain Creek	4	0
Hastings[3] Creek	2	0
Dyer Creek	1	0
Kilmer Creek	6	2
Coleman Creek	0	0
Thames Creek[4]	7	1
Canyon Creek	1	0
McCartney Creek	1	0
Taylor Creek[5]	0	0
Gallant Creek	5	1
Panorama Creek	4	0
Kai Creek	0	1
Matthews Brook	1	2
Gavles Creek	2	0
Cove Creek	3	0
Cleopatra Creek	5	0
Martin Creek	1	0
Francis Creek	1	0
Ward Creek[6]	6	0
Ostler Creek	2	0

Creek	Number of Assets Assessed[1]	
	Culverts	Storm Mains
Allan Creek[7]	4	0
Sunshine Creek	1	0
Scott Goldie Creek	0	0
Percy Creek	0	0
Vapour Creek	0	0
Gardner Brook	0	0
Shone Creek	0	0
Underhill Creek	0	0
Ragland Creek	0	0
Holmden Creek	0	0
Coldwell Creek	0	0
Friar Creek	0	0

Notes:

1. Includes assets which are connected to form a single hydraulic structure
2. Includes culverts identified by BGC upstream of the development interface along Powerline Trail.
3. Hastings Creek was previously identified as Dunnell Creek, the name has been updated in accordance with the naming convention used by GeoBC (2015).
4. Includes second culvert at Mountain Hwy that runs adjacent to STMCUL00052 and together act as a single hydraulic structure.
5. One bridge (VDHBRG00015) was assessed at Anne Macdonald Way.
6. Ward Creek assessed culverts includes the 3 culverts assessed on the Unnamed Creek immediately to the west.
7. Allan Creek bridge BGCVEHBGC00001 downstream of STMCUL00228 was also assessed.

2.3. Transportation Infrastructure

Transportation infrastructure within DNV and applicable to this study includes roadways and pedestrian or vehicle bridges. Pedestrian and vehicle bridges were reviewed by BGC where they intersect with a study creek upstream of development or within the area modelled. However, pedestrian bridges were not prioritized for evaluation of risk reduction alternatives. No vehicle bridges were identified as having credible potential for debris blockage. DNVHIT shows roads, highways and bridges within DNV.

3.0 HAZARD CHARACTERIZATION

3.1. General

The objective of the geohazard assessment is to systematically assess clear-water floods, debris-flood and debris-flow hazards on mountain creeks within DNV that could damage critical infrastructure including buildings, roads, bridges or utilities. The process encompasses geohazard characterization, development of frequency-magnitude relations and modelling of hazard scenarios to be considered during the risk assessment.

During hazard characterization, the creeks are assessed with respect to geographic location, development and local infrastructure, and geomorphic and hydrological characteristics. The process involves compilation and review of previous assessments on the creeks in conjunction with desktop analyses and field investigation. The details of each component are described in the following sections.

3.2. Previous Work

Appendix D lists previous studies conducted within DNV that focus on debris geohazard and risk estimation, assessment of stormwater management and culvert hydraulics, assessment of infrastructure vulnerability, and that provide records of previous water conveyance failures. The outcomes of the review include:

- Compilation of material that remains current and applicable to the current assessment.
- Identification of data gaps and limitations to be addressed by the current assessment.
- Identification of data gaps and limitations that lie outside the scope of work but affect the methodology or expected results of the study, and that should be considered for future assessments.

BGC also reviewed previous mapping of clear-water flood, debris flood, debris flow, landslide and other slope movement features as documented by Kerr Wood Leidal (1995, 2003a-k, 2011), Northwest Hydraulic Consultants (2010), and the Ministry of Environment (1995), as well as previous assessments completed by BGC for DNV.

3.3. Desktop Study

3.3.1. Terrain Analysis

Terrain analysis formed the initial stage of hazard characterization. The analysis involved delineation of creek and creek tributaries, delineation of watershed boundaries and mapping of geomorphic features as described in the sections below.

All terrain analysis was completed using 2013 LiDAR data provided by DNV and 1 m topographic contours.

3.3.1.1. Creek Delineation

Initial creek delineations were provided by DNV and modified based on terrain analysis coupled with field observations compiled by BGC. In particular, modifications to the creek delineations were completed where the initial version did not extend to the upstream headwaters, where notable tributaries identified during field inspections had not been delineated, or where anthropogenic modification of the landscape had altered the drainage pattern. DNVHIT shows the updated creek lines and tributaries. BGC also assigned creek "chainage" (distance measurements at regular intervals) to each creek for location reference, with the upstream limit as the zero point.

3.3.1.2. Watershed Delineation

Watershed delineation was completed within ArcGIS using watershed boundaries calculated using Global Mapper v15.2 and BASINS 4.1 (United States Environmental Protection Agency (EPA) 2013). This work was supplemented by field observations of local and regional drainage patterns and reference to previous investigations of drainage along roadways or other infrastructure. In particular, a drainage assessment was completed in 2007 along Old Grouse Mountain Highway to facilitate culvert sizing (BGC 2007). The delineated drainage patterns identified during the assessment were included in the present assessment and are shown on DNVHIT.

3.3.2. Hydrology

As outlined in Section 1.3, NHC is presently developing a drainage model for DNV concurrent with the BGC assessment. BGC's scope of work requires these data to complete frequency-magnitude analyses, evaluate culvert blockage potential and design hazard scenarios for debris-flood prone creeks. To minimize impacts to the project schedule, BGC has estimated peak flows for preliminary assessments. These results will be updated once final estimates are available from NHC. The methods used to generate preliminary storm hydrographs for the study creeks are outlined in Appendix E.

3.4. Field Investigation

Field investigations were completed between July and January 2016. Table 3-1 lists the creeks where field observations were collected.

The majority of study creeks visited were hiked from the development interface (e.g., boundary of residential development with the undeveloped upper watersheds) upstream to the point at which the creek had little potential to contribute to sediment transport to development. In many instances, creeks were hiked along the full length to the headwaters in order to delineate the full creek. Select creeks were hiked through development in order to characterize the potential for blockage of culverts and attendant flooding potential (Table 3-1).

Indian Arm creeks are not accessible by road, therefore observations related to the presence and occupancy of structures downstream of the creeks were collected from Indian Arm by boat.

Additional observations were collected at Coldwell Creek and Gardner Brook related to the creek channel and a small dam for local water supply, respectively.

The primary objective of field investigation for the hazard component of this study was to collect parameters supporting debris volume and frequency estimation, as well as hydraulic analyses at culvert crossings. These parameters included:

- Channel morphological characteristics including gradient, width, and bankfull height
- Channel bed characteristics including sediment grainsize distribution and bedrock exposure
- Creek cross sections where high water marks were reconstructable
- Sediment sources including bank erosion, rockfall, and localized slope instability
- Culvert properties including dimensions, gradient, freeboard and condition
- Debris control structure dimensions and estimation of effectiveness in preventing culvert blockage
- Pedestrian footbridge characteristics (e.g., channel geometry at the crossing)
- Observations of infrastructure requiring maintenance.

In addition to the creeks, major roadways affecting regional drainage patterns were visited to delineate the local watershed boundaries. This includes Mt. Seymour Road, Indian River Drive and Old Grouse Mountain Hwy (BGC 2007).

On the Percy Creek fan, four test pits were dug in September 2015 to characterize the debris-flow deposits and collect samples for radiocarbon dating. BGC also collected tree core samples for dendrochronological analysis.

The stormwater infrastructure characteristics collected as part of the field investigation are shown in DNVHIT and summarized in electronic format for delivery to DNV (Appendix F). Additional bridge and creek characteristics were also collected in the field and can be provided on request. They are also shown on DNVHIT.

Table 3-1. Summary of field investigation locations along DNV creeks.

Creek	Location	Previous Hazard Assessment	Data Collection	
			Upstream of DNV development	In DNV development
Mackay Creek	West of Lynn Creek	✓	×[7]	✓
Mosquito Creek		✓	×	×
Mission Creek		×	✓	✓
Thain Creek		×	✓	✓
Hastings Creek		×	✓	N/A[8]
Dyer Creek		×	✓	×
Kilmer Creek		×	✓	✓
Coleman Creek		×	×	✓
Thames Creek		×	✓	✓
Canyon Creek	East of Lynn Creek	×	✓	N/A[8]
McCartney Creek		✓	✓	N/A[8]
Taylor Creek		×	✓	N/A[8]
Gallant Creek	Deep Cove	×	✓	✓
Panorama Creek		✓	✓	N/A[9]
Kai Creek		✓	✓	N/A[9]
Matthews Brook		✓	✓	N/A[9]
Gavles Creek		✓	✓	N/A[9]
Cove Creek		✓	✓	N/A[9]
Cleopatra Creek		✓	✓	N/A[9]
Martin Creek		✓	✓	N/A[9]
Francis Creek		✓	✓	N/A[9]
Ward Creek	Indian Arm	×	✓	✓
Ostler Creek		✓	✓	✓
Allan Creek		✓	✓	✓
Sunshine Creek		✓	×[10]	×
Scott Goldie Creek		✓	×	×
Percy Creek		✓	✓	N/A[9]
Vapour Creek		✓	×	N/A[9]
Gardner Brook		✓	×	✓
Shone Creek		✓	×	✓

[7] Field observations at the Mackay Creek debris barrier and upstream pedestrian bridges were collected.
[8] Downstream of the interface with development, the creek passes through green space.
[9] Deep Cove and Indian Arm Creeks meet the ocean immediately downstream of the intersection with development.
[10] Culvert observations were collected along Sasamat Lane.

Creek	Location	Previous Hazard Assessment	Data Collection	
			Upstream of DNV development	In DNV development
Underhill Creek	Indian Arm	✓	×	×
Ragland Creek		✓	×	×
Holmden Creek		✓	×	✓
Coldwell Creek		✓	✓	✓
Friar Creek		✓	×	✓

3.5. Previous Events

The steep creeks around DNV have a history of flood events. Table 3-2 lists relevant past flood events which affected the study creeks. Appendix A outlines additional information about each event based on reports from local residents, DNV records, news reports and field observations, including a detailed description of the November 2014 event which was used to calibrate BGC's hazard analysis and debris-flood modelling. Drawings A-1 and A-2 show the location of recorded impacts to buildings, properties and infrastructure from the November 2014 event.

Table 3-2. Summary of previous events affecting DNV creeks.

Event	Description
1896 to Present	Mosquito Creek major flood events
1950s	Reports of flooding on Gallant Creek and Mosquito Creek.
October 1981	Storm event affecting DNV and the surrounding area.
November 1989	Debris flood on Shone Creek and flooding on Ostler Creek
Early 1990s	Debris flows on Mackay, Holmden, Underhill, and Allan Creeks. Flooding on Ostler and Shone Creeks.
1995	A November 23 storm led to a debris flow on Mackay Creek and flooding on Deep Cove Creeks. Creek washout on Shone creek reported on November 25.
1998	Debris flows on Holmden, Mackay and Upper Mackay Creeks and flooding on Shone Creek.
Early 2000s Flood Event Gallant Creek	Local reports from Deep Cove indicate flooding on Indian River Drive and Deep Cove Road. In November 2006, overbank flooding occurred on Mackay Creek.
November 2014	Storm event triggering debris flood on Kilmer Creek, blockage of culverts on Thames Creek and Gallant Creek, and a small debris flow on Upper Mackay Creek.
August 2015	Storm event leading to debris removal on Mission Creek and Kilmer Creek.
February 2016	Small debris flow on Mackay Creek.

In addition to the events listed above, DNV maintenance personnel maintain shortlists by area of culvert inlets which have historically required checks for debris blockage during storm events (F. Dercole, DNV, email, June 25, 2015). The culverts and storm mains included in the area short list are identified on DNVHIT and included with the stormwater infrastructure parameters provided in electronic format in Appendix F.

3.6. Hydrogeomorphic Process Assignment

The study creeks are subject to hydrogeomorphic processes whose dominant driver is water with varying sediment concentrations; these include clear water flood, debris flood, and debris flow processes (Jakob et al. 2015). A detailed discussion of hydrogeomorphic processes is included in Appendix B. Identifying the dominant hydrogeomorphic process on a creek is important for detailed assessment of flow magnitude and behaviour, selection of parameters for numerical modelling of flows, selection of criteria to estimate vulnerability, and associated risk and mitigation design.

Table 1-2 (Section 1.3) lists the dominant hydrogeomorphic process type assigned to each study creek. This interpretation is based on fieldwork and desktop study that included:

- Review of previous work
- The geomorphology of fans (where existing), channel deposits and their associated watersheds
- Channel gradients
- Field observations
- Records of previous events.

Table 3-3 provides a more detailed list of geomorphic criteria used to distinguish between debris flow, debris flood and flood process types. Note that while a single dominant process type was assigned to a given creek, some creeks are subject to more than one type of hydrogeomorphic process. In general, the following co-relationships between process types can occur (further discussion is provided in Appendix B). Creeks classified as subject to debris flows may also be subject to floods and debris floods at lower return periods, or debris flows may transition to watery afterflows in the lower runout zone and after the main debris surge. Those classified as subject to debris floods may be subject to clear water floods, but will generally not be subject to debris flows. Those classified as subject to clear water floods were interpreted as not subject to debris floods or debris flows.

Table 3-3. Sediment and geomorphic characteristics for different steep creek processes.

Sediment or Geomorphic Characteristic	Debris Flows	Debris Floods	Floods
Matrix-supported deposit stratigraphy	Yes	Rarely	No
Clast-supported deposit stratigraphy	Rarely	Often	Yes
Inverse grading of deposit (larger particles in a stratigraphic column towards the top)	Yes	No	No
Clast imbrication (clasts “shingled” which is typical for fluvial transport mechanisms)	No	Sometimes	Usually
Defined boulder lobes	Yes	Sometimes, but with less sharp boundaries than for debris flows	No
Boulder levees (ridge-like features running parallel to debris-flow prone creeks where the flow avulsed from the channel)	Yes	No	No
Paired terraces (terraces on both sides of the channel and at the same elevation and presumably same age)	Rarely	Often	Only if stream is incising into alluvial bed
Buried vegetation	Yes	Yes	Sometimes
Impact-scarred riparian vegetation	Yes	Often	Rarely
Creek channel scour	Mostly in transport zone	Yes	Yes
Fine-grained overbank deposits	Rarely	Sometimes	Usually
Channel Gradient	Typically >15°	Typically <15°	Typically <15°

4.0 FREQUENCY-MAGNITUDE ESTIMATION

This section summarizes methods to estimate hazard frequency and magnitude, and combine these to determine frequency-magnitude (F-M) relations for each creek. Hazard frequency is defined as the annual probability of the creek hazard occurring. Hazard magnitude is expressed as the peak flow and the total volume of debris mobilized in the specific hazard event. Appendix G provides more detailed description of methods and results of F-M estimation for urban creeks subject to debris-flood hazards. Appendices H and K describe methods and results of F-M estimation for Percy Creek and Indian Arm Creeks north of and including Scott Goldie Creek, respectively.

Frequency-magnitude (F-M) estimations form the core of any hazard assessment because they combine the findings from frequency and magnitude analyses in a format suitable for numerical analysis. Frequency and magnitude of hydrogeomorphic events are inversely related[11]. The higher the event frequency, the lower its magnitude and vice versa. In short, the rarer an event, the larger it will be.

4.1. Frequency Estimation

Frequency analysis determines how often hydrogeomorphic events occur, on average. Frequency can be expressed either as a return period or an annual probability of occurrence. For example, if five hydrogeomorphic events occur within a 100-year period, the average return period is 20 years. The annual probability is the inverse of the return period, and for this example is equal to 0.05, or a 5% chance that a debris flow may occur in any given year. This logic assumes data stationarity: i.e., no shift in either the mean or variance of the hydroclimate data responsible for triggering and sustaining debris flows. If a statistically significant trend in the conditions leading to debris flows or the debris-flow occurrence itself is detected (e.g., due to climate change), and if there are reasons to believe that such trend will persist in the future, reported return periods for given event volumes would need to be adjusted according to the observed and projected trend. Similarly, it is assumed that hydrogeomorphic events are not clustered in time and are thus occurring fully independently of each other. Section 4.3 discusses the influence of climate change on frequency estimation.

Frequencies of hydrogeomorphic events can be established through a large number of absolute or relative dating methods. Table 4-1 summarizes approaches used in this study and lists the creeks to which these were applied.

[11] This general paradigm stands for all geophysical phenomena (earthquakes, tornados, hurricanes, etc.).

Table 4-1. Frequency estimation methods and applicable creeks.

Method	Applicable Creeks
Flood frequency analysis, based on gauged local watersheds	All debris flood and flood creeks
Radiocarbon dating of organic material in bedded sediments	Percy Creek
Dendrochronology on damaged and post-event trees	Percy Creek
Application of regional frequency-magnitude curve	Scott-Goldie, Shone, Coldwell, Holden, Clegg creeks

4.2. Magnitude Estimation

The objective of the magnitude analysis is to estimate debris volumes and peak discharges of past hydrogeomorphic events. The results form inputs to numerical modelling and assessment of debris blockage at culverts and stormwater mains.

Appendix E describes methods used to estimate clear-water flood peak flows and develop preliminary storm hydrographs. Table 4-2 summarizes approaches used by BGC to estimate debris volumes for creeks subject to debris floods and debris flows, which have the potential to carry much higher sediment loads than clear-water floods. The tabulation is an abridged version of the methods outlined in Appendices G, H and K. For a full discussion, refer to the appropriate appendix.

Sediment bulking (i.e., choosing a multiplier to the peak discharge of debris floods or debris flows from the clearwater flood discharge estimates) was done differently amongst the creeks in DNV. Smaller creeks were not bulked while larger creeks were bulked by a factor of 3 (Jakob and Jordan 2001). The logic behind this decision is that the justification for bulking has been that extraordinary processes occur in watersheds which trigger debris floods. Those include landslide dam failures, landslides evolving into debris flows and eventually into debris floods, or stream bank collapses. Those creeks include Scott-Goldie, Vapour, Gardner Brook and Coldwell.

The other creeks, except from debris-flow prone creeks, are very unlikely to witness the processes listed above and bulking would thus likely lead to overly conservative results.

Table 4-2. Methods employed for estimation of debris magnitudes.

Application	Method	Comment
Flood and debris flood prone streams with a mobile bed but not steep enough to produce debris flows. (*Appendix G*)	• Determine the shear stress threshold (i.e., critical shear stress) required for bed mobilization. • Use average channel dimensions and Manning's equation to determine the discharge that corresponds to the critical shear stress. • Use a hydrograph associated with a specific return period to calculate the amount of time that the flow exceeded the discharge threshold. • Use Rickenmann's (2001) sediment transport equation to calculate sediment discharge based on stream power. • Calculate sediment volume based on the estimated sediment discharge rate multiplied by the duration over which the critical shear stress occurs. • Use of regionally-derived frequency-magnitude curves for debris floods as a comparative tool with previously employed methods	Application of regional frequency-magnitude curves should be interpreted with care as described in Appendix G.
Percy Creek debris flows (*Appendix H*)	• Excavation of test pits to collect samples of organic material for radiocarbon dating and collection of tree core samples for dendrochronological analysis. • Reconstruct yield rates along the creek to estimate the total volume of erodible sediment available. • Add estimated sediment volumes from point sources along the creek. • Use of regionally-derived frequency-magnitude curves for debris flows	
Debris flood, debris flow hybrids	• Use appropriate combination of methods from above.	Results presented in a triple F-M curve showing regional analysis for floods, debris floods with bulking in terms of stage and debris flows as estimated based on radiocarbon dating and dendrochronological analysis, where applicable.
Dam outbreak floods	• Determination of peak discharge and outflow hydrographs from the failure of a landslide dam using a physically based mathematical model, BREACH (Fread 1991).	Used by BGC (2010) as part of debris flood magnitude estimation on Mosquito Creek

While based on the best available data, estimates of debris-flow or debris-flood magnitudes contain uncertainties. Older deposits can be eroded or reworked and are therefore often difficult to distinguish unambiguously from one another. Creek-side development also creates practical limits to subsurface investigation of past deposits. For Indian Arm creeks, significant portions (over 50%) of the fans are under water and inaccessible in practical terms. Moreover, it is difficult to estimate the amount of new debris that is introduced to the fan from upstream past the fan apex versus debris that is recruited from bank erosion or channel bed scour from within the fan reaches. Hydromorphic processes also involve hazard mechanisms that cannot be quantified by flow modelling, such as bank erosion (e.g., Shone Creek, see Appendix K).

4.3. Climate Change Considerations

It is now scientifically broadly accepted that humans have measurably altered Earth's climate over the past 50 to 60 years (IPCC 2014). The relevance of climate change to Indian Arm creeks debris-flood and debris-flow risk is that the predicted warming of the troposphere will very likely increase the intensity of the hydrological cycle in many regions worldwide. Due to more intensive energy exchanges in the vertical air column, as well as the projected intensification of air mass exchange between the low and high latitudes, it is expected that extreme precipitation events will increase in frequency, intensity and volume (IPCC 2012; IPCC 2014). If this were indeed to take place or has already commenced, it could result in several undesirable outcomes with respect to mountain creek hazards:

- The frequency of floods, debris-floods and debris-flows may increase on small and possibly, larger rivers, especially if the timing of extreme storms coincides with the snowmelt season. Over the longer term (century time scale), however, some increases in extreme rainfall may be offset by lesser snowpack thickness due to projected temperature increases.
- The frequency and intensity (volume and peak flow) of debris floods and debris flows may increase for those basins that are sediment supply unlimited. Jakob and Lambert (2009) have shown that debris-flow frequency may increase by up to 10% by the end of the century. This could lead to higher capital costs for future maintenance of mitigation structures. Specifically, more frequent hydro-geomorphic processes will require a higher frequency of cleaning out the sediments that accumulate upstream of such structures.
- If the design of mitigation measures is based on purely stationary hydroclimatic conditions, their design capacity may, in time, be overwhelmed by extremely rare events whose magnitude had not been predicted, or by events whose return period has been reduced over time due to observed trends in hydroclimatic extremes. Thus, it appears sensible to use maximum estimates for higher return period events for numerical modeling and in the risk assessment. This introduces a reasonable element of conservativism to the analysis.

4.4. Frequency-Magnitude Relations

Peak discharge and sediment volume estimates for flood and debris flood creeks are provided in Appendix O. Appendix H provides the results of frequency-magnitude estimation for Percy Creek. Appendix K describes methods and results of frequency-magnitude estimation for the Indian Arm creeks north of Percy Creek.

5.0 HAZARD SCENARIOS

5.1. Introduction

Hazard scenarios are hypothetical scenarios where flows occur outside the normal creek channel in areas where they can impact development or infrastructure. The scenarios quantify the extent and "intensity", or destructive potential, of a flood, debris flood or debris flow to estimate building damages and risk to life.

Hazard scenarios do not encompass every possible flow or avulsion scenario that could occur. Rather, the objective is to define representative events at a range of return periods that are suitable for risk estimation. These return periods are based on the results of frequency-magnitude analysis (Section 4.0).

The steps required to obtain quantitative hazard scenarios include scenario development, numerical modelling, and hazard intensity mapping. This section summarizes methods used to complete each step. Appendix M provides more detailed descriptions of hazard scenario modelling and mapping methods and results. Appendix K describes hazard scenarios assessed for Indian Arm creeks, which were based on a review of previous work.

5.2. Hazard Scenario Development

5.2.1. Urban Debris Flood Creeks

BGC used the following steps to develop flood and debris flood hazard scenarios for each urban creek:

- BGC selected representative flows with magnitudes that could cause avulsions at culverts. The flow parameters (e.g., peak discharge) were based on the results of frequency-magnitude estimation (Section 4.0).
- For each culvert or stormwater main, BGC estimated the return period where culverts and stormwater mains are anticipated to overflow, either due to capacity exceedance and/or blockage by sediment or organic material. Appendix L describes methods used to assess culvert or stormwater main blockage potential.
- For each creek, BGC estimated the series of culverts or storm mains expected to block during a representative hazard event. Flow magnitudes and expected blockage scenarios formed the primary inputs for flow modelling.

Table 5-1 summarizes the hazard scenarios developed for each creek in this study. Mosquito Creek scenarios are based on previous modelling (BGC, 2013). Coleman, Canyon, McCartney, Kai, Cove, and Martin Creeks were estimated to have limited to no potential for flow avulsion at culverts resulting in downstream impacts to infrastructure. As such, BGC did not develop hazard scenarios for these creeks. In addition, no hazard scenarios were modelled for Thain Creek due to the sediment storage available at Prospect Road.

Table 5-1. Urban Creeks hazard scenario summary.

Creek	Scenario	Run(s)	Blockage Scenario	Blocked Asset ID(s)	Return Period (years)
Mackay Creek	1	1	Mackay West Avulsion Scenario	STMCUL00248, STMCUL00249, STMCUL00361	30-100
Mackay Creek	2	2	Mackay West Avulsion Scenario	STMCUL00248, STMCUL00249, STMCUL00361	100-300
Mackay Creek	3	3	Mackay East Scenario	STMCUL00364, STMMN00192, STMCUL00622, STMMN00365	30-100
Mackay Creek	4	4	Mackay East Scenario	STMCUL00364, STMMN00192, STMCUL00622, STMMN00365	100-300
Mosquito Creek	5	1	n/a	n/a	100-300
Mosquito Creek	6	2	n/a	n/a	30-1000
Mosquito Creek	7	3	n/a	n/a	1000-3000
Mission Creek	8	1-4	Debris Flood (Prospect Rd, Beaver Rd, Newdale Crt, Monteray Ave)	STMCUL00266, STMCUL00267, STMCUL00269, STMCUL00270, STMCUL00271	100-300
Mission Creek	9	3	Newdale Crt blocked trash rack	STMCUL00269	10-30
Mission Creek 2	10	5	Prospect Rd	STMMN01726	100-300
Mission Creek 3	11	6	Prospect Rd	STMMN09114	100-300
Hastings, Dyer Creeks [12]	12	-	E Braemar Rd	STMCUL00393 and STMCUL00395	100-300
Kilmer Creek	13	1	Kilmer Diversion	STMMN04251 and STMMN08659	Nov 2014 event
Kilmer Creek	14	2	Kilmer Diversion	STMMN04251 and STMMN08659	100-300
Thames Creek	15	1	Mountain Hwy	STMCUL00052, BGCSTMCUL00074,	100-300
Thames Creek	16	2, 3	McNair Rd, Kilmer Rd	STMCUL00152, STMCUL00412, STMMN09158	30-100
Taylor Creek	17	1	Mt Seymour Pkwy	STMCUL00259	100-300

[12] Hastings and Dyer Creeks hazard extents are based on experience and judgement following field observation not model results.

Creek	Scenario	Run(s)	Blockage Scenario	Blocked Asset ID(s)	Return Period (years)
Gallant Creek	18	1	Badger Rd and Deep Cove Rd	STMCUL00217, STMCUL00580	10-30
Gallant Creek	19	2	Badger Rd and Deep Cove Rd [13]	STMCUL00217, STMCUL00580	100-300
Panorama Creek	20	1	Panorama Dr	STMCUL00447	100-300
Mathews Brook	21	1	Overflow and avulsion of channel	None	30-100
Mathews Brook	22	2	Overflow and avulsion of channel	None	100-300
Gavles Creek	23	1	Panorama Dr	STMCUL00451, STMCUL00452	30-100
Gavles Creek	24	2	Panorama Dr	STMCUL00451, STMCUL00452	100-300
Cleopatra Creek	25	1	Panorama Dr	STMCUL00643	30-100
Cleopatra Creek	26	2	Panorama Dr	STMCUL00643	100-300
Francis Creek [14]	27	-	Panorama Dr	BGCSTMCUL00009	100-300
Unnamed Creek [15]	28	-	Fire Lane 2	BGCSTMCUL00094, BGCSTMCUL00095	30-100
Ward Creek	29	1	Indian River Dr	STMCUL00662	100-300
Ostler Creek	30	1	Indian River Dr	STMCUL00226	100-300
Allan Creek	31	1	Indian River Dr	STMCUL00227 (BGCSTMCUL00080), STMCUL00228 (BGCSTMCUL00073) [16]	30-100
Allan Creek	32	2	Indian River Dr	STMCUL00227 (BGCSTMCUL00080), STMCUL00228 (BGCSTMCUL00073)	100-300
Allan Creek	33	3	Indian River Dr	BGCVEHBRG00001	100-300

[13] Model initiated upstream at Indian River Drive to assess the potential for avulsion to the north.
[14] Francis Creek hazard extents were assessed based on field observations instead of model results.
[15] Unnamed Creek hazard extents were assessed based on field observations instead of model results.
[16] Allan culverts STMCUL00227 and STMCUL00228 are plotted at the incorrect locations. The correct locations for these culverts are shown by BGCSTMCUL00080 and BGCSTMCUL00073, respectively, on DNVHIT.

5.2.2. Percy Creek

Representative debris flow hazard scenarios for Percy Creek were based on estimated debris-flow frequency-magnitude relationships and estimated representative avulsion scenarios, as described below:

- The following four debris-flow return period classes (1-4) were modelled based on the Percy Creek frequency-magnitude analysis summarized in Section 4.4 and detailed in Appendix H:
    1. 30-100 year.
    2. 100-300 year.
    3. 300-1000 year.
    4. 1000-3000 year.
- For each of the four return period classes listed above, the following three representative avulsion scenarios (A-C) were modelled to simulate the effects of potential channel blockages near the fan apex and/or bridge crossing, or overtopping of the banks during superelevation of flows around channel bends:
    a. Avulsion towards the south fan sector (i.e., adjacent to the active creek channel).
    b. Avulsion towards the north fan sector.
    c. Avulsion towards the mid-fan sector.

The resulting twelve separate hazard scenarios are summarized in Table 5-2.

Table 5-2. Percy Creek debris flow hazard scenario summary.

Scenario	Run(s)	Avulsion	Blockage Scenario	Return Period (years)
34	1	A	South Avulsion	30-100
34	1	B	North Avulsion	30-100
34	1	C	Mid-fan Avulsion	30-100
35	2	A	South Avulsion	100-300
35	2	B	North Avulsion	100-300
35	2	C	Mid-fan Avulsion	100-300
36	3	A	South Avulsion	300-1000
36	3	B	North Avulsion	300-1000
36	3	C	Mid-fan Avulsion	300-1000
37	4	A	South Avulsion	1000-3000
37	4	B	North Avulsion	1000-3000
37	4	C	Mid-fan Avulsion	1000-3000

5.2.3. Indian Arm Creeks North of Percy Creek

Hazard scenarios considered for the Indian Arm creeks were based on a review of previous assessments by KWL (2003b, 2003d, 2003g, 2003h, 2003i, 2003j, 2011) and BGC (2009). These are described in Appendix K. No new modelling or hazard intensity mapping was completed for these creeks.

5.3. Hazard Scenario Modelling

BGC used numerical flow routing models to estimate the extent, velocity and depth of flow for each hazard scenario on a given creek. The model grid cell outputs were imported into GIS, overlaid on base maps, and used to interpret hazard intensity maps.

The model results are displayed using a "flow intensity index" (I_{DF}) and flow depth. The flow intensity index is calculated as flow depth multiplied by the square of flow velocity. The index is directly proportional to flow impact pressure (Zanchetta et al. 2004; Kang and Kim 2016) and can be empirically related to building damage (Jakob et al. 2012). It is not appropriate for estimating damages associated with low velocity flooding (e.g., approximately < 1 m/s), where values of I_{DF} will approach zero irrespective of flood depth. Below an intensity threshold of I_{DF}<1, BGC estimated flood vulnerabilities for buildings based on assumed flood depths (See Appendix N).

5.3.1. Debris Floods

BGC used the commercially available two-dimensional hydraulic model, FLO-2D (2007) to estimate debris-flood velocity, depth and the extent of inundation for each debris-flood hazard scenario.

Appendix M provides a detailed summary of debris-flood modelling methods. In summary, peak flow hydrographs for each flow scenario were routed downstream using FLO-2D. FLO-2D is suitable for this type of application as it can model both channelized and unconfined flows, and it is on the U.S. Federal Emergency Management Agency's list of approved hydraulic models. The culverts or stormwater mains listed in Table 5-1 were blocked in the model for each scenario. Estimated capacities of culverts or stormwater mains not assumed to be blocked were also input into the model.

5.3.2. Debris Flows

Debris-flow modelling for Percy Creek was carried out using the three-dimensional numerical model *DAN3D* (McDougall and Hungr 2004). *DAN3D* was developed specifically for the analysis of rapid landslide motion across complex 3D terrain and is well-suited to the simulation of coarse debris-flows that deposit on relatively steep slopes, like Percy Creek fan. BGC has used *DAN3D* for the same purposes on other recent projects.

Appendix M provides a detailed summary of the debris-flow modelling method, input and results. The model outputs include files showing the modelled maximum intensity index (I_{DF}) within the modelled runout zone.

5.4. Hazard Intensity Mapping

Hazard intensity maps show the extent and “intensity”, or destructive potential, of a flood, debris flood or debris flow. They are based on the hazard modelling results, supplemented by judgement and generalized to account for model uncertainties.

BGC prepared hazard intensity maps for each hazard scenario that form the hazard basis for risk analyses. These maps are provided in Appendix M. Appendix N describes how the maps were used for damage estimation and risk analyses.

6.0 RISK ASSESSMENT

6.1. Introduction

Risk assessment involves estimation of the likelihood that a debris flood or debris flow scenario will occur, impact elements at risk, and cause particular types and severities of consequences. BGC assessed risk for urban creeks with identified hazard scenarios (Table 5-1). BGC estimated individual and group safety risk for persons within buildings, and economic risk associated with building damages due to debris impact. Other types of risks were not quantified, such as risk to persons outside buildings or intangible community impacts. While direct building damages represents only a portion of the economic consequences that could occur due to a debris hazard event, they can be systematically estimated and used as a proxy for the larger range of consequences that would actually occur.

The primary objective of the safety risk assessment is to identify cases where the estimated risk level exceeds risk tolerance thresholds defined by DNV for the purpose of informing DNV policymaking deliberations and creation. Following consideration of safety risk, the economic risk assessment can also assist in further prioritization of risk reduction options.

BGC used both quantitative risk assessment (QRA) and semi-quantitative risk assessment (semi-QRA) methods to estimate risk. QRA estimates an annual likelihood of some consequence (economic or safety risk), considering all hazard scenarios modelled for a given creek. In contrast, semi-QRA provides a relative, numerical risk rating and considers a single hazard scenario.

Section 6.2 summarizes QRA methods, which were used to estimate safety and economic risk for each creek system as a whole. The results of QRA support risk reduction prioritization for each creek (e.g., should creek “X” be higher priority, from a risk perspective, than creek “Y”). Appendix N provides more detailed description of quantitative risk assessment (QRA) methods and results for the urban creeks, and Appendix K describes detailed assessment methods and results for the Indian Arm creeks.

Section 6.3 summarizes semi-QRA methods, which were used to assign relative risk ratings to individual culverts or storm water mains. The economic risk rating for individual culverts addresses the question, “what is the probability that a particular culvert blocks and results in some level of economic consequences?”. The results of semi-QRA support risk reduction prioritization for individual culverts (e.g., should culvert “X” be higher priority, from a risk perspective, than culvert “Y”). This rating does not account for the series of culverts that might become blocked along a given creek during a hazard scenario. As such it should be regarded as a proxy for relative risk estimation supporting mitigation prioritization.

6.2. Quantitative Risk Assessment (QRA)

6.2.1. Safety QRA

Safety risk was estimated for each creek from two perspectives: risk to individuals and groups. Individual safety risk considers the risk to a particular individual exposed to hazard, and is independent of the number of persons exposed to risk. Group safety considers the collective risk to all individuals exposed to hazard, and is proportional to the number of persons exposed to risk. In both cases, safety risk estimates consider the spectrum of hazard scenarios assessed for a given creek.

Hazard scenarios were considered as having a credible (non-negligible) risk to life only where flows exceeded a minimum intensity threshold (I_{DF} > 1), or where buildings were identified as particularly vulnerable to impact. BGC is not aware of reported fatalities resulting from flows where I_{DF} < 1 except for deep (>2.5 m) flooding, which was not identified in this study. As such, while the possibility of fatalities can never be entirely ruled out, the risk is considered to be too low to be measurable. An example of a building identified as particularly vulnerable is 2150 Badger Road (Asset ID BLDG 14092) which lies on Gallant Creek downstream of Badger Road. If a blockage of the culvert on Gallant Creek at Badger Road (Asset ID STMCUL00217) occurred this building could be directly impacted by any subsequent flooding.

BGC compared the individual risk estimate results to geohazard tolerance criteria adopted by the DNV in 2009. The DNV criteria for individual geohazard risk tolerance are as follows:

- Maximum 1:10,000 ($1x10^{-4}$) risk of fatality per year for existing developments
- Maximum 1:100,000 ($1x10^{-5}$) risk of fatality per year for new developments.

For context, the DNV risk tolerance threshold of 10^{-4} (1/10,000) for existing development is comparable to the lowest background risk of death that Canadians face, on average, throughout their lives. This tolerance threshold is also similar to the average Canadian's annual risk of death due to motor vehicle accidents, 1/12,500, for the year 2008 (Statistics Canada 2013).

For risk to groups, estimated risks were compared to group risk tolerance criteria formally adopted in Hong Kong (GEO 1998) and informally applied in Australia (AGS 2007) and DNV. Group risk tolerance criteria reflect society's general intolerance of incidents that cause higher numbers of fatalities. Group risk tolerance thresholds based on criteria adopted in Hong Kong (GEO 1998) are shown on an F-N Curve in Figure 6-1. Three zones can be defined as follows:

- Unacceptable – where risks are generally considered unacceptable by society and require mitigation
- As Low as Reasonably Practicable (ALARP) – where risks are generally considered tolerable by society only if risk reduction is not feasible or if costs are grossly disproportionate to the improvement gained (this is referred to as the ALARP principle)
- Acceptable – where risks are broadly considered acceptable by society and do not require mitigation.

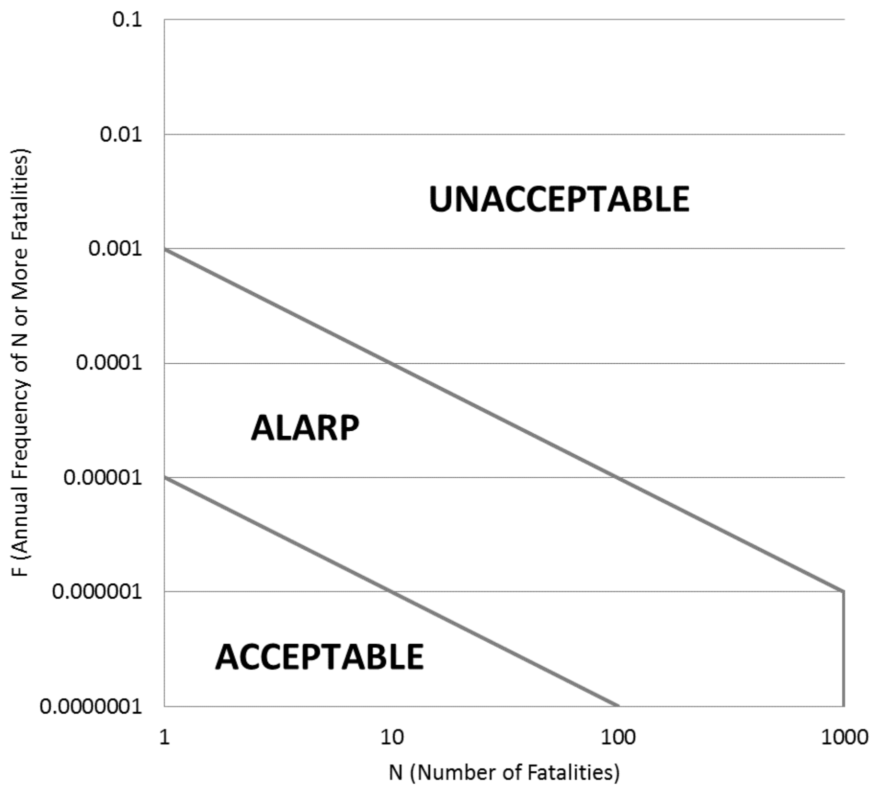

Figure 6-1. F-N curve showing group risk tolerance criteria as defined by GEO (1998).

6.2.2. Economic QRA

The economic QRA addresses the question, "what is the annualized building damage cost for each creek due to all of the debris hazard scenarios assessed?". Economic risk estimates are based on the hazard scenarios described in Section 5.0 and consider direct damages to buildings. The results are reported as a direct damage cost for each scenario, and as an annualized figure. The annualized cost is calculated by multiplying the hazard scenario probability by the estimated damage cost for a given scenario, and then summing the results for all scenarios.

Building damage cost estimates were based on vulnerability criteria relating damage levels to flows with a certain level of intensity or destructive power. Damage is measured as a proportion of the building replacement cost or as an absolute cost. Annualized damage cost is calculated by interpolating a damage curve from cost estimates for individual events at a given probability of occurrence. The curve shows estimated costs at a given event probability, and the area under the curve represents the approximate annualized direct damage cost.

Building damages associated with low intensity flows ($I_{DF} < 1$) were assumed to include inundation by water and sediment. BGC estimated building damages for such flows using flood stage-damage curves developed for Alberta Environment and Parks (AEP) following the damaging floods in southwestern Alberta in June, 2013 (IBI Group 2015). BGC selected these curves for analysis because they are the only curves that, to BGC's knowledge, have been developed specifically for residential development in Canada. Appendix N lists the stage-damage criteria used to estimate damages for lower intensity flows and describes assumptions and limitations.

Building damages associated with higher intensity flows ($I_{DF} > 1$) have the potential to cause structural building damage due to dynamic impact pressure, and were considered to have credible potential to cause loss of life. Appendix N describes the vulnerability criteria used to estimate building damages for higher intensity flows and describes assumptions and limitations.

6.3. Semi-Quantitative Risk Assessment (Semi-QRA)

BGC estimated baseline risk for existing conditions at each culvert, and residual risk assuming implementation of the risk control measures described in Appendix O. Culvert risk ratings were only assigned to culverts on creeks with identified hazard scenarios (see Section 5.2).

Table 6-1 displays the matrix used to determine relative economic risk ratings for individual culverts. The relative ratings range from 1 to 7, with 7 being highest risk. The hazard rating in the matrix corresponds to the culvert blockage rating (see Appendix L), and the consequence rating corresponds to estimated direct building damage costs downstream of the culvert.

Appendix N provides further details on methodology, assumptions and limitations of this rating. Combined with overall creek risk ratings, the culvert risk rating can support prioritization of each culvert for risk reduction implementation.

Table 6-1. Economic risk matrix.

Hazard Rating (Probability Hazard Scenario Occurs and Impacts Elements at Risk)			**Economic Risk Rating**				
Classification	**Hazard Scenario Probability**	**Culvert Overflow Rating (Years)**					
Very Low	0.001-0.0003	n/a	**1**	**1**	**2**	**3**	**4**
Low	0.003-0.001	>200	**1**	**2**	**3**	**4**	**5**
Moderate	0.01-0.003	200	**2**	**3**	**4**	**5**	**6**
High	0.03-0.01	50	**3**	**4**	**5**	**6**	**7**
Very High	0.1-0.03	20	**4**	**5**	**6**	**7**	**7**
Consequence Rating	**Indices**		Very Low	Low	Moderate	High	Very High
	Direct Damage Cost ($M)		<0.1	<0.5	0.5-1	1-10	>10

6.4. Results

Risks are assessed in this report against the District's risk tolerance criteria.

Risk tolerance criteria or standards were first utilized by the DNV in 2005, to inform decisions about the extent of landslide risk mitigation measures following the Berkley landslide.

In 2007, DNV Council held a workshop to review the natural hazards management program and approved a plan which included, "establish a process to adopt risk tolerance criteria". The Natural Hazards Task Force was assembled to provide a forum to gather input from an informed, broad-based community perspective regarding quantitative tolerable risk or risk acceptance criteria for landslides and other natural hazards. The task force presented their recommendations and the following policy was adopted by Council in December 2009:

That applicants for subdivisions, development approvals and building permits may be required to meet the following conditions:

1. Demonstration that natural hazards risks are reduced to As Low as Reasonably Practicable (ALARP).
2. In addition to ALARP, that the following risk tolerance criteria are satisfied.
   I. Maximum 1:10,000 risk of fatality per year for existing developments involving an increase to gross floor area on the property of less than or equal to 25%.
   II. Maximum 1:100,000 risk of fatality per year for new developments and for re-developments involving an increase to gross floor area on the property of greater than 25%.

The risk tolerance criteria policy is administered via the Development Permit Area application process and also applied during building permit and sub-division application processes. While the main intent of the risk tolerance criteria are to manage development risk, a secondary application of the criteria is to assist the DNV in prioritizing areas for risk mitigation where assessed risks exceed the risk tolerance criteria for existing development. This report uses the risk DNV's tolerance criteria in this secondary way.

6.4.1. Safety QRA

Table 6-2 lists individual risk results for buildings that exceed DNV's individual risk tolerance thresholds, including the urban creeks and the Indian Arm creeks described in Appendix K. Best-estimate values shown in red exceed DNV's threshold for existing development. Best-estimate values are also shown in orange where they would have exceeded DNV's threshold for proposed development. The lower, upper and best-estimate values reflect the range in criteria used to estimate building vulnerability to debris impact (Appendix N, Table N.2-5). The wide range between the lower and upper bounds reflects the large numerical difference between negligible and non-negligible vulnerability in the criteria.

Of the creeks shown in Table 6-2, Scott-Goldie, Shone/Underhill, Holmden and Friar Creeks were included in the Indian Arm Creeks scope of work. While hazards were re-evaluated with new advances in debris-flow science, risk analysis was outside the scope of work and the estimated risk is from BGC (2009). At Scott-Goldie Creek, BGC interpreted the level of hazard as lower than previously assessed. While not quantified, this implies a reduced risk. For Shone/Underhill creeks, re-evaluation of hazard characteristics suggests that safety risk due to direct building impact may be lower than previously estimated. However, additional hazard scenarios were identified that had not been considered in previous risk estimates, including bank erosion and long-term channel aggradation, that may increase risk for buildings adjacent to the active channel. For Holmden and Friar creeks, the level of safety risk is still estimated to exceed the DNV risk tolerance threshold for existing development.

Figure 6-2 displays estimated group risk for Percy Creek on an F-N curve, which is the only urban creek where the best-estimate of group risk falls in the unacceptable range. The lower, upper and best-estimates shown on the graph reflect the range used to estimate vulnerability.

Table 6-2. Number of parcels on the assessed urban creeks where estimated individual risk exceeds DNV risk tolerance criteria for existing or proposed development.

Creek	Asset ID	Address or Legal Description	Best-Estimate Risk of Fatality Per Year
Cleopatra Creek	BLDG11848	2755 PANORAMA DR	1.5E-05
Gallant Creek	BLDG14092	2150 BADGER RD	4.5E-05
Gavles Creek	BLDG12034	2683 PANORAMA DR	1.0E-04
	BLDG12061	2679 PANORAMA DR	1.0E-04
	BLDG25265	2672 PANORAMA DR	1.5E-05
Mathews Brook	BLDG12267	2603 PANORAMA DR	1.5E-05
Mission Creek	BLDG02481	310 NEWDALE CRT	6.0E-05
Panorama Creek	BLDG12516	2525 PANORAMA DR	9.0E-05
Percy Creek	BLDG22671	326 SASAMAT LANE	3.1E-04
	BLDG22672	327 SASAMAT LANE	6.4E-05
	BLDG22673	328 SASAMAT LANE	6.4E-05
	BLDG22674	330 SASAMAT LANE	6.4E-05
	BLDG22676	332 SASAMAT LANE	7.6E-05
	BLDG22677	333 SASAMAT LANE	8.1E-05
	BLDG22678	334 SASAMAT LANE	8.1E-05
	BLDG22682[1]	338 SASAMAT LANE	1.4E-05
	BLDG24327	335 SASAMAT LANE	7.5E-05
	BGCBLDG00001[2]	-	5.6E-03
Scott-Goldie Creek[3]	BLDG02865	301 SASAMAT LANE	1.7E-04
Shone / Underhill Creeks[3]	BLDG22738	LOT 3 BLOCK B DISTRICT LOT 812 PLAN 10914	1.0E-04
	BLDG22739	LOT 4 BLOCK B DISTRICT LOT 812 PLAN 10914	1.0E-04
	15 Cabins	-	1.0E-04 to 2.0E-04
Holmden Creek[3]	1 Cabin	DISTRICT LOT 871, PLAN 996 AND DISTRICT LOT 871, PLAN 2860	1.2E-03
Friar Creek[3]	BLDG22743	LOT 14 DISTRICT LOT 873 PLAN 3427	4.1E-03

Notes:

1. BLDG23261 is immediately adjacent to Vapour Creek and BLDG22682 is approximately 20 m south of Vapour Creek outlet. The estimated individual risk considers only Percy Creek and does not include some unquantified level of risk from Vapour Creek.
2. This is a cottage at the outlet of Percy Creek, which is not accessible by road. No DNV-assigned Asset ID exists for this building and it has not been 100% confirmed that it is occupied.
3. These creeks were assessed as part of the “Indian Arm creeks” scope of work. The risk estimates are from BGC (2009).

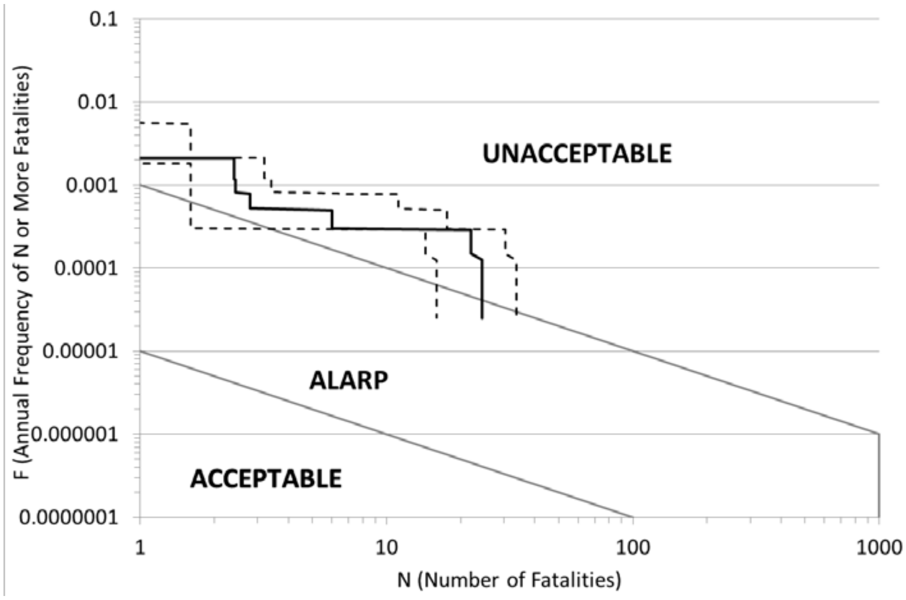

Figure 6-2. F-N Curve for Percy Creek.

Figure 6-3 displays estimated group risk for Mosquito Creek. Only the upper range estimate is shown because no fatalities[17] were estimated for the lower and best-estimate. It falls within the ALARP zone. Note that this result is based on hazard scenarios that include the effect of the debris flood net at the development interface to reduce risk on Mosquito Creek.

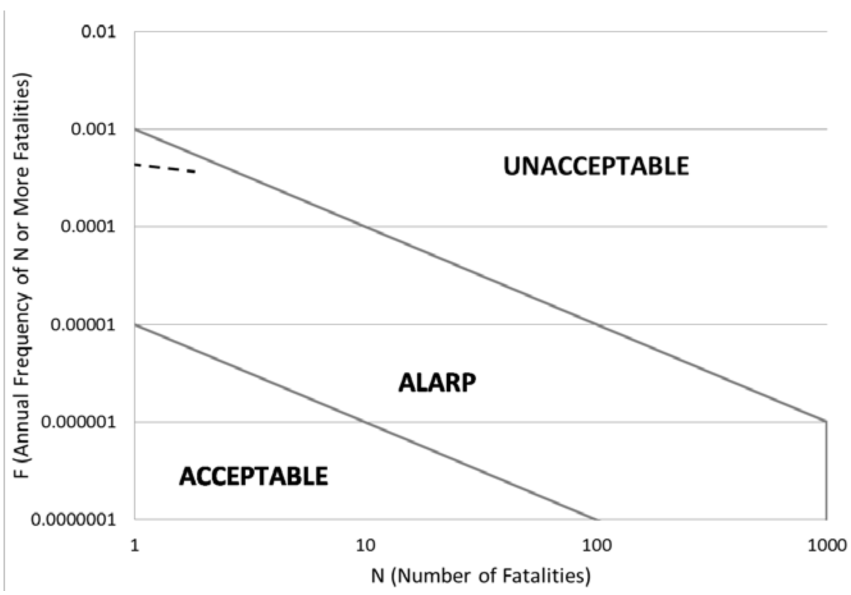

Figure 6-3. F-N Curve for Mosquito Creek, with the debris flow net installed.

[17] Technically, the lower and best-estimate of expected number of fatalities were "less than one", but not zero. However, fractional fatalities are not typically reported on F-N curves.

For the Indian Arm creeks, BGC reviewed whether the differences in hazard characteristics identified in this study could result in changes to the risk profile if new risk assessment was completed. Specifically, BGC considered whether additional, more detailed risk assessment could be justified, based on whether the additional work could affect recommendations for risk control measures or change the buildings identified as exceeding DNV's individual risk tolerance threshold for existing development (PDI > 1:10,000).

Table K.4.2 in Appendix K provides a detailed list of buildings within the hazard zones mapped by BGC (2009), with considerations for future studies and risk management options. Also shown are buildings where BGC (2009) estimated PDI > 1:10,000.

In summary, at Holmden and Friar Creeks, it is unlikely that additional risk assessment would change the conclusions of previous work with the exception of the home on the south side of Friar Creek (Lot 16 District Lot 873 Plan 3427). Additional risk analysis may indicate that the building on Lot 16, Plan 3427 exceeds DNV's tolerable threshold for existing development. The risk exposure to the single property on Scott-Goldie Creek could not be accurately determined without the benefit of additional hazard assessment and modelling.

At Shone, Underhill and Coldwell Creeks, previously assessed hazard scenarios did not explicitly consider the potential for bank erosion during events. Consideration of the vulnerability of buildings adjacent to the active channel to bank erosion could increase estimated risk levels. Moreover, at Camp Jubilee on the Shone Creek fan, new buildings have been added to the DNV building catalog since the 2009 assessment (BGC 2009) was conducted. The 2011 assessment (KWL 2011) focused on the proposed building layout instead of the existing layout. Additional work would be required to accurately assess the risk to occupants on Shone, Underhill and Coldwell Creeks.

6.4.2. Economic QRA - Creeks

Table 6-3 lists the estimated damage costs for each hazard scenario and the annualized direct damage costs for a given creek. Note that the annualized cost is calculated by multiplying the hazard scenario probability by the estimated damage cost for a given scenario, and then summing the results for all scenarios. As such, it is proportional to event frequency, and highly damaging but rare events may have a lower annualized cost than more frequent but less damaging events.

The annualized damage cost estimates displayed in Table 6-3 can be used to prioritize creeks for risk reduction implementation based on economic risk. As emphasized in Section 6.1, these costs are a proxy for the larger spectrum of potential damages that could occur due to a debris hazard event.

Table 6-3. Annualized direct damage costs for urban creeks.

Creek	Scenario	Hazard Scenarios Assessed (Annual Return Period Range)	Total # Homes In Hazard Zones	"Effective" Number of Buildings Impacted[1]	"Effective" Total Damage Cost[2]	Annualized Damage[2]
Mackay Creek	1	30-100	102	52	$3,200,000	$92,000
	2	100-300	103	52	$3,300,000	
	3	30-100	70	35	$2,300,000	
	4	100-300	71	36	$2,300,000	
Gallant Creek	18	10-30	16	8	$800,000	$90,000
	19	100-300	63	32	$2,100,000	
Kilmer Creek	15	30-100	77	39	$2,400,000	$83,000
	14	100-300	85	43	$2,700,000	
Thames Creek	15	100-300	150	75	$4,900,000	$79,000
	16	30-100	43	22	$1,300,000	
Mission Creek 3	11	100-300	48	24	$1,800,000	$18,000
Mosquito Creek	5	100-300	5	3	$140,000	$14,000
	6	300-1000	31	16	$2,200,000	
	7	1000-3000	45	34	$7,900,000	
Cleopatra Creek	25	30-100	9	5	$350,000	$12,000
	26	100-300	9	5	$350,000	
Mission Creek	8	100-300	24	12	$1,000,000	$10,000
Gavles Creek	23	30-100	13	7	$300,000	$10,000
	24	100-300	15	8	$300,000	
Mission Creek 2	10	100-300	25	13	$1,000,000	$10,000

Creek	Scenario	Hazard Scenarios Assessed (Annual Return Period Range)	Total # Homes In Hazard Zones	"Effective" Number of Buildings Impacted[1]	"Effective" Total Damage Cost[2]	Annualized Damage[2]
Percy Creek	34	30-100	13	1-13	$10,000 - $1,100,000	$4,800
	35	100-300	13	1-13	$10,000 - $1,100,000	
	36	300-1000	13	2-13	$200,000 - $1,600,000	
	37	1000-3000	13	2-13	$200,000 - $1,600,000	
Unnamed Creek	28	30-100	4	2	$140,000	$4,800
Mathews Brook	21	30-100	4	2	$120,000	$3,800
	22	100-300	4	2	$120,000	
Ward Creek	29	100-300	9	5	$380,000	$3,800
Panorama Creek	20	100-300	5	3	$280,000	$2,800
Taylor Creek	17	100-300	6	3	$190,000	$1,900
Allan Creek	31	30-100	2	1	$40,000	$1,600
	32	100-300	3	2	$80,000	
	33	100-300	3	2	$80,000	
Ostler Creek	30	100-300	3	2	$130,000	$1,300
Hastings and Dyer	12	100-300	1	1	$40,000	$400
Francis Creek	27	100-300	2	1	$10,000	$100

Notes:

1 Count of buildings in impact zones assuming 50% chance of building impact where intensity <=1.

2 Values are rounded to the nearest $100 or $1,000 if exceeding $1k or $10k respectively.

6.4.3. Economic Semi-QRA – Individual Assets

Appendix O lists risk ratings for each stormwater drainage asset, as part of risk control summaries for each creek. Risk ratings for each culvert are also provided on DNVHIT.

7.0 RISK CONTROL OPTIONS ASSESSMENT

7.1. Introduction

The debris hazard risk assessment identified and ranked locations where debris hazards are most likely to result in economic loss or loss of life within DNV. This work fulfilled phases 1 to 4 of the risk management framework adopted for this study (Section 1.1, Table 1-1). Phase 5 of the risk management framework, risk control, includes the following steps:

- Identify options to reduce economic and safety risks to levels that do not exceed DNV risk tolerance criteria.
- Select option(s) providing the greatest risk reduction for the lowest life-cycle cost.

This section completes the first of the above steps and provides guidance to the DNV for the second step. It describes options to reduce debris hazard risks at locations identified in the risk assessment, including:

- General comparison of risk control options
- Summary of factors that influence risk control designs
- Summary of the design basis for site-specific conceptual risk control designs that are presented in Appendix O.

BGC assumes that the process to select a preferred risk control option at a given site will also include risk evaluation (e.g., comparison to safety and economic risk tolerance criteria), communication of risk and consultation with stakeholders, and an on-going review of the risk management process. The information presented in this report section and in Appendix O is intended to be a starting point for selection and detailed design of individual risk control measures.

7.2. Risk Control Design Considerations

The following items influence feasibility, selection, and optimization of site-specific risk control designs. They have been initially considered for the conceptual risk control designs presented in this report (Appendix O), but should be re-assessed during each future stage of the design process, particularly during design options assessments, where there is the most flexibility to modify the design.

- Risk reduction targets – DNV's Development Servicing Bylaw No. 7388, Schedule D.1, Section C9.5, specifies that all culverts on creeks be designed to convey the 200-year peak instantaneous flow (Q_{i200}) or greater with the design headwater not exceeding the top of culvert. This bylaw implies that risks associated with larger flows are generally tolerable (design for an event larger than the Q_{i200} is needed where the estimated risk associated with the 200-year flow is intolerable). Safety risk targets have been set by DNV (see Section 6.2); however, BGC is not aware of economic risk targets that have been set by the DNV.

- Creek system design – Segments along a creek behave as an interconnected system. An action at one point along the creek will typically cause a response at other locations downstream of that point. The risk control design process must therefore consider the downstream response to each risk control design element. Designs that increase conveyance of flood flows or divert flows have the potential to increase downstream risks (called "risk transfer"). Designs that capture sediment or prevent sediment mobilization have the potential to reduce downstream risk. Risk transfer can be managed by ensuring that the downstream infrastructure is designed to accommodate the increased discharge and debris volume contributed by upstream designs. It is recommended that the DNV develop a functional chain of interacting risk control elements that addresses the debris hazard for each complete creek system where risk assessment demonstrates intolerable risk.
- Causes of culvert overtopping – Where risk is associated with culvert overtopping, risk control designs should seek to address the specific cause of water exiting the channel. BGC identified the following causes of culvert overtopping (Appendix L):
    1. Culvert is "undersized", meaning that flow exceeds culvert capacity.
    2. "Flat culvert inlet" causes debris accumulation at culvert inlet.
    3. "Flat culvert" causes debris accumulation within culvert.
    4. Boulders block the culvert inlet or reduce capacity within culvert.
    5. Large woody debris blocks the culvert inlet.
    6. Trash rack screen is undersized and blocks during storm events.
- Development density – Feasible risk control options can be influenced by the density of development surrounding and downstream of the hazard site. Typically, risk control options are constrained in residential areas due to limited space and landowner issues. Additionally, the possibility of risk transfer typically increases with increasing development density. For example, flow diversion would typically not be permitted in densely populated, urban areas due to risk transfer concerns, but could be a preferred option in sparsely populated areas where flow can be directed away from development.
- Hydrogeomorphic process type – Clear-water floods, debris floods, and debris flows have different likelihood of occurrence at different creeks across DNV. Table 1-2 (Section 1.3) lists the hydrogeomorphic process type assigned to each creek[18]. The primary distinction between the process types for risk control design is flow depth and flow velocity, with debris flows having relatively the highest values, followed by debris floods. Risk control measures designed for clear-water floods and debris floods may not adequately reduce risks associated with debris flows.
- Long-term vs short-term measures – Long-term measures tend to result in a more permanent and greater level of risk reduction, but also at a greater cost, than short-term measures. Limited resources and funding may prevent the DNV from immediately

[18] Process types were assigned based on the most destructive flow type. For example, creeks subject to debris floods are also subject to floods, but are classified as debris food creeks.

implementing long-term risk control measures at all sites. Short-term measures can often be implemented in the short-term (within budget and resource limits) to somewhat reduce risks in the interim period until a long-term measure is implemented. For example, culvert replacement to reduce flood risk associated with an undersized culvert is a long-term measure that may not be practical to implement in the short term. Short-term risk control measures that could be implemented at the site before culvert replacement is achieved include diverting flood water that overtops the stream back into the channel, and emergency response planning. BGC's site-specific risk control options are organized in terms of "Short-term" measures that can be implemented as soon as possible, and "Long-term" measures that meet DNV's risk reduction target, but are often more costly or disruptive to implement.

- Access and maintenance requirements – Different physical risk control measures require varying degrees of future maintenance. Maintenance requires site access. Planning for access and maintenance for the full design life of risk control measures should occur during the initial design stage. Where possible, structures should be designed and constructed to allow maintenance access for equipment that is readily available to DNV (e.g., backhoe for sediment removal). The degree and cost of long-term maintenance should be a factor considered in risk control options comparisons.
- Costs – Risk control options should be compared on the basis of life-cycle costs that include construction costs and life-cycle maintenance costs. Annual maintenance effort and costs can exceed the initial construction costs, in some cases.
- Social and Environmental impacts – Environmental and social impacts are other types of costs that can be difficult to quantify, but should be qualitatively considered during risk control options assessments. Short-term impacts during construction (e.g., road closure during culvert installation), as well as long-term impacts (e.g., reduction of sediment for aquatic habitat) should be considered. Social impacts include closure of roads and trails and other inconveniences or perceived impacts that cause residents to object to the proposed risk control measure.
- Design confidence – The level of confidence that a risk control measure will perform as intended can vary based on the risk control measure, design details, and the creek system. In some cases, hazard intensity and frequency can change with time, for example due to climate change or modifications to a watershed. The consequences of failure of a risk control measure should be considered during design. Design of a functional chain of risk control elements along the creek, resulting in some redundancy, is a common practice for managing uncertainty in risk control performance.

7.3. Risk Control Design Options

7.3.1. Design Option Comparison

Table 7-1 summarizes options for reducing debris hazard risks within the DNV. Many of the options are also effective for reducing risks associated with culverts or bridge openings that are too small to convey clear-water flood flows. Not all of the methods described will be feasible at each hazard site. A suggested design approach is to list all possible risk reduction options applicable to a specific site, and then systematically eliminate options until a preferred risk control design is selected. The optimized final design may be a combination of multiple options. Table 7-1 is intended to be a starting point for the initial listing of risk control options. Illustrations and examples of selected risk control options are provided in Figure 7-1.

Table 7-1 focuses primarily on physical measures to control existing debris hazard risks. The frequency and magnitude of debris hazards could change with time due to modifications to the watershed and/or climate change. Climate changes and modifications to the watershed should be monitored, and modifications that increase debris hazards should be avoided. Programs are recommended to educate landowners along creeks about debris hazards and the landowner's role in managing debris hazards, which includes:

- Do not discard yard waste (e.g., tree limbs, grass clippings) in creek corridors; yard waste can clog culvert and bridge openings, leading to overland flooding.
- Remove yard waste or garbage that is found in creek corridors, or alert DNV to request maintenance.
- Avoid landscape or home improvements that encroach upon or constrict the creek channel.
- Homeowners that live in a flood hazard zone should consider measures to protect the individual home from flooding by shallow overland flow. Measures may include swales, gutters, curbs or modifications to driveway and surface grading around the home. Consider that water often enters homes through doors or ground level windows.

Table 7-2 compares various aspects of the risk control design options in general terms.

Table 7-1. Debris hazard risk reduction options.

	Method	Description	DNV Application	Design Details	Advantages	Disadvantages
1	**Debris Control**					
1a	Channel Stabilization	Reduce the volume of sediment that is mobilized in critical stream reaches, which would typically include between the development interface and about 300 m upstream. Could be achieved with regularly spaced check-dams constructed from log-cribs, concrete, or large boulders.	May be applicable upstream of development at creeks that traverse long distances through development (e.g., Mackay, Mission, Kilmer, Thames). Could be used in place of, or in combination with, sediment basins or barriers.	Check-dam height is typically on the order of 1 to 2 m, and spacing on the order of 20 m to 50 m, and these vary based on sediment size and stream inclination. Seek opinion of stream restoration and bio engineering specialist. Log crib structures are likely to be considered more aesthetically pleasing than other options.	Likely to require less maintenance than sediment basins because sediment is not removed from behind the check dams. Can reduce debris hazards for the entire creek downstream of structure. Can typically be installed without significant changes to existing infrastructure.	Requires disturbance to multiple creek locations, and poor construction or maintenance practices could lead to large disturbed area along the creek.
1b	Debris Capture	A basin or barrier located upstream of development that is designed to capture coarse sediment and allow water flow to pass. A concrete and steel outlet structure that captures sediment and ensures water outlets to the channel is typically required. Sediment removal is required to maintain storage capacity.	Relatively large basins or barriers may be applicable upstream of development at creeks that traverse long distances through development (e.g., Mackay, Mission, Kilmer, Thames). Small basins or barriers could be used to protect individual culverts.	Size basin to store the design sediment volume and manage water discharge during storm event. The 200-year sediment volume reported by BGC could be used as a starting point. Select barrier height and erosion protection to prevent overtopping, erosion, or out flanking. Include features that allow easy access by common machinery for routine sediment removal and maintenance.	Can reduce debris hazards for the entire creek downstream of structure. Can typically be installed without significant changes to existing infrastructure. Length of creek disturbance may be less than channel stabilization option.	Permanent access to the basin or barrier site, regular sediment removal, and regular maintenance is required.
1c	Culvert inlet debris control	Prevent entry of sediment and debris into the culvert inlet with a debris rack, trash rack, or debris crib located a short distance upstream of the culvert inlet. Provide a small basin upstream of the debris control structure for sediment and debris storage, and routinely remove sediment and debris to maintain storage capacity.	Broadly applicable at most culverts.	Design details, including structure type, post spacing, post height, and barrier position, vary based on the sediment and debris size that is intended to be captured, and the specific site geometry. See details provided in the Risk Reduction Options section of the report.	Typically less expensive than most other options. Some variation of culvert inlet debris control is possible at all culverts, and can be a first line of defense against culvert blockage.	Design options are often limited by channel geometry constraints, and it is not always possible to reduce debris risks to desired levels with culvert inlet debris control alone. Poor designs can increase culvert blockage and avulsion likelihood.
1d	Trash rack modifications	Trash racks located at culvert inlets have blocked with fine debris, such as small twigs and leaves, leading to flow avulsions from the channel. Inlet blockage with fine debris that can be conveyed through the stormwater system can be reduced by modifying or removing trash racks. Potential modifications include increasing bar spacing, increasing trash rack surface area, or modifying the trash rack shape.	Broadly applicable at culverts and storm sewer inlets with existing trash racks.	Modify bar opening size to capture as little debris as possible while preventing blockage downstream. Screening area should be at least 3 times the culvert area, and ideally 10 times the culvert area. Bar opening size on security screens (intended to prevent human entry) should be approximately 15 cm.	Improves performance of existing infrastructure, and can be relatively less expensive than most other options.	Trash rack size may be limited by channel or culvert inlet geometry constraints, and overly small bar openings and/or screening area may result in debris blockage leading to flow avulsions from the channel. Trash rack removal may not be desirable due to safety considerations (e.g., preventing human entry).
2	**Conveyance**					
2a	Increase opening size	Replace existing culvert with a larger diameter culvert that is sized to convey design clear-water flow and sediment.	Broadly applicable where existing culverts are undersized to convey the design flow peak discharge. Could also be applied to reduce debris blockage potential where an existing culvert is likely to be blocked with sediment or debris.	Select culvert diameter that can convey at least the 200-year peak discharge with an allowance for sedimentation and climate change. For example, this may be the 200-year discharge plus 20%. A single culvert is less likely to block than twin culverts with same total capacity.	Replacing with a larger culvert can address avulsion risks caused by both debris blockage and undersized flow capacity. Performance of this option is more reliable and predictable than most other options.	Requires culvert replacement, with associated construction disturbances. Can transfer sediment and debris blockage and avulsion risks downstream.
2b	Increase culvert inclination	Replace existing culvert with a culvert that is inclined similar to the creek gradient or as steeply as is allowed by flow velocity restrictions. Increased culvert inclination will improve conveyance of sediment through the culvert.	Applicable where an existing culvert has high sediment blockage potential due to very shallow culvert or inlet area inclination and where the outlet is suspended above the creek level. Would often be combined with installation of a larger diameter culvert.	All new culverts should be installed with a gradient that is similar to the average creek gradient. The selected gradient should take into account flow velocity in the culvert to ensure self-cleaning and to minimize channel instability at the outlet. A minimum two-year flow velocity of 1 m/s is recommended so that fine material is self-cleaned from the culvert. Maximum flow velocities at the culvert should be consistent with channel stability requirements at the culvert outlet; flow velocities of 4.5 m/s or greater require appropriate energy dissipation at the outlet to prevent erosion.	Promotes fine sediment movement along the creek system, which can reduce maintenance and sediment removal effort, and has positive environmental implications (e.g., maintains natural stream function). Reduces potential for sediment and debris build up within the culvert.	Requires culvert replacement, with associated construction disturbances. Can transfer sediment and debris blockage and avulsion risks downstream.

Method		Description	DNV Application	Design Details	Advantages	Disadvantages
2c	Overflow culvert	Install an additional culvert adjacent to an existing culvert to increase the total flow capacity of the crossing.	An alternative to replacing an existing culvert that is too small to convey the 200-year peak flood discharge. Would be applicable at locations where it is less disruptive or expensive to install an additional culvert than to replace an existing culvert.	Overflow culverts are commonly installed at an elevation above the existing culvert so that water can be conveyed if the lower culvert is overwhelmed or blocked with debris. Overflow culvert should be sized considering the combined capacity of the existing and overflow culvert and potential for debris blockage.	Removal of the existing culvert is not required. Can be used to manage debris hazards by increasing the sediment storage potential at the culvert inlet area.	Requires favorable site geometry and infrastructure layout, which is not always possible to achieve. Can cause construction disturbance that is similar to culvert replacement. Can transfer sediment and debris blockage and avulsion risks downstream.
2d	Improve hydraulics	Increase flow capacity of an existing culvert by improving the culvert hydraulics. Options may include: installing or raising the headwall, modifying the channel gradient at the outlet, modifying the culvert inlet or outlet, or modifying the channel alignment.	Broadly applicable to maximize performance of existing culverts. May also be an alternative to replacing an existing culvert that is slightly too small to convey the 200-year peak flood discharge.	Headwalls should tie in to upstream creek banks to prevent avulsion around the headwall. New culverts should be installed in alignment with the upstream and downstream channel.	Can be used to increase culvert flow capacity without requiring major construction disturbance across the roadway.	Typically, will only marginally increase the culvert capacity compared to the existing condition.
2e	Channel upgrades	Reduce avulsion potential from the channel by increasing the cross-sectional area of the channel. This can be achieved by increasing the channel width or using training berms to increase the flow depth.	Applicable where flow has potential to overtop the existing channel in open areas away from culverts and bridges. This typically occurs where residential landscaping or retaining walls form an obstruction that reduces the natural channel width.	Ideally, the typical natural channel width and depth that exist upstream of the obstruction would be restored. Where this is not feasible, ensure that the flow capacity through the obstructed area is equal to the flow capacity in the typical natural channel, and allow for sedimentation in the channel.	May be the only feasible option for reducing flow avulsion risks where channel obstructions exist.	Typically requires action from private residences.
3	**Designated overflow**					
3a	Return flow to channel	Direct excess flow during flood events to a designated overflow area that is designed to resist overtopping with minimal damage. May reduce area of impact, and related economic risks, caused by culvert blockage. Typically requires water flow to overtop the roadway.	May be applicable at low traffic volume roadways with a reasonable detour, and where favorable site geometry for flow diversion back to the channel exists.	Overflow channel geometry would typically need to convey flows less than about 30 cm depth. Provide a rolling dip or swale in the roadway, raise the roadway curb, or provide cut-outs or drains beneath the curb that direct water flow to the desired location. Anticipate that drain inlets will be blocked with sediment and debris. Protect the flow path from erosion.	Can be used to manage risks associated with undersized culverts or debris hazards without requiring culvert replacement.	Road service would likely be disrupted and the road may be closed during and immediately following the event. May result in erosion damage or sedimentation on the roadway.
3b	Divert flow to other channel	Direct excess flow during flood events to another channel. Erosion protection along the diversion is required. The channel that receives the diverted flow should be designed to convey the combined discharge.	Risk transfer is a primary consideration. This option may be applicable where flow can be diverted to an undeveloped area. For example, it may be feasible to divert hazardous flows from Mackay Creek away from development towards Capilano reservoir.	Diversion channel geometry would typically need to convey flows less than about 30 cm depth. Provide a rolling dip or swale in the roadway, raise the roadway curb, or provide cutouts or drains beneath the curb that direct water flow to the desired location. Anticipate that drain inlets will be blocked with sediment and debris. Protect the flow path from erosion.	Can reduce debris hazards for the entire creek downstream of the diversion point. Can be used to manage risks associated with undersized culverts or debris hazards without requiring culvert replacement.	Debris hazard is transferred to the channel that receives the diverted flows, with the potential for risk transfer.
4	**Watershed Area Reduction**					
4a	Upper watershed diversion	Reduce the watershed area, and resulting peak discharge in the creek system, by diverting water that is captured in upper areas of the watershed.	Risk transfer is a primary consideration. This option may be applicable where flow can be transferred to an uninhabited watershed.	Watershed boundaries in the upper watersheds are often overlapping and have been modified by forest road drainage ditches and culverts. Careful mapping of features in the upper watershed is needed to ensure that flow diversion occurs as intended. It may be possible to divert only excess flow that exceeds the capacity of an existing culvert or channel.	Can reduce debris and flood hazards for the entire creek downstream of the diversion point.	Debris hazard is transferred to the watershed that receives the diverted flows, with the potential for risk transfer.
5	**Operations and Maintenance**					
5a	Routine maintenance	Maintain maximum possible flow capacity of existing culverts, and maximum storage potential of debris control structures by periodic maintenance, which may include removal of sediment and debris and structural repairs. Includes developing a maintenance schedule, and may include repair of erosion protection elements.	Broadly applicable. Should be applied in all areas that are subject to debris hazards.	Excavate and remove sediment and debris from the culvert inlet area and channel, in addition to the current practice of removing debris from debris control structures by hand. Use a backhoe for sediment excavation, where feasible. Consider adding stage indicators to debris control structures to indicate the depth of captured sediment (e.g., 0%, 50%, and 75% full). Record the volume and date of sediment removal to support on-going review of the risk management process.	Maximizes performance and lifespan of existing infrastructure. Can reduce total life cycle costs.	Requires perpetual funding and staffing resources.

Method		Description	DNV Application	Design Details	Advantages	Disadvantages
5b	Emergency response	Prepare emergency response plans that are implemented during forecasted high flow events, for example by staging equipment that can remove sediment and debris from critical culverts, trash racks, sediment basins and flow paths during an event.	Broadly applicable. Should be applied in all areas that are subject to debris hazards. May be a short-term measure until construction of other physical works is complete.	Consider that large storm events will likely impact multiple creeks simultaneously, and emergency response actions should prioritize the highest risk creek segments. Planning, resources allocation, and access construction is required prior to the emergency event.	Maximizes performance of existing infrastructure.	Requires a large volume of resources and equipment dispersed across DNV during the high flow event.
6	**Remove Elements at Risk from Hazard Zones**					
6a	Temporary evacuation	Evacuate homes and public buildings during periods of heavy rainfall and elevated debris hazard. Monitor rainfall and issue a warning and evacuation notice when rainfall thresholds are exceeded.	Only safety risk is addressed. Economic risk is not reduced. Would typically only be applicable at debris flow creeks, where safety risk is intolerable.	Rainfall monitoring and thresholds have been established for the DNV. Monitoring system operation and response plans need to be established.	No physical measures are needed. Low capital cost to implement.	Residents may not follow evacuation notices because of frequent false alarms. Does not address economic risks.

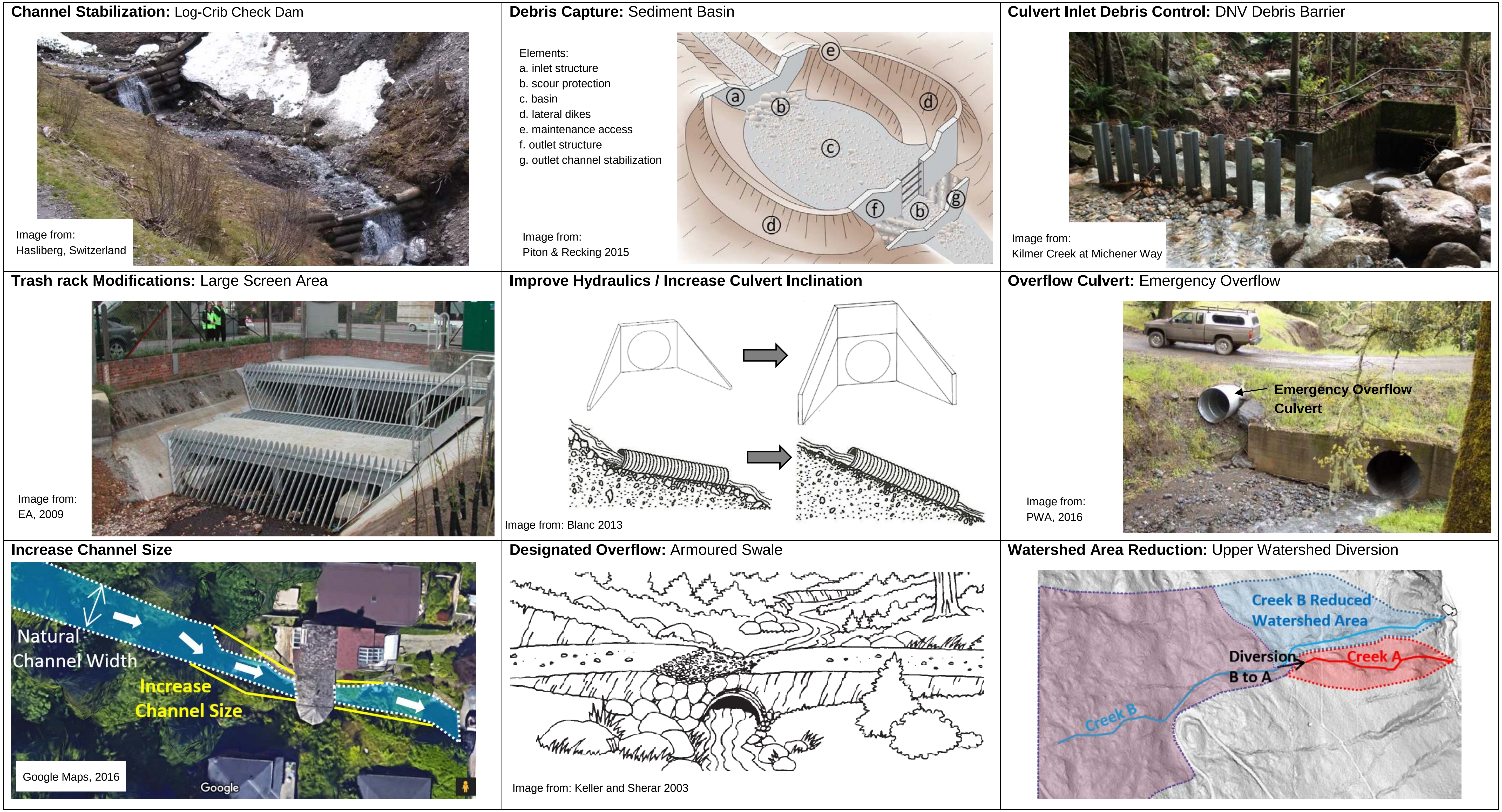

Figure 7-1. Examples of debris hazard risk reduction options.

Table 7-2. Comparison of debris hazard risk reduction options.

No.	Method	Economic Risk Reduction	Potential for Risk Transfer	Design Life	Maintenance Requirements	Costs	Social Impact	Environmental Impact	Design Confidence
1a	Channel stabilization	+	++	L	+	-	+	=	-
1b	Debris capture	++	++	L	- -	- -	-	-	+
1c	Culvert inlet debris control	+	++	S/L	-	+	+	=	-
1d	Trash rack modifications	++	+	S/L	-	=	+	=	-
2a	Increase opening size	++	-	L	+	-	=	=	+
2b	Increase culvert inclination	+	-	L	+	-	=	=	=
2c	Overflow culvert	+	-	S/L	=	-	=	=	+
2d	Improve hydraulics	+	-	S/L	=	=	+	=	=
2e	Increase channel size	+	=	L	=	=	-	=	+
3a	Return flow to channel	+	-	S/L	=	+	-	=	=
3b	Divert flow to other channel	+	- -	S/L	=	+	-	-	+
4a	Upper watershed diversion	++	- -	L	=	+	=	- -	=
5a	Routine maintenance	+	+	S/L	NA	+	+	-	=
5b	Emergency response planning	+	++	S/L	NA	++	++	+	-
6a	Temporary evacuation	- -	++	S/L	NA	=	- -	++	-

Notes. "- -" very undesirable, "-" undesirable, "=" neutral, "+" desirable, "++" very desirable, "NA" not applicable, "L" long-term, "S/L" short-term or long-term.

7.3.2. Channel Stabilization versus Debris Capture

Channel stabilization (Option 1a) and debris capture (Option 1b) can be used to reduce debris hazards along the entire creek system. Typically, these measures would be installed upstream of the development interface, and are intended to significantly reduce the quantity of sediment that arrives at development. The following paragraphs compare these two options.

Channel stabilization is achieved by the creation of non-erodible features within the channel and low velocity reaches that promote sediment deposition and discourage sediment entrainment. Although there appears to be little experience with this method in Canada, channel stabilization is popular in Europe, and often involves installation of log-crib or stone check dams evenly spaced along the channel. Check dams are intended to fill with sediment, creating a low-gradient reach that reduces flow velocity and reduces the grain size and quantity of mobilized sediment. BGC estimates that channel stabilization along a couple hundred meter long reach immediately upstream of development would be sufficient to capture coarse sediment from the upper watershed and limit the volume of mobilized sediment in developed areas.

Debris capture refers to construction of a sediment basin or barrier at a single location upstream of development. The basin or barrier is intended to capture sediment during high flow events, and must be cleaned periodically to maintain sediment storage capacity. Debris capture requires a favorable site geometry (typically a wide ‘flat spot’ along the channel), and perpetual maintenance.

Although DNV has more experience with debris capture techniques, DNV may wish to install and monitor performance of channel stabilization measures in a select area as a short-term pilot project. If performance and costs can be verified, channel stabilization measures may represent a favourable, low maintenance alternative that is suitable for the steep, confined channels common in DNV.

7.3.3. Culvert Inlet Debris Control

Debris control at culvert inlets (Option 1c) is widely applied across DNV, and should continue to be a standard practice applied at nearly all culverts. When properly designed, debris control at culvert inlets reduces the potential for culvert blockage due to boulders and large woody debris (including Christmas trees and yard waste). Currently, DNV has a design standard for culvert inlet debris control that is illustrated in DNV Development Servicing Bylaw 7388 Schedule D, dated October 2005 (Figure 7-2):

- Debris Barriers for Large Watercourses (Drawing SSD-D.11)
- Debris Barriers for Small Watercourses (Drawing SSD-D.12).

Figure 7-2. Standard debris barriers specified by DNV Development Servicing Bylaw 7388. Small watercourse barrier (left). Large watercourse barrier (right).

Culvert inlet protection that is currently in place in DNV varies widely in terms of both design and effectiveness[19], and frequently is not consistent with the DNV design standard. General observations of the design and effectiveness of existing culvert inlet protection, includes:

- Barriers are typically effective at stopping downstream movement of boulders.
- Barriers are typically not effective at stopping downstream movement of sand, gravel, and cobble-sized sediment.
- Woody debris (e.g., small sticks, leaves) is typically retained on the upstream side of barriers, and BGC understands that this debris is removed frequently by DNV maintenance staff.
- Barrier height is often less than the expected flow depth during storm events, and therefore woody debris typically floats over the top of the barrier during storm events.
- The position of barriers along the channel is typically not optimized to maximize potential sediment storage volume.
- The horizontal rebar member on the downstream side of the small watercourse barrier (Figure 7-2) is sometimes detached from several of the vertical rebar members due to failure of the weld.

DNV should enforce a design standard for future culvert inlet protection installations. Bylaw 7388 drawings (SSD-D.11, 12) appear to be suitable design standards, although the following modifications could improve inlet protection effectiveness (Figure 7-3):

[19] Photographs and descriptions of all debris control structures identified by BGC during the study are displayed on DNVHIT.

- Open width between posts – Specify the open width between posts, as opposed to the post spacing, and allow for this spacing to be determined during detailed design of the barrier. The open width between posts determines the size of material that is retained by the barrier. Open width between posts less than one-half the culvert diameter is sometimes needed. Open width should allow passage of fine material that can travel through the culvert without causing blockage. Open width should be 1.0 to 1.5 times the diameter of the particle that is desired to be captured. An open width of 30 cm to 50 cm would be appropriate at typical DNV creek locations to capture coarse sediment mobilized during debris floods, while allowing finer sediment to pass.
- Post Height – Where adequate channel confinement and freeboard exist, the post height should generally extend approximately 0.5 m above the height of the culvert so that large woody debris floating on the surface of the flow is captured during storm events. The condition of a fully blocked barrier should be evaluated to ensure that water overtopping the barrier is contained within the channel.
- Location – As possible, the barrier should be located at a position that maximizes sediment storage behind the barrier, and allows access for a backhoe to remove sediment trapped by the barrier. Additionally, the barrier posts should be setback from the culvert inlet a distance that permits sediment removal with heavy machinery in the area between the barrier and culvert. The minimum 4 m setback specified in the DNV drawing for large watercourses (SSD-D.11) appears suitable. Typically, the optimum position for the barrier is near the 4 m setback distance, rather than farther away.
- Access – Provide for long-term maintenance access whenever possible. Access should be provided for equipment that is readily available by the DNV to excavate and remove sediment, for example a backhoe.
- Maintenance – Currently, large woody debris is periodically removed by hand, but sediment is not routinely excavated and removed from behind barriers. Periodic removal of sediment that accumulates at the culvert inlet and behind barriers is recommended. This should be done with a backhoe where possible, and using hand shovels where heavy equipment access is not permitted. Sediment should be removed from site, and not stored in the channel. Consider marking barriers with sediment depth indicators and adopt a standard of removing sediment when a certain stage (e.g., 25% full) is reached.
- Debris Cribs – A debris crib is an alternative culvert inlet debris control design that can be used where it is necessary to prevent ingress of sand and gravel sized sediment, for example at culverts with a very shallow gradient, prone to sediment accumulation within the culvert or at the inlet area. A crib is constructed in log-cabin fashion around the culvert inlet (see examples in Bradley et al. 2005), leaving horizontal slots (approximately 15 cm opening width) that permit the passage of water. Debris cribs can be effective at capturing fine sediment, but result in increased maintenance to remove accumulated sediment, and can increase the depth of ponded water at the culvert inlet area.
- Small Watercourse Barriers – Small watercourse barriers (SSD-D.12) appear to have little effectiveness at preventing blockage of the culvert inlet during storm events, typically because the post height is shorter than the flow depth, and the rebar members are

damaged by boulders. Large watercourse barriers (SSD-D.11) appear to be a more effective option. Where small watercourse barriers are used, the horizontal rebar member may be less likely to detach if it was welded to the upstream side of the vertical members.

7.3.4. Trash Racks

A trash rack is a screen placed across a culvert inlet, typically attached to the concrete headwall and wing walls. Trash racks are typically comprised of closely-spaced parallel bars, installed to reduce the possibility of debris causing blockage within the culvert or stormwater sewer, and to prevent human entry into the culvert.

Trash rack installations can increase the potential for flooding caused by debris accumulation that blocks the screen. DNV maintenance personnel report that excessive debris accumulation and flooding is a common issue that affects culverts with trash racks in DNV. This hazard can be reduced with proper trash rack design. The following points outline best practices for trash rack design (EA 2009; Blanc 2013):

- Do not install a trash rack when it is not needed or where other practical alternatives are possible. Environment Agency of Bristol, UK, have published a Trash and Security Screen Guide (EA 2009; available online) that provides a framework for assessing the need for a trash rack and detailed design recommendations.
- Before deciding a trash rack is necessary, assess the probability of blockage within the culvert or stormwater system based on the debris size and culvert characteristics, including length, diameter, inclination, and layout. Also consider other methods for preventing human entry, including fencing, community engagement, and warning signs.
- Trap as little debris as possible, while preventing material that could cause a blockage within the culvert from progressing downstream (EA 2009). Leaves, small sticks, and fine sediment are unlikely to cause a blockage within the vast majority of DNV culverts, and should generally be allowed to pass trash racks.
- Bar spacing should be as wide as possible, while still meeting objectives. EA (2009) recommends that clear space between bars should be 14 cm at security screens (designed to prevent human entry), and that a clear space of 30 cm may be appropriate where smaller debris can safely pass through the culvert or stormwater system.
- In general, the screen area should be approximately 10 times the culvert area (Bradley et al. 2005). Screen areas ranging from 3 times to 30 times the culvert area may be appropriate based on site-specific characteristics (EA 2009). A larger screen area reduces the potential for screen blockage and flooding. Increased screen area can be achieved by modifying the screen alignment and layout, for example by orienting the screen diagonally across the channel or by including horizontal screen segments (Figure 7-3).
- Plan for regular cleaning of the trash rack, disposal of debris, and emergency response in the event that the screen becomes blocked during a flood event. Maintenance personnel should have safe access to the screen during flood events.

Figure 7-3. Trash rack layout options. Horizontal screen segments (left). Oriented diagonally across channel (right). Images from EA (2009).

7.4. Site-Specific Risk Control Assessment

BGC completed a conceptual-level, site-specific risk control assessment at identified sites that do not pass the 200-year flood. The assessments are organized by creek in Appendix O, and include BGC's interpretation of:

- Risk control design options.
- An indication of costs and potential risk reduction associated with each option.
- Comparison of advantages and disadvantages of risk control design options.
- Recommendations regardinng a preferred option.

The site specific information provided summarizes our current understanding of possible risk control options, and supports further assessment and risk control design by DNV. Site specific information provided is not intended to be the sole basis for final risk control design. In all cases, further work will be needed to assess and complete final design of risk control.

7.5. Risk Control Implementation

The results of this report support prioritization of risk control options primarily from a safety and economic perspective. BGC understands that safety risk is the over-riding priority and that following consideration of safety risk, economic risk and other considerations beyond the scope of this assessment (e.g., environmental, social) can form an additional basis to make risk control decisions.

Section 6.2.1 discussed DNV safety risk tolerance criteria that can form the basis to prioritize risk control measures from a safety risk perspective. The following additional factors should also be considered when selecting and implementing risk control measures:

- Develop economic risk tolerance criteria – Economic risk tolerance criteria could be used as benchmarks for design, in addition to the 200-year return period design standard. They would describe thresholds that separate sites that require risk reduction from those that do not. Designs would be optimized so that the residual risk following implementation of the design is less than the risk threshold. Implementation of economic risk tolerance criteria may require that higher return period design events (e.g., 500-year event) be selected at sites with relatively high potential flood consequences. Risk tolerance criteria could be in terms of annualized flood damage for the creek system (Section 6.4.2) or semi-quantitative risk level at an individual asset (Section 6.3) or a combination of both. BGC understands that DNV is integrating natural capital into the District's asset management program. Defining economic risk associated with natural hazards as a liability could assist with decisions to manage geohazard risk alongside other economic risks faced by the district.

- Select sites for risk control – Site selection should consider that the creeks behave as a system. Changes made at one site typically affect risk at other locations along the creek. The risk assessment ranks risk at each creek based on annualized flood damage (Table 6- 3). This prioritization could be directly used for site selection by allocating resources and funding systematically from the highest to lowest risk creeks, as resources become available, until all creeks are addressed. Appendix O provides guidance for selecting measures and sites along a particular creek. Generally, priority should be given to individual sites along the creek that have the highest semi-quantitative risk rating. The 'priority' column in the creek-specific risk control tables (Appendix O) identify sites that are interpreted to have the highest potential risk reduction benefit for the creek system (called Priority 1 in Appendix O). Priority 2 and 3 sites have relatively less impact on total risk at the creek. When selecting risk control elements, consider that some proposed risk control measures in Appendix O may be redundant (i.e., they assume other proposed measures along the creek have not been implemented). Some amount of redundancy in the final design is desirable because it can overcome uncertainties in design performance and design input parameter; however, implementation of all the proposed design measures along a given creek may not be needed to reduce risk to tolerable levels.

- Option selection – Appendix O provides multiple risk control options for many sites, and additional risk control designs that are not presented in Appendix O may be appropriate. An option selection assessment would consider the life cycle costs of the proposed measures versus the benefits, in terms of risk reduction, and compare other design considerations (e.g., Section 7.2). Short-term options that can be implemented quickly may exist that can be replaced by long-term options, as appropriate. Potential short-term risk control measures are provided in Appendix O for some relatively higher risk creeks. The intent is to highlight design elements that BGC estimates could be implemented relatively easily (e.g., modify trashrack) compared to other design options (e.g., culvert replacement). Proposed short-term measures should be considered along with the long-term measures during the option selection assessment.

- Detailed design and construction – The design information provided in this report is preliminary and conceptual. All risk control elements require further site-specific detailed design work. Future design stages may include: a preliminary phase focused on defining project objectives, selecting risk control options along the creek, and comparing design alternatives; and a detailed phase that refines the selected alternative and prepares plans for implementation and construction. The risk control designs completed during the detailed design phase will supersede those described in this report.
- Update asset management database – Information provided in this report is intended to be a starting point for a living database that integrates with DNV's existing asset management system. Semi-quantitative risk ratings for individual assets, and creek system risk should be updated as risk reduction measures are implemented.
- Operation and maintenance – Periodic inspection of risk control measures is recommended, typically at an annual interval and following high flow events. Maintenance requirements vary with the design element, but often include periodic sediment and debris removal, and repair of erosion protection elements.

8.0 CONCLUSIONS

This study provided debris geohazard risk assessment and conceptual debris risk control options for creeks within the District of North Vancouver (DNV). The study includes 35 "steep" creeks (creeks with channel gradients >5%) prone to flood, debris-flood or debris-flow processes that could cause economic damages or pose risk to life for persons within buildings.

BGC's work was subdivided into "urban creeks" within DNV's stormwater drainage network and accessible by road, and the Indian Arm creeks north of and including Sunshine Creek. The majority of the study focused on the urban creeks to support risk reduction planning for these areas.

For stormwater drainage assets with credible potential for debris blockage, BGC estimated the return period where culverts and stormwater mains are anticipated to overflow either due to capacity exceedance or blockage by sediment or organic material. BGC developed and modelled representative hazard scenarios and estimated safety and economic risk associated with these scenarios. BGC developed an interactive geohazard asset management application, DNVHIT, to display the results and supporting data for this work.

Risk estimates for each creek will assist the DNV in making policy decisions regarding overall watershed risk reduction planning and prioritization. Relative risk ratings at the asset level will assist DNV decision making with respect to risk reduction prioritization for each stormwater management asset, and will assist in quantifying the level of risk reduction achieved by implementing the risk control measures.

BGC safety risk assessment identified 5 single family residential buildings with estimated risk levels exceeding DNV's individual risk tolerance standard for existing development (>$1x10^{-4}$ risk of fatality per year). These residences are located on Gavles Creek (2 buildings), Mission Creek (1 building), and Percy Creek (2 buildings). An additional 13 buildings exceeded DNV's individual risk tolerance standard for proposed development (>$1x10^{-4}$ risk of fatality per year). Percy Creek was the only creek identified where estimated group risk fell within the unacceptable range when compared to international risk tolerance standards.

BGC's risk control assessment included identification of economic and safety risk reduction options to assist DNV policymaking deliberations and creation and guidance regarding the options that provide the greatest risk reduction for the lowest life-cycle cost. The major results and conclusions of this work are provided in Appendix O as individual creek summaries.

Appendix K describes assessment of Indian Arm creeks including Scott-Goldie Creek and creeks north of Percy Creek and not accessible by road. Of these creeks, Scott-Goldie, Shone, Underhill and Coldwell Creeks are considered the highest priority for future study.

At Shone, Underhill and Coldwell Creeks, previously assessed hazard scenarios did not explicitly consider the potential for bank erosion during events. Consideration of the vulnerability of buildings adjacent to the active channel to bank erosion could increase estimated risk levels. Moreover, at Camp Jubilee on the Shone Creek fan, new buildings have been added to the DNV building catalog since BGC's 2009 assessment. The 2011 assessment (KWL 2011) focused on a proposed Camp Jubilee building layout and did not consider the existing residential buildings on the north side of the creek.

9.0 RECOMMENDATIONS

This section provides recommendations to address data gaps, and to update and integrate the study results into a broader geohazard risk management plan. Recommendations for general risk control design considerations and design options are provided in Section 7.0. Site-specific risk control design considerations and recommended risk control options are provided Appendix O.

9.1. Stormwater Management Asset Data

After DNV updates their asset inventory with the data collected for this study and for the stormwater management model, BGC recommends a data gap analysis to address any remaining gaps in the basic parameters listed for each asset (e.g., shape, material, dimensions, length and gradient).

9.2. Monitoring Requirements for Upper Watersheds

The risk assessment is a snapshot in time based on conditions that currently exist. Landscape changes such as road building, culvert blockage, culvert replacement, road deactivation, and forest fires have the potential to change the hydrologic regime of the assessed creek watersheds, which in turn may affect the frequency or magnitude of debris geohazard events. Areas of particular interest include:

- Deep Cove creeks (Gallant Creek to Allan Creek) intersecting Indian River Drive and Mount Seymour Road, and creeks intersecting powerline trail (Mission and Thain), where road ditches and culverts affect channel flows and watershed boundaries. The maintenance and operation of these culverts should be monitored as part of ongoing geohazard management for these creeks.
- Upper Mackay Creek East, where drainage alterations exist within or near the Grouse Mountain Ski area operation (e.g., road drainage management). Debris movements have occurred frequently in this watershed, including in 1995, 1998, 2014, and 2015.

9.3. Indian Arm Creeks – Further Studies

BGC recommends that DNV consider further hazard and risk assessment for Vapour, Scott-Goldie, Shone, Underhill and Coldwell Creeks. At Scott-Goldie Creek, it is possible that additional assessment will show a reduction in estimated hazard levels and associated risk.

Additional assessment at Shone, Underhill and Coldwell Creeks should further consider the potential for bank erosion during events. Assessment at Shone Creek should consider the current configuration of Camp Jubilee and include all buildings on the fan for estimation of group risk.

9.4. Flow Estimate Updates and Climate Change

A stormwater drainage model is presently being completed by NHC that will provide more detailed, calibrated estimation of creek flows than was available during the preparation of this report (see Appendix E). BGC understands that NHC's work will include both current conditions and estimated flows that consider climate change.

BGC also notes that Thames Creek shares its upper watershed drainage with Kilmer Creek. The majority of flow from the upper watershed drains into Kilmer Creek, with a smaller portion avulsing into Thames Creek at a poorly confined creek section. Figure 0.9.3 in Appendix O shows the location of this "flow split". The proportion of flow directed to either creek has not yet been quantified, and flow estimates within this report assume that the entire watershed upstream from the flow split drains into Kilmer Creek. As such, Thames Creek flows may be higher than estimated and the overflow ratings assigned to culverts on Thames Creek may be non-conservative.

BGC recommends updating this study to reflect the results of NHC's assessment, once available. Flow estimates on Thames and Kilmer Creek should also be updated to reflect the portion of upper watershed flows directed into either creek.

9.5. Geospatial Tools for Geohazard Management Planning

This assessment will require regular updates to remain useful for decision making over the long term. BGC recommends that DNV work with BGC to develop protocols and standards for updates and data transfers between DNVHIT and DNV Geomatics staff.

BGC also recommends that DNV use a geospatial web application to integrate site inspections, creek and slope geohazard risk management planning, monitoring programs and the management of geohazard-related information (e.g., assessment reports and mapping). Because the framework of risk assessment is fundamentally similar across hazard types, it could include the complete range of creek and slope hazards within the district.

The application would not replace functions of the existing DNV geospatial tools (e.g., Geotools or Geoweb). Rather, the objective would be to accomplish the following goals that would, in turn, support broader goals within DNV related to emergency management, development permitting and general asset management:

- Reduce administrative requirements and information management effort by providing a common platform to manage geohazard-related mapping, inspections, and reports.
- Simplify communication of geohazards-related information to DNV staff and 3rd party consultants.
- Support consistent responses to development permit applications in DPA creek and slope hazard areas.
- Provide a tool to help plan annual geohazard management work programs.

- Provide a common platform to enter site inspections and assessment data and for geohazard monitoring (e.g., slope monitoring).
- Support evaluation of risk control options, including capital and maintenance costs, and the level of risk reduction achieved.
- Support updates to geohazard and risk assessments following implementation of risk reduction measures (including those measures described in Appendix O of this report) or changes to development or infrastructure.
- Reduce costs and level of effort required by DNV geomatics staff to incorporate new data by enforcing data management standards.
- Ensure that geohazard information within the DNV can grow over time as an integrated, organized knowledge base.

The DNVHIT application developed for this study already provides many of the needed requirements and it is designed to accommodate the upgrades required to fulfill this role. Examples of functions that would help accomplish the above goals but are currently missing on DNVHIT include:

- User interface to enter data on web forms and with mobile devices
- Functions to handle versioning and metadata (e.g., for recording repeat inspections)
- Functions to query, view and export data in tabular and geospatial formats
- Functions to manage reports and documents
- Functions to manage instrumentation and slope monitoring data
- Ability to handle multiple levels of access (e.g., read-only versus read-write).

Development of such functions could be accomplished on an as-needed basis.

10.0 CLOSURE

We trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC ENGINEERING INC.
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APPENDIX A
PREVIOUS EVENTS

A.1. PREVIOUS EVENTS

Residents of DNV have been affected by flood, debris flood and debris flow events for as long as the land has been occupied. Most of these events do not have written accounts, and those that were recorded do not typically contain detailed descriptions of damages, culverts avulsions, or flow characteristics (e.g., depth or velocity).

This appendix describes select steep creek events recorded by local residents, DNV records, and news reports and field observations used to calibrate the hazard analysis and debris flood modelling completed for this assessment. The appendix is not exhaustive and a detailed accounting of recorded hazard events is outside the scope of work. A focus of this appendix is the November 2014 event, as it has detailed records of damages and blockage of stormwater infrastructure.

A.1.1. 1896 to Present Mosquito Creek

Mosquito Creek has a long history of hydrogeomorphic events. Major floods have been reported in 1896, 1906, 1918, 1919, 1949, 1950, 1954, 1956, 1958, and 1961. In the early 1960s, Mosquito Creek was culverted for a distance above Queens Road. However, flood events in 1968, 1971 and 1975 still caused flooding and erosion issues (BGC, 2013 and KWL, 2003d).

A.1.2. 1950s Flood Event

In the 1950s, local resident reports indicate that Gallant Creek flooded Gallant Road. Personal notes from the Deputy City Engineer in the 1980s and 1990s indicated extensive flooding on the North Shore in the mid-1950s leading to property damage and construction of dikes from 2nd to 22nd Street on Mosquito Creek (KWL, 2014c).

A.1.3. October 1981 Flood Event

On October 31, 1981, a storm event affected the DNV and surrounding area with far reaching consequences. On the Squamish Highway, the M Creek Bridge was destroyed by a debris flow leading to the deaths of nine people (Squamish History Archives, 2011). In the DNV, Seymour River flooding caused severe erosion to river banks and damage to residences downstream of the Dollarton Highway Bridge (Figure A.1-1) (Province of BC Ministry of Environment, 1995). Water intakes in Lynn Creek formerly used as part of DNV's water supply system were also damaged and their use terminated.

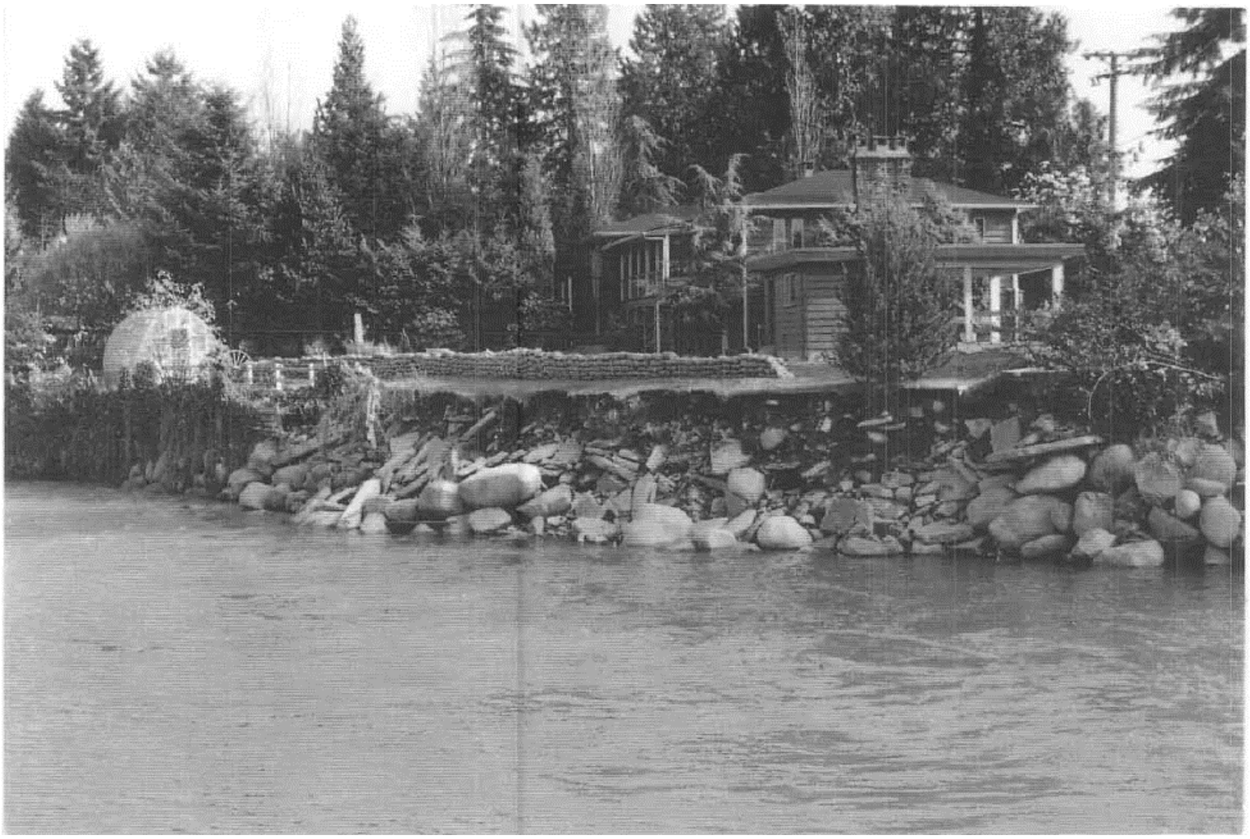


Figure A.1-1. Seymour River looking upstream at the east bank upstream of the Dollarton Highway Bridge following the October 31, 1981 storm showing severe bank erosion (Province of BC Ministry of Environment, 1995).

A debris flood on Mosquito Creek during this storm event caused flooding of Fire Hall #3 at 550 Montroyal Boulevard, erosion above Montroyal Boulevard, blockage of the Evergreen Basin, and one death (KWL, 2003d). Subsequent channel reconstruction between Montroyal Boulevard and Evergreen Basin was undertaken (KWL 2014c). The peak instantaneous flow measured at the Mackay Creek gauge was 15.5 m^3/s for that storm event (KWL 2014c).

The Kilmer Diversion stormwater main intakes at Thames Creek (STMMN09158) and Coleman Creek (STMMN09149) were reportedly blocked with woody debris during the flood, which resulted in Kilmer Road being washed out at Thames Creek (KWL, 1982).

A.1.4. November 1989 Events

In November, 1989, a severe debris flood occurred on Shone Creek. The Camp Jubilee dyke was eroded on the north bank leaving only small sections of the dyke on the south bank. Following the event, Hay & Company Consultants Inc. (HayCo) recommended short-term mitigation works involving armouring of the banks using onsite boulders; however, the materials used were too rounded to resist bank erosion and the works have since failed. Severe flooding was previously experienced in 1983 and the first know channel works were implemented following this flood (KWL, 2003f).

Flooding also occurred on Ostler Creek on November 10, 1989 (KWL 2003e).

A.1.5. Early 1990s Events

The following events were reported in the early 1990s:

- November 25, 1991 debris flow on Mackay Creek.
- Debris flow event on Holmden Creek in the early 1990s (KWL 2003b).
- Ostler Creek flooded in October/November 1990, November 14, 1994 and November 23, 1995. These floods caused property damage (KWL 2003e).
- A debris flood originated in the north gully of Allan Creek in 1992 or 1993 and flowed onto Firelane 7 (KWL 2003e).
- In November 1990 a flood was reported on Shone Creek which caused erosion to the north bank. This event may have avulsed towards Gardner Brook fan to the southeast (KWL 2003f).
- A debris flow occurred on Underhill Creek in the fall of 1990 or early 1991 and transported < 1000 m^3 of sediment to Shone Creek.

A.1.6. 1995 Events

On November 23, 1995, a debris flow event occurred on Mackay Creek leading to bank erosion in the lower channel reaches. Following the event, DNV completed in-stream works including cleaning and placement of large boulders along the creek banks. Lock-block wing walls were also constructed on either side of the creek on the upstream side of the Ranger Avenue culverts (VanDine, 1996a).

During the event, the west channel bank at 5171 Ranger Avenue experienced erosion and a small masonry retaining wall was removed by the flow. In the summer of 1996, additional channel improvements were requested to stabilize the bank from future erosion. KWL developed preliminary drawings of the works entitled “Mackay Creek Channel Improvements Downstream of Ranger Avenue” in the fall of 1996. An additional request was made to update the configuration of lock-block walls in order to reduce the risk to the property in the event of water overtopping the Ranger Ave culverts (DNV, 1996a, DNV, 1996b, Fisheries and Oceans, 1996, VanDine, 1996a, VanDine, 1996b). BGC inspected the site and indicated that mitigation works had been completed as part of the upstream construction work on the debris basin (BGC, 2006).

The November 23 rainstorm also resulted in flooding on the Deep Cove creeks which resulted in culvert blockage, overland flooding and destabilized channels (KWL 2003a).

A creek washout was reported on Shone Creek on November 25, 1995 (KWL 2003f).

A.1.7. 1998 Events

A debris flow occurred on Holmden Creek in 1998 that damaged the house on the fan and two docks. The house was relocated after the event (KWL 2003b).

On November 14, 1998 an intense rainfall event caused small magnitude (several hundred cubic meters of material) debris events on Mackay and Upper Mackay Creek. The majority of debris was deposited in an old gravel pit rather than entering Grousewoods development (KWL 2003c).

The November 1998 rainfall event caused flooding on Shone Creek resulting in land loss due to erosion of the south bank (KWL 2003f).

A.1.8. Early 2000s Flood Event Gallant Creek

Local accounts by Deep Cove residents describe a flood event in the early 2000s that resulted in flooding along Indian River Drive where Gallant passes through two box culverts under the road. Additional flooding was reported where Gallant Creek crosses Deep Cove Road. The exact date of the flood event is unknown.

Overbank flooding on Mackay Creek was reported in November 2006 (KWL, 2014c).

A.1.9. November 2014 flood event

Much of the impetus for the present study was a storm event on November 3, 2014, which resulted in damages to homes, properties, or DNV infrastructure along Kilmer, Thames, Upper Mackay, Gallant, Coleman, Mission and Thain creeks. The damages were particularly notable as they were unexpected, occurring on creeks previously assessed as low hazard from a safety perspective. This event is analysed in greater detail than other recorded events as it provides calibration for debris flood modelling and risk analyses completed for this assessment.

An overview report on debris flow and debris flood hazards within the District of North Vancouver (DNV) was completed by EBA Engineering Consultants Ltd. (EBA) and Kerr Wood Leidal Associates Ltd. (KWL) in 1999. In that report, the debris flood hazard on Kilmer Creek was rated as low. Subsequent detailed hazard studies by KWL were only conducted on those creeks with a high or very high hazard rating, with the exception of a few moderate hazard creeks (e.g., the Deep Cove Creeks). Thus, there are a number of creeks within the DNV that are prone to debris floods, but have never been studied in detail, including Kilmer Creek.

Known damages associated with the November storm include:

- High flows in Kilmer Creek initiated a debris flood causing culvert blockages at the Kilmer Creek Diversion and at Fromme Road, resulting in overland flooding and property damage.
- Two culverts on Thames Creek became blocked, resulting in overland flooding.
- The east branch of Upper Mackay Creek had a small debris flow that caused a large amount of debris to accumulate above a pedestrian bridge on the east branch of Upper Mackay Creek, ultimately leading to the failure of this bridge.
- Gallant Creek experienced high flows with sediment and debris depositing at culverts and causing some overland flooding along Deep Cove and Gallant Roads.

These events and damages are described in greater detail below.

A.1.9.1. November 2014 Storm

The DNV maintains a total of four rain gauges within the District: the DNV weather station, Fire Hall 4 rain gauge, the Hastings rain gauge, and the Mackay Debris Basin rain gauge. A number of rain gauges are also operated by Metro Vancouver on the North Shore.

Table A.1-1 summarizes rainfall totals at the four DNV stations for various durations. The highest rainfall totals were observed at the Hastings rain gauge, where a total of 164 mm was recorded in a 24-hour period. This is expected given its higher elevation compared to the other gauges. An increase in precipitation amounts in mountainous terrain is common as moist air masses are forced upwards by the mountains leading to an increase in condensation and rain.

Table A.1-1. November 3, 2014 maximum rainfall totals at DNV gauges for various durations.

Duration	Municipal Hall (mm)	Fire Hall 4 (mm)	Mackay Basin (mm)	Hastings (mm)
1-hour	13	14	16	21
2-hour	23	26	29	40
6-hour	57	55	91	68
12-hour	82	84	97	132
24-hour	99	102	120	164

The storm began on the morning of November 3 and peaked toward the late evening, as illustrated by the Hastings rain gauge hyetograph below. Maximum hourly intensities reached 21 mm/hr.

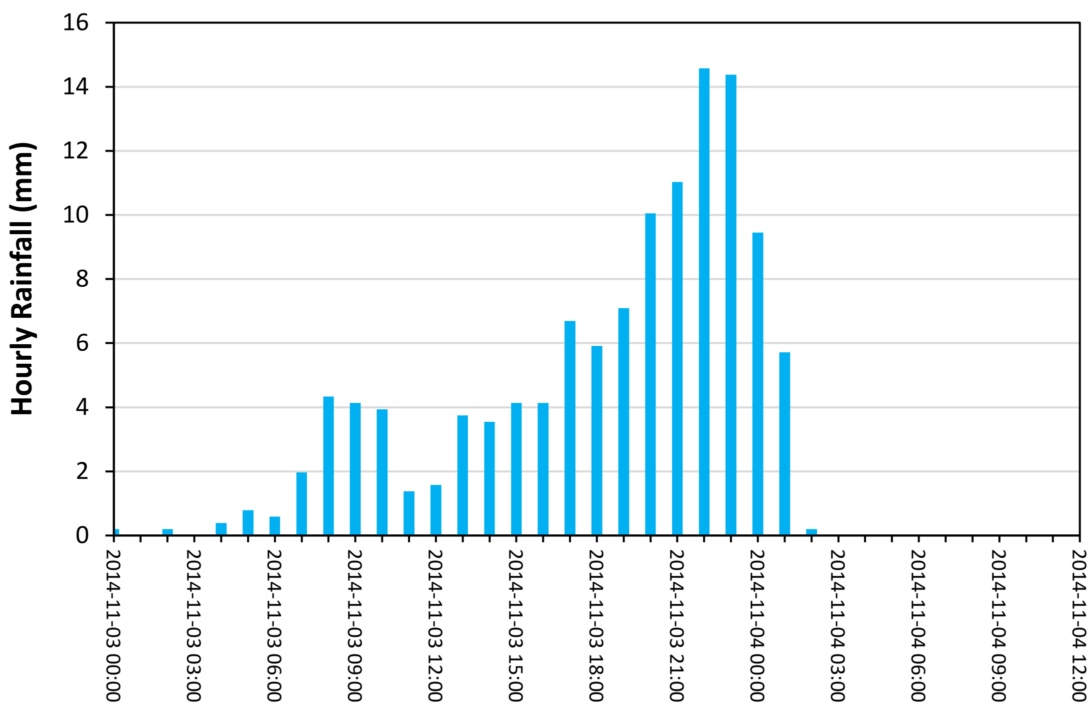


Figure A.1-2. November 3, 2014 hourly storm hyetograph for the DNV Hastings rain gauge.

A hydrometric station is located on Kilmer Creek above the diversion intake. This station is maintained by Northwest Hydraulic Consultants for the DNV (NHC, 2015). However, this station was severely damaged during the November 3, 2014 storm (see below) and no streamflow data for the event was recorded. As a result, streamflow data measured before and after the storm event at the Water Survey of Canada (WSC) *Mackay Creek at Montroyal Boulevard* (ID: 08GA061; 492860 E, 5467048 N; 167 masl) hydrometric station are shown in Figure A.1-3.

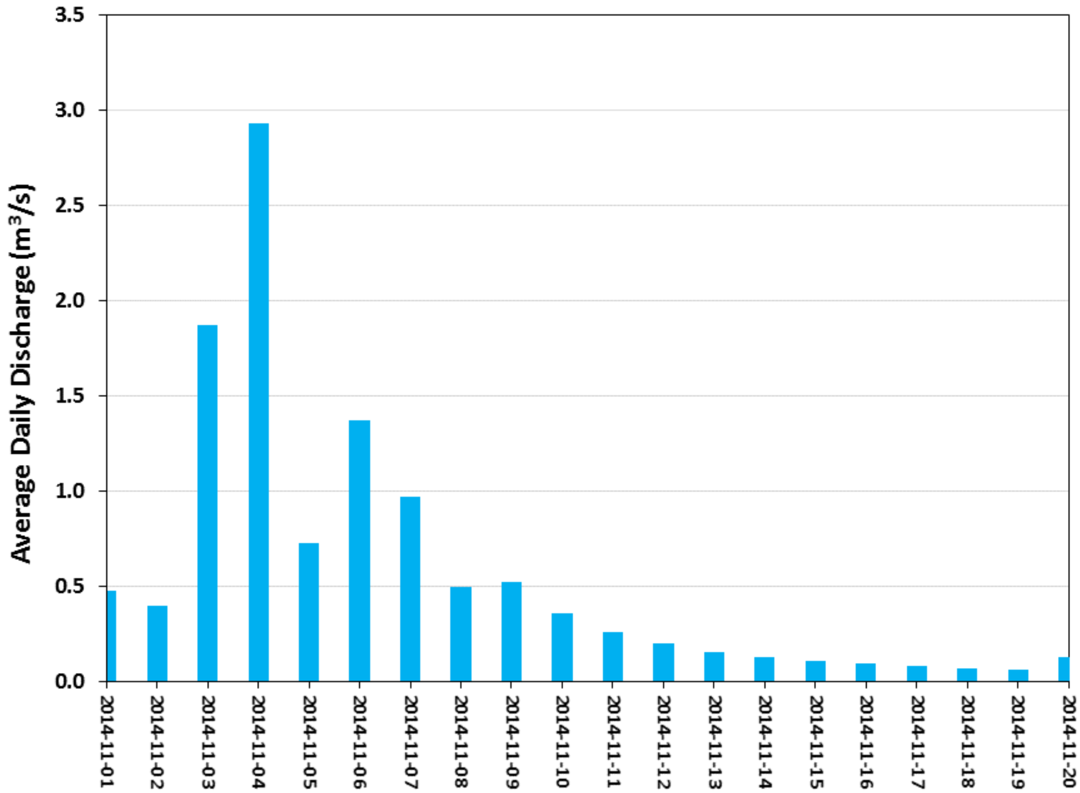


Figure A.1-3. November 2014 average daily discharge for the WSC Mackay Creek at Montroyal Boulevard (08GA061) hydrometric station. Data are preliminary and subject to revision.

A.1.9.2. Flood Damages

BGC compiled information on damages during the November 2014 event from post-event inspections by DNV staff (DNV 2015), Emergency Management BC (EMBC) (2015), post-event inspection reports (KWL 2015c), and personal communication with DNV staff. In summary, recorded damages included sediment deposition and flood impact to roads, properties and buildings; erosion; and debris blockage of stormwater drainage infrastructure.

Table A.1-2 summarizes the costs of emergency response and recovery for DNV. These costs exclude damages to private buildings, which were not recorded, or the costs of long-term debris hazard risk management. As such they represent a minimum.

Table A.1-2. Summary of DNV Emergency Response and Recovery Costs, November 2014 event, as provided by DNV (2015a).

Account	Amount ($ CAD)
Regular salaries	69,242.81
Overtime salaries	71,200.57
Fringe benefits	11,847.77
Temporary salaries	10,805.30
Equipment charges	15,251.75
Hired equipment	528,916.60
Inventory issues	1,801.34
Miscellaneous operating materials	13,243.22
Contract for service	387,289.28
Consulting costs	206,230.32
Meeting costs	356.11
Total	**1,316,185.07**

Drawings A-1 and A-2 show buildings and properties known to have been damaged during the November 2014 flood. Table A.1-3 summarizes the number of parcels (properties) with recorded property and/or building damage, their assessed value, and the volumes of sediment removed. Table A.1-4 lists compensation provided to homeowners by Emergency Management BC (EMBC).

The following sections discuss the damages observed at a number of creeks throughout the DNV.

Table A.1-3. Known flood damages associated with the November 3, 2014 storm event.

Creek	Parcels with Property Damage	Parcels with Building Damage	Property Value of Damaged Parcels[1]	Improvement Value of Damaged Buildings[1]	Parcels Requiring Sediment Removal	Volume of Removed Sediment (m^3)
Coleman Creek	1	1	$806,000	$67,900		
Gallant Creek	12	11	$68,994,000	$21,147,800	-	-
Hastings Creek	5	5	$3,731,000	$450,800	3	56
Keith Creek[2]	2	2	$1,487,000	$148,300		
Kilmer Creek	53	48	$54,879,000	$30,491,900	18	130
Lynn Creek[2]	1	1	$1,471,000	$291,000	-	-
Mackay Creek	6	3	$38,032,000	$15,876,000	-	-
McCartney Creek	2	2	$1,471,000	$218,200	-	-
Mission Creek	9	9	$10,313,000	$1,392,300	5	22
Mosquito Creek	1	1	$1,152,000	$730,000	-	-
Panorama Creek	1	0	$873,000	-	-	-

Creek	Parcels with Property Damage	Parcels with Building Damage	Property Value of Damaged Parcels[1]	Improvement Value of Damaged Buildings[1]	Parcels Requiring Sediment Removal	Volume of Removed Sediment (m^3)
St. Martins Creek[2]	1	1	$820,000	$173,000		
Thain Creek	1	1	$964,000	$296,000	-	-
Thames Creek	23	19	$21,805,000	$3,143,800	-	-
Wagg[2]	1	1	$1,050,000	$104,000		
None[3]	7	6	$7,111,000	$1,431,900	-	-
Total	**126**	**111**	**$214,971,000**	**$75,962,900**	**26**	**208**

Notes:

1 Values shown are 2015 assessed values. They are provided to indicate the value of development impacted and do not reflect the total cost of damages, which were not recorded.

2 Creek is not classified as a "steep creek" and is outside the scope of this study (see Section 1.3 of the Main Report).

3 Locations where impacts did not appear to be located on a creek.

Table A.1-4. Emergency Management BC compensation.

Creek	Number of Properties Receiving EMBC Compensation	Total Amount of EMBC Compensation
Hastings Creek	2	$16,107
Keith Creek[1]	1	$17,248
Kilmer Creek	11	$138,461
McCartney Creek	1	$15,613
Mission Creek	1	$715
St. Martins Creek[1]	1	$17, 636
Thames Creek	3	$35,375
None[2]	2	$7,756
Total	**22**	**$248,911**

Notes:

1 Creek is not classified as a "steep creek" and is outside the scope of this study (see Section 1.3 of the Main Report).

2 Locations where impacts did not appear to be located on a creek.

A.1.9.3. Kilmer Creek

Most of the damage at Kilmer Creek was sustained as a result of the culvert blockage at the Kilmer Creek Diversion. An overview of the Kilmer Creek watershed is first provided, followed by a description of the debris flood event, and damages incurred.

A.1.9.3.1 Watershed

The headwaters of Kilmer Creek are located on the south slopes of Mount Fromme. Two hydrological anomalies exist in the Kilmer Creek watershed that need to be taken into account for hazard assessments. The first is the influence of the Old Grouse Mountain Highway on drainage patterns. The Old Grouse Mountain Highway is a gravel road constructed in 1927 to provide access to the Grouse Mountain Ski Area. The road is accessed off the end of Mountain Highway, and initially traverses across the south slopes of Mount Fromme. After seven switchbacks, the access road crosses into the Mosquito Creek watershed on the west slopes of Mount Fromme.

Natural drainage paths have been altered as a result of this road construction. Most notably for Kilmer Creek is the area upslope of the seventh and final switchback. The watershed area shown in DNVHIT should drain naturally toward Mission Creek, Hastings Creek and Dyer Creek. However, because there are no culverts for about a 700 m section of road upslope of the seventh switchback, surface runoff from this area is intercepted by the road ditch, discharging into Kilmer Creek to the immediate east of the seventh switchback.

The second anomaly derives from a location about 600 m downstream of the seventh switchback and about 100 m upstream of the Old Grouse Mountain Highway. Here, the creek splits into two channels, Thames draining to the east and Kilmer draining to the west (Photograph A.1-1). The flow split is likely a legacy of past logging, as a number of old skid roads dissect the slopes in this area. The flow split allows creek flows to continue to flow south within the Kilmer Creek channel or divert to the southeast and enter the Thames Creek drainage. During the November 2014 storm, it appears that most of the flow remained in the Kilmer Creek channel, but it is possible that some flow was diverted into Thames Creek, exacerbating flood conditions in that watershed. Furthermore, future aggradation in the location of the flow split could lead to a larger portion entering the Thames Creek watershed.

Photograph A.1-1. Looking downstream at flow split between Kilmer Creek (right) and Thames Creek (left). BGC photograph of July 7, 2015.

Below the Old Grouse Mountain Highway, Kilmer Creek flows to the south for about 1.4 km at an average gradient of 22% before reaching the residential development boundary at Dempsey Road. Here the creek flows through an approximate 1.4 m x 2.4 m concrete box culvert (STMCUL00507). A debris barrier, consisting of four 0.8 m high steel posts, is located above the culvert inlet and is intended to trap large woody debris and prevent blockage of the culvert (Photograph A.1-2). A tributary discharges into Kilmer Creek downstream of Dempsey Road from the east. The creek flows through a similarly-sized box culvert 75 m downstream at Michener Way (STMCUL00045) (Photograph A.1-3). Immediately below this second culvert, the creek discharges into a small basin constructed as part of the Kilmer Creek Diversion.

Photograph A.1-2. Downstream view of Kilmer Creek at Dempsey Road (STMCUL00507). BGC photograph of November 18, 2014.

Photograph A.1-3. Upstream view of Kilmer Creek box culvert at Michener Way (STMCUL00045). BGC photograph of November 18, 2014.

The Kilmer Creek Diversion (STMMN04251) principally consists of two concrete culverts with diameters of 1800 mm (72 inch) and 600 mm (24 inch). The larger culvert runs due east under Kilmer Road for a distance of about 2.2 km before discharging into Lynn Creek (Raincoast Applied Ecology and KWL, 2013). The smaller culvert discharges back into the Kilmer Creek channel. The intent of the diversion is to limit peak flows in the downstream channel to the capacity of the 600 mm diameter baseflow culvert. A manually operated slide gate allows the baseflow culvert to be closed entirely. Two additional tributaries to Hastings Creek, Kilmer Creek and Coleman Creek, are intersected by the diversion. Similar to Kilmer Creek, flows from these two creeks are only partially diverted into the Kilmer Diversion and the remaining flow continues to Hastings Creek.

Concrete wing walls convey flow toward the diversion intake, while a trash rack envelops the entire structure. The trash rack prevents members of the public to enter the culverts, but also prevents woody debris from entering the culverts and blocking them (Photograph A.1-4). Similar trash racks have been constructed at the Thames Creek (STMMN09158) and Coleman Creek (STMMN09149) intakes. Both of those were reportedly blocked with woody debris during the Halloween 1981 flood, which resulted in Kilmer Road being washed out at Thames Creek (KWL, 1982).

Photograph A.1-4. Downstream view of Kilmer Creek looking at the intake structure and trash rack (STMMN04251). BGC photograph of November 18, 2014.

Photograph A.1-5. Upstream view of Kilmer Creek from the top of the intake structure (STMMN04251). BGC photograph of November 18, 2014.

Downstream of the diversion structure, Kilmer Creek is confined in a well-defined gully down cut through till to Wellington Drive where it passes through a third box culvert (STMCUL00043). The average channel gradient over this 600 m long reach is 16%. Downstream of the culvert the creek makes a sharp turn to the east toward Fromme Road, flowing through an 1800 mm diameter culvert under the road. The creek then flows through several residential properties in a southerly direction before flowing through a 900 mm diameter culvert under Argyle Secondary school, eventually discharging into Hastings Creek at the culvert outlet.

A.1.9.3.2 Impacted Areas

The November 2014 storm event had sufficiently intense rainfall to result in exceedance of a critical shear stress threshold of the channel bed on Kilmer Creek. This resulted in mobilization of the erodible portion of the creek bed sediments. Sediment mobilized by the debris flood in lower reaches of the creek passed through the two box culverts at Dempsey Road (STMCUL00507) and Michener Way (STMCUL00045), eventually depositing in the channel upstream of the diversion intake and filling the intake structure to the top of the trash rack. With the trash rack blocked, the creek flows overtopped the structure on the left (east) bank, downcutting into the granular fill of the access road. This downcutting resulted in exposure of the 1500 mm diversion culvert, as illustrated by Photograph A.1-6. The outlet of the Michener Way culvert was also almost completely blocked by the end of the storm event (Photograph A.1-7). DNV personnel estimated that about 300 m^3 of sediment had to be removed from the immediate vicinity of the diversion structure following the debris flood event (Steve Bridger, DNV, pers. comm.). It is not known how much sediment bypassed the diversion structure and deposited further downstream, although based on the aggradation observed in downstream reaches by BGC this figure could easily be on the order of two hundred cubic meters of sediment.

Much of the creek flow which overtopped the diversion structure was confined to the downstream Kilmer Creek channel. However, a portion of the flow also avulsed down the access road to the east, flowing onto Kilmer Road (Photograph A.1-8) and along Kilmer Road discharging downslope at topographic lows. As a result, a number of residential properties were impacted by shallow overland flow. The approximate extent of the damage caused by this overland flow was bounded by Kilmer Road to the north, Wellington Drive to the south, and Fromme Road to the east. Drawing A-1 shows known properties impacted by the flooding. About 20 m^3 of sediment had to be removed from properties along Kilmer Road following the debris flood (Mike Blackmon, DNV, pers. comm.).

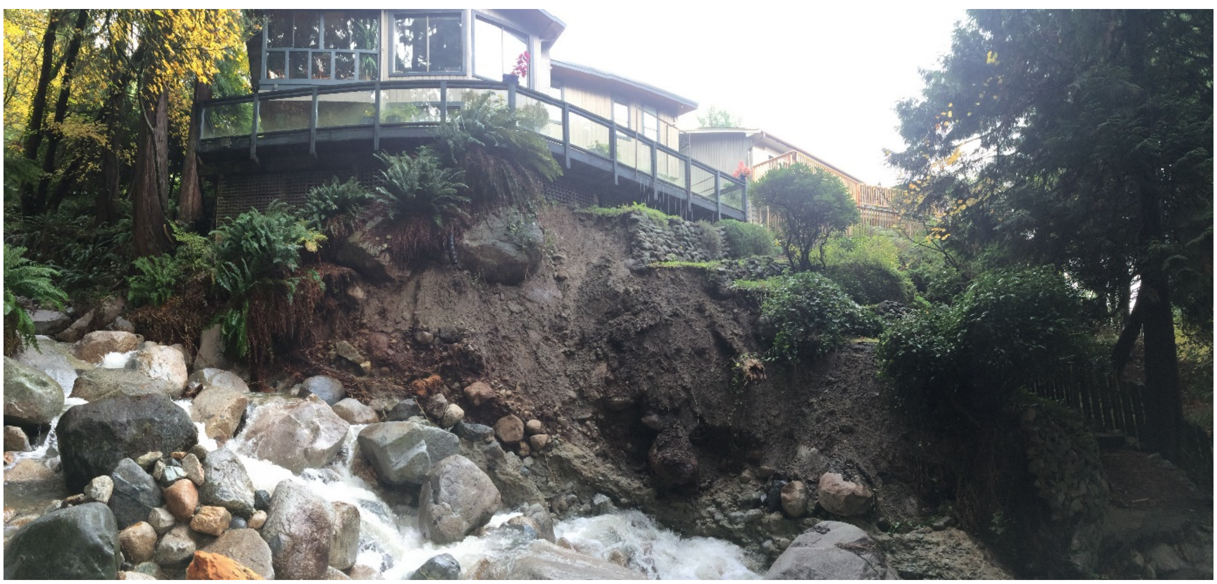
The flows that continued down Kilmer Creek also caused significant damage. At 1017 Doran Road, the high flows resulted in undermining of an over-steepened slope on the left bank, threatening the stability of a deck (Photograph A.1-9Further downstream, undermining and erosion of the right bank resulted in significant property damage to 1014 Wellington Drive, including damaging the deck beyond repair and threatening the foundation of the house (Photograph A.1-10). Proposed restoration works for those two properties are described in KWL (2014b).

Downstream of Wellington Drive, sediment mobilized in lower reaches of Kilmer Creek blocked the 1800 mm diameter culvert at Fromme Road. These overflows, combined with the overland flows from further upstream, continued down Fromme Road, eventually discharging into Hastings Creek (Photograph A.1-11). Properties on either side of Fromme Road were impacted by these overland flows (Drawing A-1). Approximately 130 m^3 of sediment was removed by DNV personnel from streets and properties following the debris flood (Mike Blackmon, pers. comm.).

Photograph A.1-6. Upstream view toward the Kilmer Creek Diversion. BGC photograph of November 4, 2014.

Photograph A.1-7. Upstream view of Kilmer Creek box culvert under Michener Way following the November 2014 debris flood. BGC photograph of November 4, 2014.

Photograph A.1-8. Upslope view of the Kilmer Creek Diversion intake from the access road. The avulsion flow path is delineated in blue. BGC photograph of November 18, 2014.



Photograph A.1-9. View looking east of eroded fillslope adjacent to 1017 Doran Road. BGC photograph of November 4, 2014.

Photograph A.1-10. View looking west of eroded bank of Kilmer Creek at 1014 Wellington Drive. BGC photograph of November 4, 2014.

Photograph A.1-11. Downstream view of Fromme Road at Hastings Creek. BGC photograph of November 4, 2014.

A.1.9.3.3 Short-Term Mitigation

Following the debris flood, a number of high priority short-term works were recommended by KWL (2014a, 2014b). These works included:

- Removal of debris in the "grizzly" barriers upstream of the Dempsey Road culvert
- Removal of a large fallen tree alongside and spanning the stream channel immediately upstream of the Michener Way culvert
- Removal of sediment at and upstream of the diversion inlet

- Restoration of the Kilmer Diversion pipe bedding, the pedestrian pathway and reconstruction of the collapsed bank immediately below the diversion
- Stabilization of the banks, debris removal and armouring of the channel at 1017 Doran Road and 1014 Wellington Drive
- Removal of debris within the Wellington Drive culvert basin
- Re-establishment and restoration of the channel through the L'Ecole Française Internationale de Vancouver property
- Removal of debris within the culvert beneath Fromme Road
- Armouring of the Fromme Road culvert outlet to minimize undercutting and further erosion at 3650 Fromme Road
- Removal of sediment within the channel to pre-flood conditions along Fromme Road from Croft Road to Frederick Road.

BGC understands that all of these short-term recommendations have been acted upon by the DNV.

A.1.9.3.4 Kilmer Creek above the Development Boundary

Hamish Weatherly, M.Sc., P.Geo., and Matthias Jakob, Ph.D., P.Geo., of BGC hiked the Kilmer Creek channel on November 18, 2014. The intent of the traverse was to assess the channel condition of Kilmer Creek above Dempsey Road and ascertain the cause of the debris flood. It was obvious that that the debris flood was the result of a critical shear stress threshold being exceeded, leading to mobilization of the erodible channel bed. Kilmer Creek is generally weakly incised in a morainal blanket of typically no more than 2 m depth. The looser ablation till (the morainal material that was deposited on top of the late Pleistocene glacier) has been incised and mobilized some time ago, and now the much denser basal till (the morainal material overridden by glacial ice) is exposed. Therefore, sources of sediment are limited to bank erosion and the very slow downcutting of the channel into the basal till.

The entire channel from Dempsey Road up to the Old Grouse Mountain Highway showed evidence of channel disturbance across the full channel width. Locally, the channel had scoured down to bedrock or the dense basal till (Photograph A.1-12, Photograph A.1-13). Channel aggradation was associated with local changes in channel gradient or large woody debris jams (Photograph A.1-14, Photograph A.1-15, Photograph A.1-16).

Photograph A.1-12. **Upstream view of Kilmer Creek scoured down to the underlying dense basal till. The hands of the person in mid picture indicate the approximate high water mark. BGC photograph of November 4, 2014.**

Photograph A.1-13. **Upstream view of Kilmer Creek scoured to bedrock. BGC photograph of November 18, 2014.**

Photograph A.1-14. Upstream view of sediment wedge developed behind a woody debris jam and large locked-in boulder. BGC photograph of November 18, 2014.

Photograph A.1-15. **Downstream view of sediment deposition and channel avulsion in a lower gradient channel section of Kilmer Creek. BGC photograph of November 18, 2014.**

Photograph A.1-16. **Downstream view of sediment deposition in a lower gradient channel section of Kilmer Creek. BGC photograph of November 18, 2014.**

The maximum grain size mobilized by the debris flood was estimated as 300 mm. Boulders up to 1 m in diameter are present in the channel, but it is extremely unlikely that these grain sizes would be mobilized even during an extreme event. These boulders represent eroded clasts from the basal till. They are now acting as agents of flow resistance, creating turbulence and slowing flows. Smaller grain sizes accumulate upstream, often creating small sediment wedges. Those wedges can be mobilized but the large boulders will stay in place.

A.1.9.3.5 Peak Flow

The peak flow of the 2014 November debris flood on Kilmer Creek were back-calculated based on high water marks observed along the channel, about 100 m upstream of Dempsey Road. Here, a 25 m section of creek displayed well-developed high water marks (Photograph A.1-17).

Photograph A.1-17. Upstream view of a high water mark measurement on Kilmer Creek. BGC photograph of August 26, 2015.

Eight cross-sections were surveyed using a tape measure at each of the high water marks and local channel gradients using a clinometer. Manning's equation for uniform flow was then used to estimate channel hydraulics at the eight cross-section locations. Results are summarized in Table A.1-5.

Manning's *n* was calculated using the formula of Jarrett (1984), who investigated roughness coefficients for steep cobble-boulder streams in Colorado. Jarrett's formula is a function of channel slope and hydraulic radius:

$$n = 0.39s^{0.38}R^{-0.16} \qquad \text{[Eq. A-1]}$$

where *s* is channel gradient (ft/ft) and *R* is the hydraulic radius (ft).

This approach is not ideal, because Manning's n values have not been investigated for channel gradients as steep as observed at the waypoints. Jarrett's (1984) research focused on streams with channel gradients of less than 5%. In comparison, channel gradients at the high water marks varied from 2% to 23% (Table A.1-5). However, research by Yochum et al. (2014) indicates that for greater than bankfull conditions, Manning's n values have been observed to vary between 0.1 and 0.3 for creeks with gradients between 10 and 35%. The Manning's n values calculated for the steeper reaches (Waypoints 1, 2 and 3) using Jarrett's method fall within this range.

Table A.1-5. Channel hydraulics for November 3, 2014 high water marks on Kilmer Creek.

Location	**Gradient (m/m)**	**Jarrett**		**Zimmerman**
		Manning's	**Peak Discharge (m^3/s)**	**Peak Discharge (m^3/s)**
Waypoint 1	0.20	0.15	6.1	11.9
Waypoint 2	0.20	0.16	4.9	8.8
Waypoint 3	0.23	0.16	4.6	8.3
Waypoint 4	0.06	0.10	2.4	1.9
Waypoint 5	0.08	0.11	7.0	8.6
Waypoint 6	0.08	0.11	4.2	4.6
Waypoint 7	0.07	0.11	2.7	2.2
Waypoint 8	0.02	0.06	3.0	1.1
		Average	**4.4**	**5.9**

The resulting peak flow estimates range between 2.4 and 6.1 m^3/s using the method of Jarrett (1984), with an average of 4.4 m^3/s. BGC also considered an experimental study by Zimmerman (2010), who investigated flow resistance in steep streams. Rather than use a traditional approach based on the use of a resistance co-efficient (i.e., Manning's n), Zimmerman developed a hydraulic geometry equation in which velocity is calculated as:

$$v = 2.3 g^{0.5} y^{1.2} D_{84}^{-0.72} S^{0.72} \qquad \text{[Eq. A-2]}$$

where v = velocity (m/s), y = flow depth (m), S = slope (m/m), and D_{84} (m) is the 84th percentile grain size of the channel substrate. Application of this equation resulted in higher peak discharges compared to Jarrett (1984). Based on the observed high water marks, the peak flow of the November 2014 event on Kilmer is thus estimated to have ranged between an average of approximately 4.4 and 5.9 m^3/s.

A.1.9.4. Thames Creek

On Thames Creek, two culverted crossings within the developed area became partially blocked: at McNair Drive (2 x 1200 mm diameter concrete culverts, STMCUL00152) and at Kilmer Road (600 mm diameter low flow concrete culvert with a 2100 mm diameter diversion culvert, STMCUL00412). These culvert locations are shown on Figure A.1-4. The two culverts at McNair

Drive were partially blocked by a combination of sediment and woody debris, resulting in some overland flow down Ramsay Road and damage to several homes (Drawing A-1). Debris was removed from the approach channel following the event (Dillon, 2014a).

At Kilmer Road, a trash rack (similar to the one in place on Kilmer Creek) covered the inlet to the diversion structure prior to the November 2014 storm. This trash rack became blocked with sediment and woody debris during the peak of the flood, resulting in overland flows across Kilmer Road and flood damage to the property immediately downstream. The trash rack and accumulated debris was removed from the culvert inlet following the storm event using a mini-excavator, which was lifted into the site (Dillon, 2014b). KWL (2015a) also reported a partial blockage of the culverted crossing at Dempsey Road (2 x 1200 mm diameter concrete culverts, STMCUL00409) during the event. A sanitary pipeline under Thames Creek also ruptured during the flood event along the eastern property line of 4660 Valley Road. This pipeline was repaired between November 14 and December 5, 2014 (Dillon, 2014c).

Following the flood event, KWL (2015a) was retained by the DNV to conduct a flood hazard and damage assessment of the upper half of Thames Creek (upstream of the Kilmer Diversion). The assessment includes recommendations for short-term channel restoration and repair work along this reach. A total of 17 proposed remedial and potential improvements works were identified by KWL (these works are identified on Figure A.1-4). Of these works, the following were considered to be higher priority and were recommended as short-term actions:

- Consider constructing a debris interceptor upstream of the two culverts at McNair Drive (ID #2)
- Remove deposited material and overgrown vegetation at numerous locations (ID #s 3, 7, 10, 15 and 17)
- Repair riprap for a pedestrian bridge (ID #11)
- Remove a failed pedestrian bridge, reconstruct the right bank and repair/install bank protection between McNair Drive and Valley Road (ID #s 5 and 6)
- Consult an arborist regrading a tree at the crest of an eroded bank (ID #12)
- Consider installing ‘grizzly’ barriers (debris barriers) upstream of the two culverts at Dempsey Road (ID #14).

DNV has informed BGC that these measures are being evaluated and balanced with the findings of the present study. In addition, the DNV has retained ISL Engineering and Land Services (ISL) to provide design drawings for debris barriers along Thames Creek at Mountain Highway (ID #1), McNair Drive (ID #3), and Kilmer Road (ID #17) (ISL, 2015). The barriers consist of 200 mm diameter sonotubes (steel tubes), which are embedded in concrete. The lengths and spacings specified for the tubes are unique to each crossing, as summarized below.

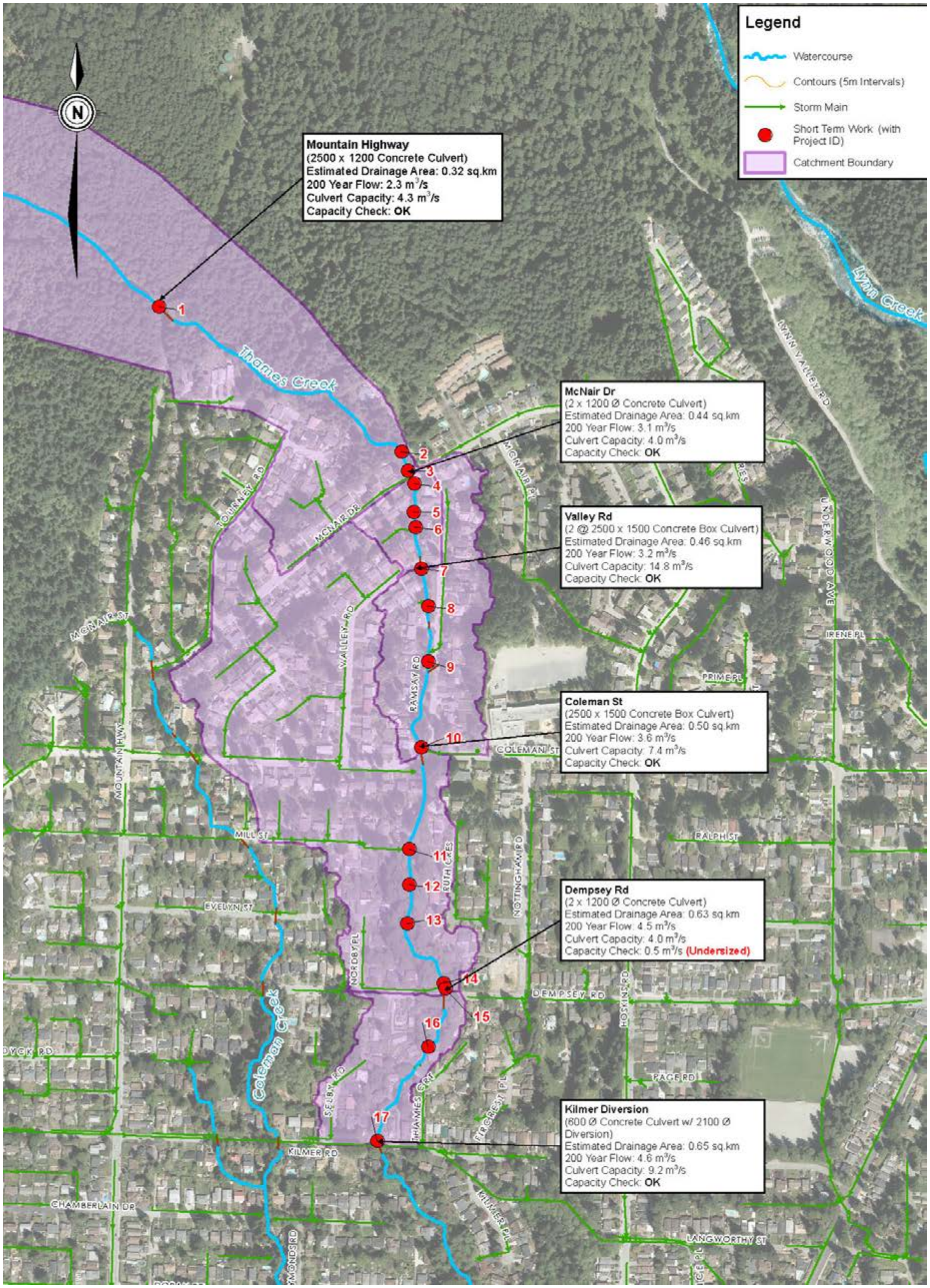


Figure A.1-4. Thames Creek potential short-term works (after KWL, 2015a).

Table A.1-6. Proposed debris barrier configurations along Thames Creek (after ISL, 2015).

Location	Width (m)	Total Length (m)	Buried Length (m)	Spacing (m)	Distance from Culvert Inlet (m)
Mountain Highway	5.8	2.7	1.35	0.8	1.5 to 3
McNair Drive	4.8	1.8	1.0	0.6	15.5
Kilmer Road	4.5	1.6	0.8	0.9	4.7

The proposed works also include a proposed headwall extension at McNair Drive and the re-installation of a trash rack at Kilmer Road.

Based on BGC's field assessment, it appears that a majority of the sediment mobilized on Thames Creek during the November 3, 2014 event originated downstream of Mountain Highway. BGC did not encounter evidence of a debris flood or significant sediment transport initiating upstream of Mountain Highway.

Downstream of the Kilmer Diversion, the channel of Thames Creek is considerably smaller given that peak flows are diverted out of the catchment. At 3531 Allan Road (above Lynn Valley Road), Thames Creek flows through a 1200 mm diameter pipe and discharges into a concrete channel that directs the creek around the back of the house before it passes into 3523 Allan Road. At the boundary between the two properties, the channel capacity was insufficient to convey the peak flow associated with the November 3, 2014, resulting in building and property damage to several residences (Drawing A-1). During a subsequent site inspection, KWL (2015b) observed that the streambed had aggraded to within 0.2 to 0.3 m of the top of bank at this location. This aggradation is likely associated with the November 2014 flood, as a partial blockage at the Kilmer Diversion likely resulted in elevated peak flows in downstream sections, compared to the usual scenario where the diversion is fully operational. KWL recommended that the channel be cleaned out at this location to return it to its pre-event capacity.

A.1.9.5. Mackay Creek

Immediately following the November 2014 storm, KWL (2014a) conducted a field review on Upper Mackay Creek between Ranger Avenue and approximately 200 m above the Powerline Trail (i.e., above the debris basin). KWL observed that the east branch of Upper Mackay Creek had experienced a small debris flow, which caused a significant amount of debris to accumulate above the east branch pedestrian bridge and severely damage it. KWL (2014a) recommended:

- Removal of debris accumulated at and in the channel above the damaged pedestrian bridge to reduce the potential of debris avulsing to the left (east) and around the debris basin during a high flow event
- Restoration of the maintenance access
- Replacement of the damaged pedestrian bridge (a relatively low bridge with minimal anchoring is preferred in order to reduce the potential for a blockage)

- Re-installation of warning signs to alert the public of the severe flood conditions during heavy rainfall and that the debris basin area is generally not safe.

Further downstream within Upper Mackay Creek Park, the western abutment of the June Smith Trail Bridge failed due to erosion. This pedestrian bridge is located north of 3906 Sunnycrest Drive. Repairs to the bridge were conducted between January 19 and 28, 2015 (Dillon, 2015a).

BGC inspected the site on November 4, 2014 during which time ponded water was observed at the Grouse Mountain parking lot and overtopping Powerline Trail. No sediment was observed at Grousewoods Dr. storm mains. At the time of the inspection, no evidence for flow down Nancy Greene Way was observed; however, the inspection was at mid-morning and DNV maintainance staff could have cleaned up evidence before this inspection.

A.1.9.6. Gallant Creek

Gallant Creek is crossed by roads three times along its lower reaches:

- At Badger Road (2 x 900 mm diameter concrete culverts, STMCUL00217, with a common concrete headwall; a debris barrier of 150 mm steel I-beams is located 10 m above the inlet)
- At Deep Cove Road (1200 mm diameter concrete culvert, STMCUL00580, with a concrete headwall and wingwalls)
- At Gallant Avenue (1200 mm diameter concrete culvert with two rebar debris barriers located 2 to 3 m above the inlet; an angular grate at the inlet further screens the culvert from debris).

Partial blockages were reported at all three crossings during the November 2014 flood, resulting in damages to properties below Deep Cove Road (Drawing A-2). Subsequent to the storm, sediment was removed from the culvert inlets at Badger Road and Deep Cove Road (Dillon, 2014d). ISL (2015f) recommended that the existing debris barriers at Badger Road and Gallant Avenue be replaced, as they were ineffective in trapping sediment, and that new barriers be constructed at all three crossings. ISL (2015f) also recommended that the section of Gallant Creek between Deep Cove Road and Gallant Avenue be realigned to improve flow hydraulics, and that the banks be built up and armoured with large diameter riprap.

ISL (2015h) was subsequently retained by the DNV to provide design drawings for some of the recommended elements including:

- A gravel maintenance road (3 m width) on the right bank of Gallant Creek at the Badger Road crossing to assist with culvert clean-out
- A debris barrier at Deep Cove Road consisting of 4 Sonotubes with a spacing of 0.6 m and a total length of 1.6 m (with 0.8 m buried and embedded in concrete)
- A re-aligned channel with armoured banks for the reach between Deep Cove Road and Gallant Avenue.

These designs were constructed in September 2015 (Photograph A.1-18 to Photograph A.1-20).

Photograph A.1-18. Downstream view of the armoured right bank of Gallant Creek downstream of Deep Cove Road. BGC photograph of October 19, 2015.

Photograph A.1-19. Upstream view of constructed debris barrier at Deep Cove Road. BGC photograph of October 19, 2015.

Photograph A.1-20. Upstream view of gravel maintenance road constructed adjacent to Gallant Creek at Badger Road. BGC photograph of October 19, 2015.

A.1.9.7. Other Creeks

A.1.9.7.1 Coleman Creek

A 1200 mm culvert conveys Coleman Creek underneath Coleman Street (STMCUL0400) and discharges into the western corner of 1343 Coleman Street. The alignment of the culvert directs flows toward the retaining wall and on-going erosion has undermined the retaining structure and the concrete slab at the outlet (BGC, 2015b). Following the November 2014 flood, this erosion issue became more acute and repairs were conducted by the DNV in February 2015 (Dillon, 2015b).

Figure A.1-5. Before and after views of Coleman Creek at 1343 Coleman Street (from Dillon, 2015b).

A.1.9.7.2 Mission Creek

A small tributary of Mission Creek flows through St. Albans Park and a residential property at 4442 Prospect Road. This tributary has a small storm inlet between 4442 and 4434 Prospect Road (STMMN01726) before being directed into a storm sewer and eventually discharging into Mission Creek just above Montroyal Boulevard. This inlet reportedly blocked during the November 2014 flood event, resulting in the flooding of several properties downslope of the inlet (Drawing A-1). According to KWL, the inlet and upstream channel through the subject property appears overgrown and prone to blockage by small woody debris (KWL, 2015b). KWL (2015c) recommended that the DNV consider reconfiguration of the inlet, maintenance of the channel and partial vegetation clearing to minimize the risk of overtopping and downstream flooding during future storm events.

Further downstream on Mission Creek, sediment accumulated behind debris barriers constructed upstream of West Windsor Road (Dillon, 2014e) and Evergreen Place (Dillon, 2014f) (Figure A.1-6). These sediments were removed from the channel following the November 2014 flood event.

Figure A.1-6. Upstream view of Mission Creek grizzly barriers at West Windsor Road (left) and Evergreen Place (right) (from Dillon, 2014d and 2014e). The image on the left was taken prior to debris removal, while the image on the right post-dates debris removal.

A.1.9.7.3 Thain Creek

Sediment was removed from behind a grizzly barrier located on Thain Creek at Evergreen Place (STMCUL00020) following the November 2014 flood (Dillon, 2014g). This site is located immediately due east of the Mission Creek crossing of Evergreen Place (Drawing A-1).

A.1.9.8. Follow-up Drainage Studies

Following the November 2014 storm, the DNV retained KWL and ISL to inspect all culverts downstream of the undeveloped/developed interface and conduct a desktop culvert capacity assessment. KWL (2015c) was tasked with assessing Mackay Creek, Mission Creek, Thain Creek, Kilmer Creek, Coleman Creek and Thames Creek. ISL was tasked with assessing Canyon

Creek (2015b), McCartney Creek (2015c), Parkside Creek (2015d), Taylor Creek (2015e), Gallant Creek (2015f) and the Deep Cove Creeks (2015g).

Both studies noted that a number of culverts appeared to be undersized for the design flow (200-year peak instantaneous flow). NHC is currently undertaking a drainage model study for the DNV, which will result in updated design flows for all of the above creeks. Therefore, the conclusions of the KWL and ISL reports may be revised following the NHC study.

Table A.1-7 summarizes recommendations by KWL and ISL in addition to identification of undersized culverts.

A.1.10. August 2015 Storm Event

On August 31, 2015, a storm event in DNV required overnight monitoring and removal of debris from culverts on Mission Creek at Newdale Crt and Kilmer Creek at Frederick Rd.

At Newdale Crt (STMCUL00269), DNV personnel reported that the trash rack on the culvert was plugging with debris every hour and needed to be cleaned to prevent flooding of the home downstream. At Frederick Rd. (STMCUL00175) across from Argyle Secondary School, DNV personnel reported ongoing sedimentation throughout the previous night resulting in the removal of five small truckloads of rock and debris (S. Ono, DNV, email, September 1, 2015).

A.1.11. 2016 Mackay Debris Flow Event

In early 2016, DNV alerted BGC to a small debris flow event on Mackay Creek. BGC visited the site on February 12th, 2016 and summarized findings in BGC (2016).

Table A.1-7. Proposed remedial works by KWL (2015c) and ISL (2015b-g).

Watershed	Location	Asset ID	Action
Mackay	Grouse Mountain Overflow Parking Lot (East End)	STMCUL00364	Clear overgrown vegetation and consider upgrading the downstream channel and pipe network (Grousewoods Drive culvert 5) to accommodate the Q_{200}
	Grouse Mountain Lower Overflow Parking	STMCUL00249	Consider removal of the outlet structure at the Works Yard to improve hydraulics and reduce the risk of culvert blockages.
	Grousewoods Drive Stormwater system	STMMN00181 to STMMN00198	Consider a review and upgrade of the storm system and the creek/stream inlets to reduce the risk of surcharge and subsequent flooding.
	5559 Staghorn Place (Grousewoods Drive Culvert 10)	STMCUL00622	Clear away overgrown vegetation and remove deposited gravels to restore culvert capacity
Mission	Mission Creek at Monterray Avenue	STMCUL00270	Consider channel and bank stabilization upstream of the culvert to limit erosion and debris mobilization
	Culvert between 4442 and 4434 Prospect Road	STMMN01726	Consider reconfiguration of the inlet, maintenance of the channel and partial clearing of the landscaping to minimize the risk of overtopping.

Watershed	Location	Asset ID	Action
Coleman	4737 Mountain Highway	STMMN03828	Consider widening the upstream channel approach and reconfiguring the storm inlet to reduce the probability of blockages
	Coleman Creek at Mill Street	STMCUL00163	Consider replacement or repair of the Delta-Lock wall to limit the ongoing erosion at the culvert outlet.
Thames	Thames Creek at Mountain Highway	STMCUL00052	Consider regrading Mountain Highway at the Thames Creek culvert to create an overflow spillway. A culvert blockage could direct water down Mountain Highway into the Coleman Creek catchment with the potential to cause significant flood damage.
Canyon	Hyannis Drive culvert	STMCUL00548	Armour the eroded area of the inlet with riprap and re-align a 20 m section upstream of the inlet to match the culvert alignment.
	2440 and 2433 Riverside Drive	STMMN04914	Erosion is occurring on private land and it is recommended that the homeowners retain a geotechnical consultant to conduct a slope assessment.
McCartney	Larkhall Crescent culvert	STMCUL00308	Remove large trees that have fallen into the creek channel downstream of the culvert. Also remove accumulated debris at the barrier located upstream of the inlet.
	Mount Seymour Parkway	STMCUL00299	The dam sac headwall at the Mount Seymour Parkway culvert inlet has partially failed and caused the west bank to erode. Recommended to construct a new concrete headwall.
Taylor	Mount Seymour Parkway	STMCUL00259	Re-align a 20 m section of Taylor Creek above the inlet to match the culvert alignment. The upstream flow path is perpendicular to the culvert alignment and erosion has begun on the west embankment behind the concrete wingwall.
Parkside	Slope failure		Approximately 70 m upstream of the Cliffwood Road multi-use pathway (MUP) crossing, a slope failure is contributing sediment to the creek during storm events. Recommended that fallen trees be removed from the channel, as the trees are exacerbating bank erosion.
	Cliffwood Road culvert	STMCUL00427	The above-mentioned slope failure resulted in some sediment deposition upstream of and within the MUP box culvert (0.3 m thick). Recommended that the culvert be cleaned out and a new steel barrier be installed, as the existing barrier was ineffective during the storm.
	Deep Cove Road culvert	STMCUL00427	During an initial site visit, the culvert was almost completely blocked by sediment with a maximum size of 300 mm. The culvert was subsequently cleaned out (see Dillon, 2015c). An existing grizzly barrier (three I-beam steel posts) above the culvert inlet was ineffective in collecting boulder debris and ISL recommends a new steel barrier 2 m upstream of the inlet.
Gallant Creek	Indian River Drive	STMCUL00181	Gallant Creek crosses Indian River Drive at two locations. Both crossings consist of 4 m wide box culverts, which appear to be more than 75% filled with sediment. These culverts need to be cleaned out.

Watershed	Location	Asset ID	Action
Panorama Creek	Mount Seymour Road		Debris needs to be cleaned from the inlet of the 900 mm diameter culvert.
	Indian River Drive		The existing culvert is a 1200 mm diameter wood stave culvert at the inlet and changes to a 1050 mm diameter concrete culvert 2 m short of the outlet. A sinkhole has developed at the interface, creating a culvert blockage. Culvert needs replacing.
	Indian River Drive		A second tributary is located immediately north of the crossing noted above. Here the 600 mm diameter culvert bottom is severely eroded and needs to be replaced. At the inlet to the culvert, there is a forebay in the ditch constructed with concrete walls. Stormwater enters the forebay from the upstream ditch via a cutout in the concrete wall. The cutout is partially blocked and needs cleaning, and does the approach ditch/

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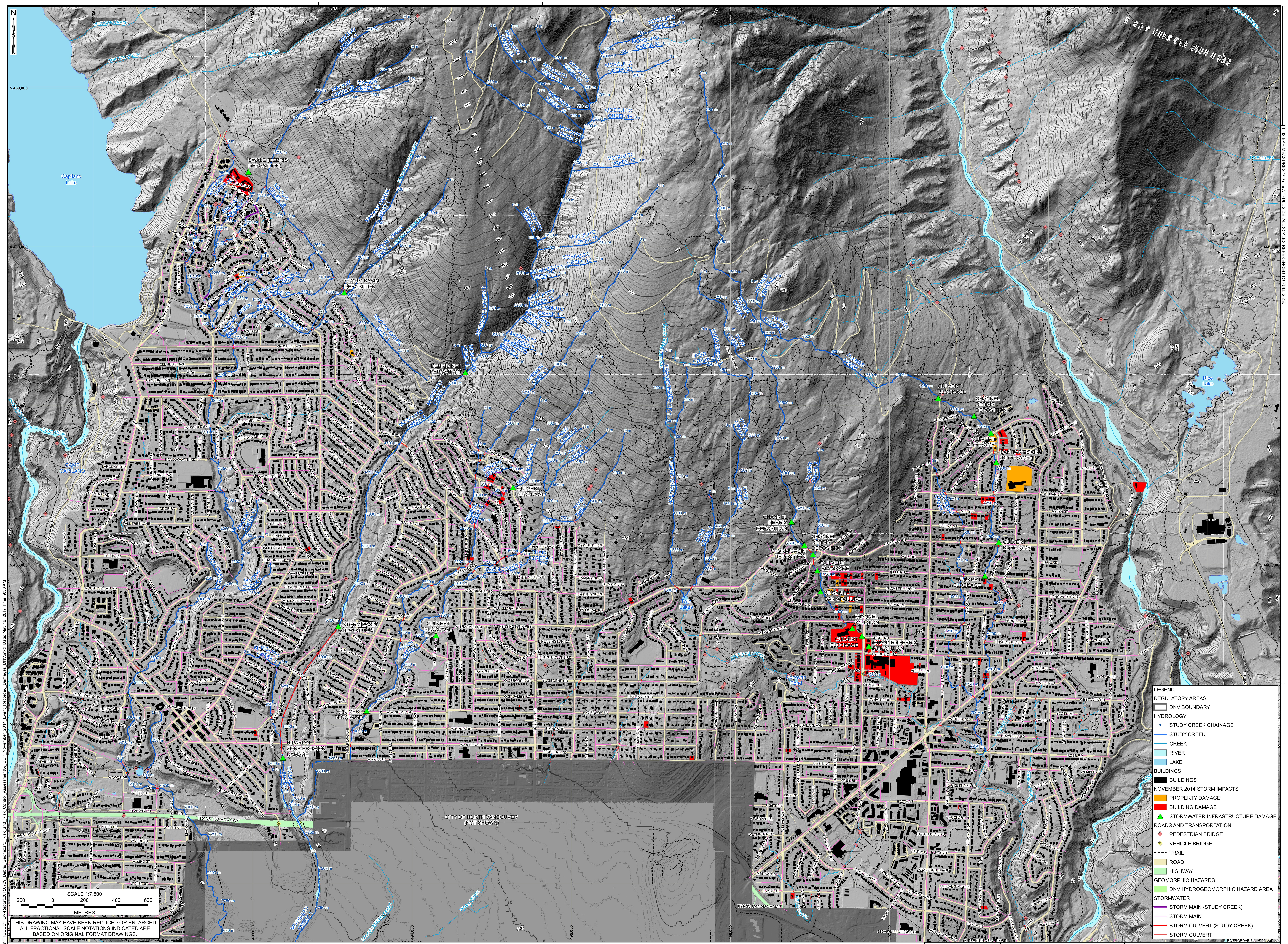

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PROJECT: DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT

TITLE: NOVEMBER 2014 EVENT RECORDED DAMAGES DNV WEST

PROJECT No.: 0404054

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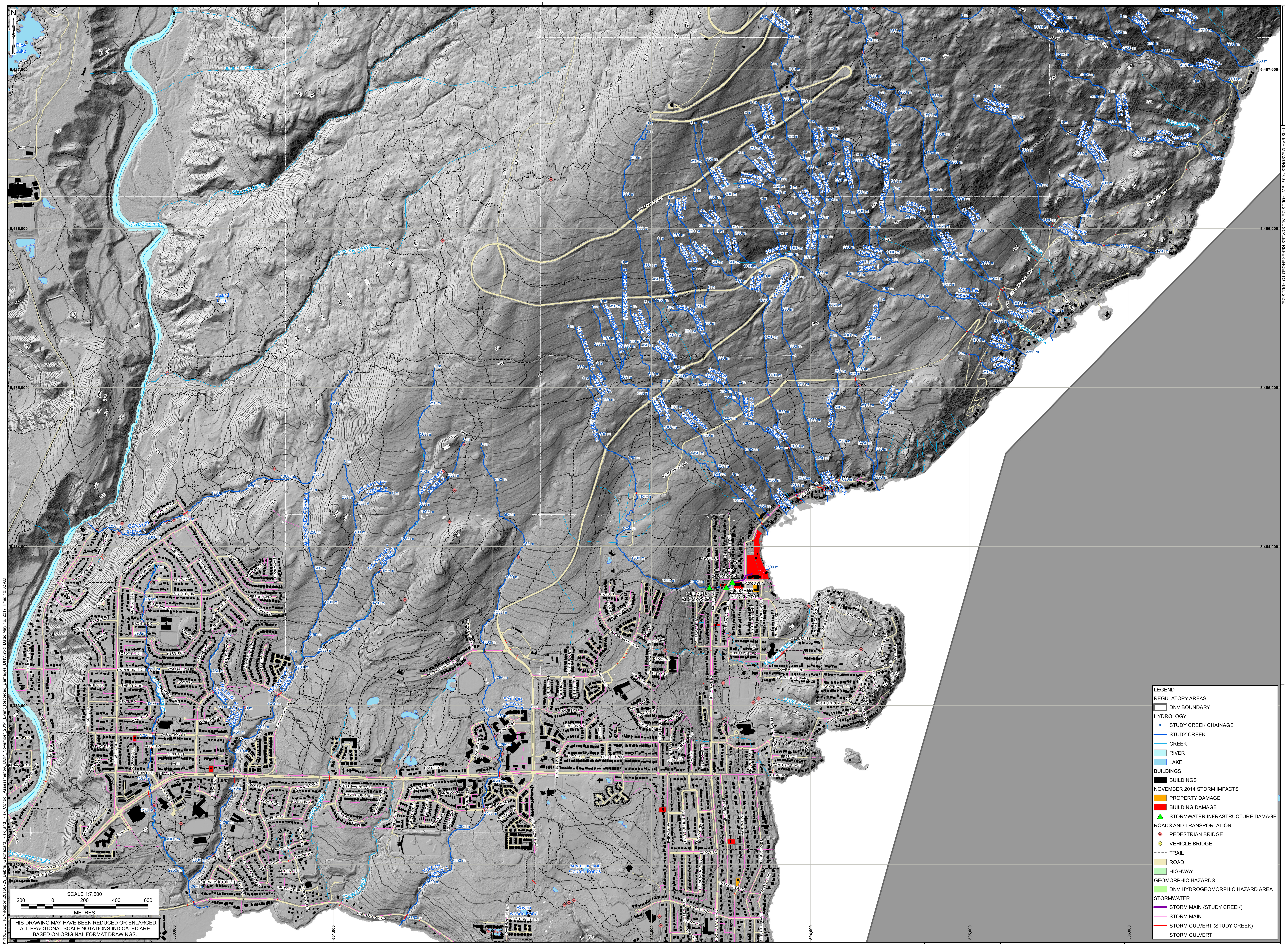

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PROJECT: DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT

TITLE: NOVEMBER 2014 EVENT RECORDED DAMAGES DNV EAST

PROJECT No.: 0404054

DWG No.: A-2

APPENDIX B
HYDROGEOMORPHIC FLOODS

B.1. HYDROGEOMORPHIC FLOODS

Steep mountain creeks (here-in defined as having channel gradients steeper than 5%) are typically subject to a spectrum of mass movement processes that range from clear water floods to debris floods to hyperconcentrated flows to debris flows in order of increasing sediment concentration. In this report they are referred to collectively as hydrogeomorphic[1] floods or processes. There is a continuum between these processes in space and time with floods transitioning into debris floods and eventually debris flows through progressive sediment entrainment. Conversely, dilution of a debris flow through partial sediment deposition and tributary injection of water can lead to a transition towards hyperconcentrated flows and debris floods and eventually floods.

In BC, most infrastructure on such creeks have been designed for clearwater floods with return periods of up to 200 years. This design does not account for hydrogeomorphic processes such as debris floods and debris flows in which parts of or the entire channel bed sediments are mobilized and lead to erosion of channel bed and banks and debris inundation on terminal alluvial fans (Jakob et al., 2015).

Ignoring the specific hydrogeomorphic processes that act on steep creeks can and has led to a plethora of problems, many of which are caused by the fact that culverts and sometimes bridges have not been designed for heavy sediment loads or severe bank erosion. When such culverts are overwhelmed, blockage and re-direction of waters and sediment can occur.

B.1.1. Steep Creeks

Hydrogeomorphic floods are a phenomenon of steep channels. The morphology and processes in steep channels have been described by Church (2010, 2013). Sediment transfer occurs by a continuum of processes ranging from fluvial transport (bedload and suspended load) through debris floods to debris flows. These phenomena are transitional within time and space along the channel, depending on the sediment-water mixture. To understand the significance of these different modes of sediment transfer it is useful to consider the characteristic anatomy of a steep channel system. Steep mountain slopes deliver sedimentary debris to the upper channels by rock fall, rock slides, debris avalanches, debris flows, slumps and raveling. Landslides may create temporary dams that pond water: when the dam breaks, a debris flow may be initiated in the channel. Debris flows and debris floods characteristically gain power and material as they move downstream, debouching onto a terminal fan where the channel enters the main valley floor. Here sediment is deposited and widespread damage may occur (Jakob et al., 2015).

The following subsections adapted from Jakob et al. (2015) provides a brief summary of debris flow and debris flood processes.

[1] Hydrogeomorphology is an interdisciplinary science that focuses on the interaction and linkage of hydrologic processes with landforms or earth materials and the interaction of geomorphic processes with surface and subsurface water in temporal and spatial dimensions (Sidle and Onda, 2004).

B.2. DEBRIS FLOW

'Debris flow', as defined by Hungr et al. (2014), is a very rapid, channelized flow of saturated debris containing fines (i.e., sand and finer fractions) with a plasticity index of less than 5%. Debris flows originate from single or distributed source areas in regolith mobilized by the influx of ground- or surface water. Liquefaction occurs shortly after the onset of landsliding due to turbulent mixing of water and sediment, and the slurry begins to flow downstream, 'bulking' by entraining additional water and channel debris.

Sediment bulking is the process by which rapidly flowing water entrains bed and bank materials either through erosion or preferential "plucking" until a certain sediment conveyance capacity (saturation) is reached. At this time, further sediment entrainment may still occur through bank undercutting and transitional deposition of debris with a zero net change in sediment concentration. The volume of the flowing mass is thereby increased (bulked). Bulking may be confined to partial channel substrate mobilization of the top gravel layer, or – in the case of debris flows – may entail entrainment of the entire loose channel debris. Scour to bedrock in the transport zone is expected.

Unlike debris avalanches, which travel on unconfined slopes, debris flows travel in confined channels bordered by steep slopes. In this environment, the flow volume, peak discharge, and flow depth increase, and the debris becomes sorted along the flow path. Debris-flow physics are highly complex and video recordings of events in progress have demonstrated that no unique rheology can describe the range of mechanical behaviours observed (Iverson, 1997). Flow velocities typically range from 1 to 10 m/s, although very large debris flows from volcanic edifices, often containing substantial fines, can travel at more than 20 m/s along much of their path (Major et al., 2005). The front of the rapidly advancing flow is steep and commonly followed by several secondary surges that form due to particle segregation and upwards or outwards migration of boulders. Hence, one of the distinguishing characteristics of coarse granular debris flows is vertical inverse grading, in which larger particles are concentrated at the top of the deposit. This characteristic behaviour leads to the formation of lateral levees along the channel that become part of the debris flow legacy. Similarly, depositional lobes are formed where frictional resistance from coarse-grained or large organic debris-rich fronts is high enough to slow and eventually stop the motion of the trailing liquefied debris. Debris-flow deposits remain saturated for some time after deposition, but become rigid once seepage and desiccation have removed pore water.

Typical debris flows require a channel gradient of at least 27% (15°) for transport over significant distances (Takahashi, 1991) and have volumetric sediment concentrations in excess of 50%. Between the main surges a fluid slurry with a hyperconcentration (>10%) of suspended fines occurs. Transport is possible at gradients as low as 20% (11°), although some type of momentum transfer from side-slope landslides is needed to sustain flow on those slopes. Debris flows may continue to run out onto lower gradients even as they lose momentum and drain: the higher the fines content, and hence the slower the sediment-water mixture loses its water content, the lower the ultimate stopping angle. The silt-clay fraction is thus the most important textural control on debris-flow mobility. The surface gradient of a debris-flow fan approximates the stopping angle for flows issuing from the drainage basin.

Due to their high flow velocities, peak discharges are at least an order of magnitude larger than those of comparable return-period floods. Further, the large caliber of transported sediment and wood means that debris flows are highly destructive along their channels and on fans.

Channel banks can be severely eroded during debris flows, although lateral erosion is often associated with the trailing hyperconcentrated flow phase that is characterized by lower volumetric sediment concentrations. The most severe damage results from direct impact of large clasts or coarse woody debris against structures that are not designed for the impact forces. Even where the supporting walls of buildings may be able to withstand the loads associated with debris flows, building windows and doors are crushed and debris may enter the building, leading to extensive damage to the interior of the structure (Jakob et al., 2012). Similarly, linear infrastructure such as roads and railways are subject to complete destruction. On fans, debris flows tend to deposit their sediment rather than scour. Therefore, exposure or rupture of buried infrastructure such as telecommunication lines or pipelines is very rare. However, if a linear infrastructure is buried in a recent debris deposit, it is likely that over time or during a significant runoff event, the tractive forces of water will erode through the debris until an equilibrium slope is achieved, and the infrastructure thereby becomes exposed. This necessitates understanding the geomorphic state of the fans being traversed by a buried linear infrastructure.

Avulsions are likely in poorly confined channel sections, particularly on the outside of channel bends where debris flows tend to superelevate. Sudden loss of confinement and decrease in channel slope cause debris flows to decelerate, drain their inter-granular water, and increase shearing resistance, which slow the advancing bouldery flow front and block the channel. The more fluid afterflow (hyperconcentrated flow) is then often deflected by the slowing front, leading to secondary avulsions and the creation of distributary channels on the fan. Because debris flows often display surging behaviour, in which bouldery fronts alternate with hyperconcentrated afterflows, the cycle of coarse bouldery lobe and levee formation and afterflow deflection can be repeated several times during a single debris flow event. These flow aberrations and varying rheological characteristics pose a particular challenge to numerical modelers seeking to create an equivalent fluid (Iverson, 2014).

Figure B.2-1 summarizes the different hydrogeomorphic processes by their appearance in plan form, velocity and sediment concentration.

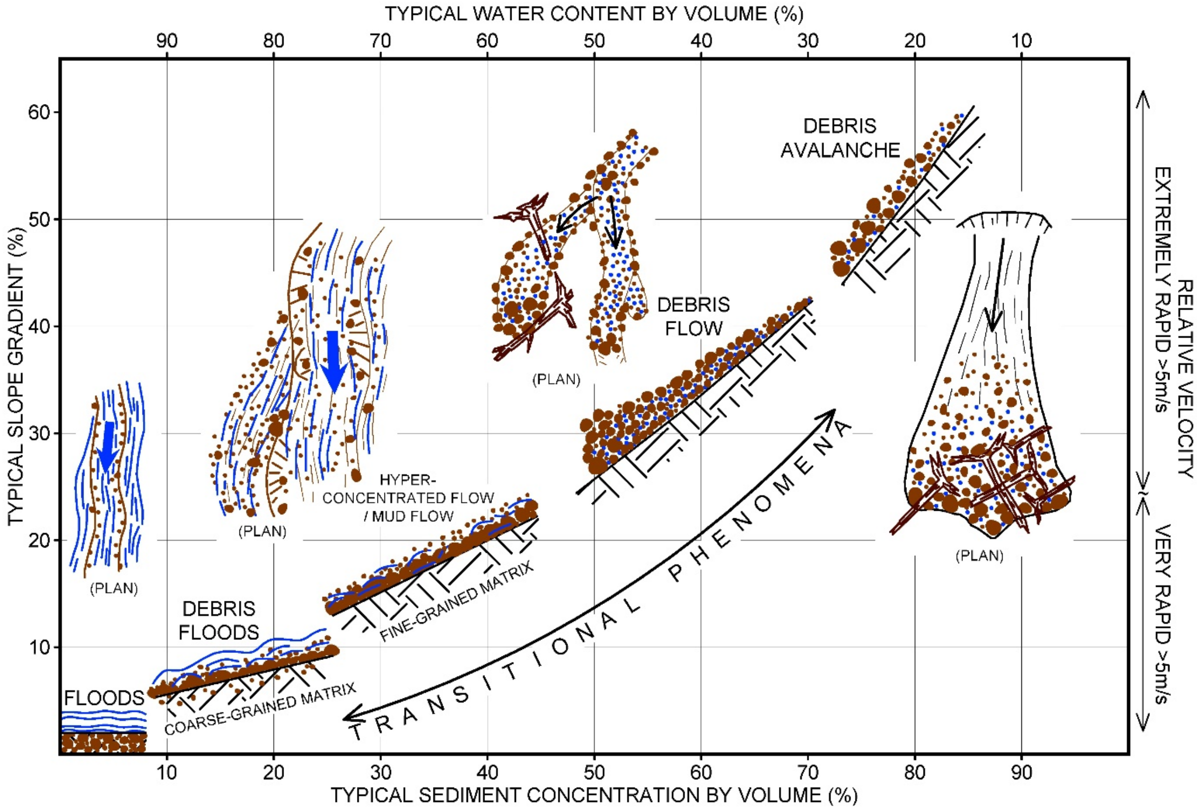

Figure B.2-1. Hydrogeomorphic process classification by sediment concentration, slope, velocity and planform appearance.

B.3. DEBRIS FLOODS AND HYPERCONCENTRATED FLOWS

A ‘debris flood’ is “a very rapid surging flow of water heavily charged with debris in a steep channel” (Hungr et al., 2014). Transitions from floods to debris floods occur at minimum volumetric sediment concentrations of 3 to 10%, the exact value depending on the particle size distribution of the entrained sediment and the ability to acquire yield strength[2]. Because debris floods are characterized by heavy bedload transport, rather than by a more homogenous mixture of suspended sediments typical of hyperconcentrated flows (Pierson, 2005), the exact definition of sediment concentration depends on how sediment is transported in the water column. Debris floods typically occur on creeks with channel gradients between 5 and 30% (3-17º).

The term “debris flood” is similar to the term “hyperconcentrated flow”, defined by Pierson (2005) on the basis of sediment concentration as “a type of two-phase, non-Newtonian flow of sediment and water that operates between normal streamflow (water flow) and debris flow (or mudflow)”. Debris floods (as defined by Hungr et al., 2014) have lower sediment concentrations than hyperconcentrated flows (as defined by Pierson). Thus, there is a continuum of geomorphic

[2] The yield strength is the internal resistance of the sediment mixture to shear stress deformation; it is the result of friction between grains and cohesion (Pierson, 2005).

events that progress from floods to debris floods to hyperconcentrated flows to debris flows, as volumetric sediment concentrations increase. Some creeks are hybrids, which implies that the dominant process oscillates between debris floods and debris flows. For example, Shone Creek on Indian Arm is capable of producing debris flows, most of which are likely to dilute into debris floods by the time they reach lower reaches of the low-gradient fan. However, debris flows from Underhill Creek, which joins Shone Creek near the fan apex, can retain debris-flow characteristics until reaching the ocean. Other creeks, such as Percy Creek, are unlikely to have a debris-flood phase because their channels are very steep and because very large (> 3 m diameter) boulders provide significant flow resistance hindering debris flood generation.

Due to their initially relatively low sediment concentration, debris floods are more erosive along channel banks and beds than debris flows; the latter can reach a sediment saturation point whereby bank or bed erosion is significantly reduced. Bank erosion and excessive amounts of bedload introduce large amounts of sediment to the fan where they accumulate (aggrade) in channel sections with decreased slope. In fact, debris floods can be initiated on the fan itself through rapid bed erosion and entrainment of bank materials. Because typical synoptic storm hydrographs fluctuate several times over the course of the storm, several cycles of aggradation and remobilization of deposited sediments on channel and fan reaches can be expected during the same event (Jakob et al., 2015).

Debris floods can be triggered by a variety of processes. One trigger is transition from a debris flow when lower stream channel gradients are encountered (Shone Creek and Coldwell Creek are examples). Another trigger is exceedance of a critical shear stress threshold of the channel bed and full bed mobilization (Church, 2013). More uncommon triggers are landslide dam, beaver dam or glacial lake outburst floods as well as the failure of man-made dams (Jakob and Jordan, 2001; Jakob et al., 2015). Photograph B.3-1 is an example of a debris flood triggered by the failure of a human-made dam on Cougar Creek in Canmore, Alberta. The recent flood event on Kilmer Creek in November 2014 is an excellent example of a debris flood being initiated by full bed mobilization. Additional details of the recent event on Kilmer Creek are provided in Appendix A and Appendix G.

Photograph B.3-1. Example of the failure of a human-made dam on Cougar Creek shortly after the breach initiated. May 25, 1990 (Alberta Environment, 1991).

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APPENDIX C
DISTRICT OF NORTH VANCOUVER HAZARD INFORMATION TOOL (DNVHIT)

C.1. INTRODUCTION

The DNV Hazard Info Tool (DNVHIT) is an online-accessible, interactive map that displays debris geohazards[1] identified and characterized by BGC as they relate to DNV drainage infrastructure.

DNVHIT displays results from the 2016 Debris Geohazard Risk and Risk Control Assessment completed by BGC for DNV. In future it may additionally include the results of periodic inspections, a reporting database, and additional geohazard types (e.g., additional landslide hazards) as a more comprehensive geohazard risk management application for DNV.

The DNVHIT terms, conditions, and limitations are displayed upon login. By clicking ***Ok***, the user confirms that he/she has read, understands, and agrees to those terms, conditions, and limitations; DNVHIT will then continue to the map interface.

C.2. NAVIGATION

Figure C.2-1 presents a screen shot from the DNVHIT map interface. Map navigation is similar to Google Maps. To move the map (pan):

- Click and hold the left mouse button, then drag; or
- Use the keyboard's arrow keys.

Progressively higher detail is shown at increased zoom levels. To zoom in and out of the map:

- Use the mouse scroll wheel or trackpad;
- Click on the "+" or "-" symbols on the upper left corner of the map; or
- Double-click to zoom in.

The base map can be viewed as either topographic or satellite imagery; a toggle button to switch between the two is located at the lower right corner of the map window. Base map data sources are shown on the lower right corner of the map.

[1] Debris hazard (geohazard): the continuum of floods, debris floods and debris flows (referred to as hydrogeomorphic processes) with their associated phenomena of channel bed scour, bank erosion, avulsion and debris deposition, that have the potential to cause economic damages, injury and potential loss of life.

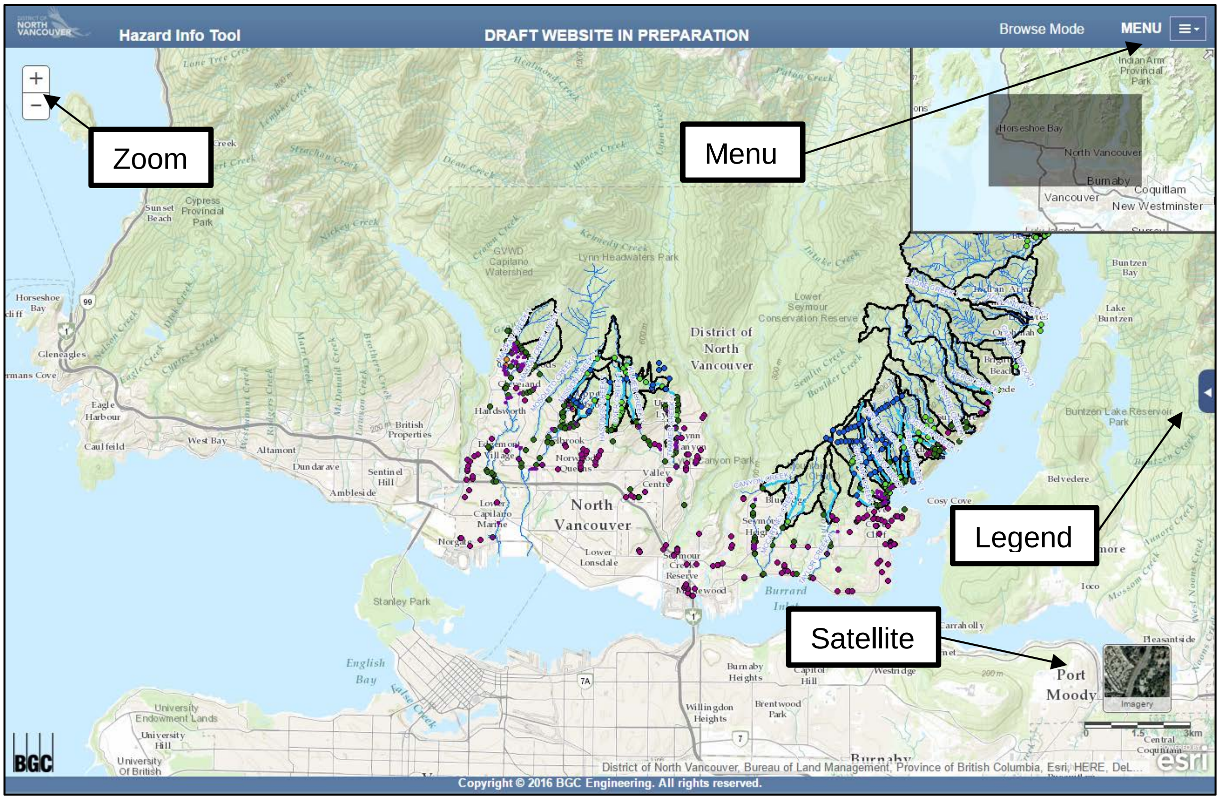

Figure C.2-1. Online map overview.

C.3. MAP LEGEND AND INFO

Scrolling over the arrow on the right edge of the screen, and clicking ***Map Legend & Info***, opens a sidebar on the right side of the map. This sidebar contains:

1. ***Search.*** This section allows searches for creeks, culverts, debris control structures, bridges, and addresses. Creeks are searched by name. Culverts, debris control structures and bridges are searched by asset ID. Addresses are searched by street number and street name. To search:
    a. Select the search type from the drop-down menu
    b. Scroll through the dropdown list to select the feature of interest, or begin typing the feature's name or asset ID.
2. ***Basemap Layers.*** This section allows the user to select which datatypes to display on the map and which to hide using checkboxes. Layers are organized into the following categories:
    - Bridges
    - Storm Water
    - Debris Control Structure
    - Hydrology

- Slope Stability (not included in this report)
- Buildings

3. ***Legend*.** The legend expands to show more features as they appear at higher zoom levels on the map. The legend lists and defines symbols for:
    - Storm Water
    - Debris Control Structure
    - Hydrology
    - Buildings

    a. ***Metadata and Limitations*.** This drop-down list the limitations to which the user agreed before entering DNVHIT, and data sources for the base map and elements at risk displayed on DNVHIT.

    b. ***Instructions.*** Contains this document for referral.

    c. ***Measurements.*** Contains tools for measuring area and distance, as well as location latitude and longitude. To start a measurement, select an icon from the options in the drop down. To close this tool when a measurement is complete, click on the same measurement icon in the sidebar.

The map legend can be collapsed by clicking the *arrow* on the left edge of the sidebar.

C.4. ADDITIONAL INFORMATION SIDEBAR

Clicking on an asset, watershed or creek on the map opens a pop-up window with basic information about the selected item. For culverts, storm mains, bridges, and debris control structures, there is a link to more info. Likewise, creek segments that were hiked as part of the field investigation (creek segments shown in lighter blue) can be clicked for more information. This link will open a sidebar along the left side of the window, which displays dropdown headings which can be clicked to reveal more information about the selected item.

The available information differs between the infrastructure and creeks. Table C.4-1 outlines the information available for each type of selection. All clickable infrastructure has an embedded option to open a separate Google Maps window to see the area in aerial or street view.

Table C.4-1. Summary of information presented for DNV infrastructure, creek segments.

Item	Information Presented
Culvert / Storm Main	Photographs
	Risk Rating
	Hazard Rating
	Consequence Rating
	Asset Information
	Inspections
	Risk Control
Debris Control Structure	Asset Information
	Risk Control
	Photos
Bridge	Watershed Summary
	Asset Information
	2015 Inspection
	Photos
Creek Segment	Location
	Channel Characteristics
	Sediment Characteristics
	Description
	Photos

The small “i” information icon located on each subrow will open a pop-up when hovered over that provides the source of the information provided. Figure C.4-1 shows the layout of the sidebar. Photos and Inspections are displayed chronologically. The site is designed to display multiple inspections to show changes at each location. This allows the information displayed by DNVHIT to change and adapt as upgrades are made to assets.

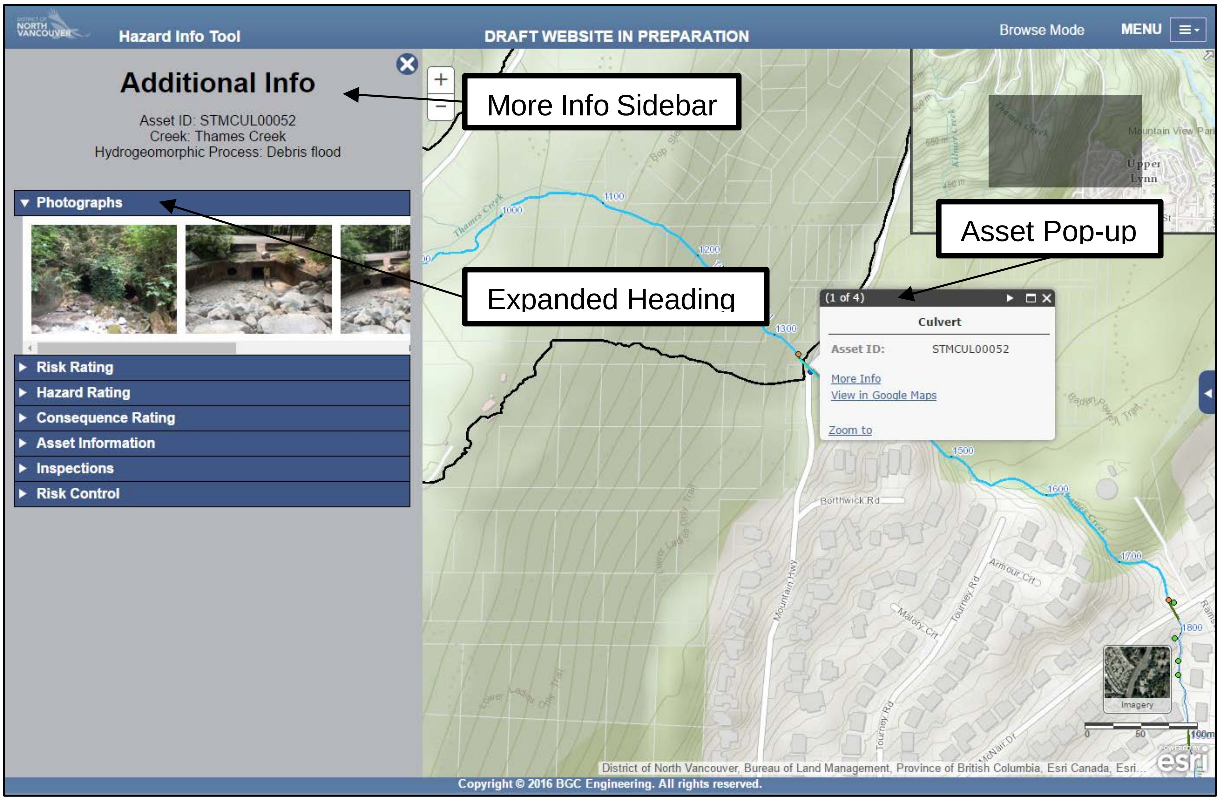

Figure C.4-1. Sidebar layout

APPENDIX D
PREVIOUS ASSESSMENTS

D.1. PREVIOUS ASSESSMENTS

Table D.1-1. Summary of previous assessments reviewed by BGC.

Report Number	Report Name	Report Type	Source	Date Published
CDNV DISTRICT HALL 2769705 v1	Recorded Damages Drawings 01-03	2014 Damages	BGC	Aug-15
CDNV DISTRICT HALL 2333279 v1	Mackay Creek and Mosquito Creek ISMP RFP	Integrated Storm Water Management Plan	City of North Vancouver	Mar-14
	Geologic hazard and risk assessment, Lot 2, DL1249, Plan 27929, Lillooet District, Mars Crossing near Birken, BC	Hazard and Risk Assessment	Cordilleran Geoscience	22-Oct-08
	Geologic Hazards assessment for proposed residence, 303 Sasamat Lane, North Vancouver, BC	Hazard Assessment	Cordilleran Geoscience	17-Dec-11
	Geologic hazard assessment for subdivision approval, Keir property on Walker Creek, Gun Lake near Goldbridge, BC	Hazard Assessment	Cordilleran Geoscience	9-Jul-15
	2672 Panorama Dr. Flood Hazard Report	Hazard Assessment	CREUS Engineering Ltd.	Jul-14
CDNV DISTRICT HALL 2584312 v1	Environmental Monitoring - Upper Mackay Debris Removal at Powerline Trail	2014 Damages	Dillon	9-Jan-14
CDNV DISTRICT HALL 2605947 v1	Mosquito Creek Channel Improvements Drawing	2014 Damages	Dillon	Jul-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Kilmer Diversion Structure	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Mount Fromme Debris Jam, Thames Creek	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Upper Mackay Debris Removal at Powerline Trail	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Kilmer Creek between Frederick and Wellington	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Kilmer Creek	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Gallant Creek at Badger Road Debris Jam	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584306 v1	Environmental Monitoring - Braemar Elementary Storm Culvert	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584307 v1	Environmental Monitoring - Kilmer Creek Trash Rack	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584324 v1	Environmental Monitoring - Gallant Creek Debris Removal	2014 Damages	Dillon	Nov-14
CDNV DISTRICT HALL 2584308 v1	Environmental Monitoring - Lower MacKay Dam	2014 Damages	Dillon	6-Nov-14
CDNV DISTRICT HALL 2584309 v1	Environmental Monitoring - Thames Creek McNair Drive Debris Jam	2014 Damages	Dillon	19-Nov-14
CDNV DISTRICT HALL 2584311 v1	Environmental Monitoring - Thames Creek Trash Rack	2014 Damages	Dillon	20-Nov-14
CDNV DISTRICT HALL 2584314 v1	Environmental Monitoring - Mission Creek Trash Rack at Windsor Road	2014 Damages	Dillon	20-Nov-14
CDNV DISTRICT HALL 2584315 v1	Environmental Monitoring - Thain Creek 480 Evergreen Place Trash Rack	2014 Damages	Dillon	20-Nov-14
CDNV DISTRICT HALL 2584316 v1	Environmental Monitoring - Mission Creek 550 Evergreen Place Trash Rack	2014 Damages	Dillon	20-Nov-14
CDNV DISTRICT HALL 2584313 v1	Environmental Monitoring - Frederick Road to Wellington Drive	2014 Damages	Dillon	5-Dec-14
CDNV DISTRICT HALL 2584325 v1	Environmental Monitoring - June Smith Bridge within Upper Mackay Creek Park	2014 Damages	Dillon	Jan-15
CDNV DISTRICT HALL 2584318 v1	Enivronmental Monitoring - Coleman Creek Erosion Project	2014 Damages	Dillon	Feb-15
CDNV DISTRICT HALL 2584319 v1	Environmental Monitoring - Deep Cove Road Debris Removal at Parkside Creek	2014 Damages	Dillon	Feb-15
CDNV DISTRICT HALL 2584321 v1	Environmental Monitoring - Thames Creek at 4660 Valley Road	2014 Damages	Dillon	Dec-15
CDNV-DISTRICT HALL 1266622 v1	Proposed Bank Protection at 5171 Ranger Avenue, North Vancouver	Mitigation	DNV	2-Nov-96

Report Number	Report Name	Report Type	Source	Date Published
CDVN-DISTRICT HALL 1266622 v1	Proposed Residential Development 302 and 303 Sasamat Lane, North Vancouver	Inspections	DNV	19-Jan-11
DNV2645189	Local Government Body Recovery Plan	2014 Damages	DNV	27-Mar-14
DNV2567800	November Flooding Financial Report	2014 Damages	DNV	Mar-15
2658403-2691	2691 Panorama Drive Subdivision Conditions Letter Addendum	Hazard Assessment	DNV	23-Jun-15
	Area 1 Inlet Hotpots	Hot Spots	DNV	9-Jul-15
	Area 2 Inlet Hotpots	Hot Spots	DNV	9-Jul-15
	Area 3 Inlet Hotpots	Hot Spots	DNV	9-Jul-15
	Creek Channel Stability and Potential for Erosion	Mitigation	DNV	13-Oct-15
	DNV Nov 2014 Damage Records.kmz	2014 Damages	DNV	-
CDNV DISTRICT HALL 2554046 v1	Canyon Creek Assessment	Hazard Assessment	ISL	20-Feb-15
CDNV DISTRICT HALL 2554047 v1	Gallant Creek Assessment	Hazard Assessment	ISL	20-Feb-15
CDNV DISTRICT HALL 2554048 v1	McCartney Creek Assessment	Hazard Assessment	ISL	20-Feb-15
CDNV DISTRICT HALL 2554055 v1	Taylor Creek Assessment	Hazard Assessment	ISL	20-Feb-15
CDNV DISTRICT HALL 2554054 v1	Parkside Creek Assessment	Hazard Assessment	ISL	24-Feb-15
	Panorama Area Culvert Assessments	Culverts	ISL	24-Mar-15
	Report on Mission Creek	Inspections	KWL	Apr-82
	Report on McCartney Creek	Inspections	KWL	Jun-82
	Report on Hastings Creek	Inspections	KWL	Jun-82
	Report on Deep Cove-Dollarton Area	Inspections	KWL	Jul-82
	Debris Flow Study and Risk Mitigation Alternatives for Clegg Creek	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Coldwell Creek and Friar Creek	Hazard Assessment	KWL	Dec-03
	Debris Flow - Debris Flood Study and Risk Mitigation Alternatives for Deep Cove Creeks	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Holmden Creek	Hazard Assessment	KWL	Dec-03
	Summary of KWL Reports	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Mackay Creek	Hazard Assessment	KWL	Dec-03
	Debris Flood Study and Risk Mitigation Alternatives for Mosquito Creek	Hazard Assessment	KWL	Dec-03
	Debris Flood Study and Risk Mitigation Alternatives for Ostler Creek and Allan Creek	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Percy Creek and Vapour Creek	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Scott-Goldie Creek and Sunshine Creek	Hazard Assessment	KWL	Dec-03
	Debris Flow Study and Risk Mitigation Alternatives for Shone Creek	Hazard Assessment	KWL	Dec-03
	Summary Report on Debris Flow Studies	Hazard Assessment	KWL	Dec-03
	Camp Jubilee Flood and Geohazard Assessment	Hazard Assessment	KWL	Aug-11
	Hastings Creek Watershed: Ecology and Hydrotechnical Assessment	Environmental Assessment	KWL	Jun-13
CDNV DISTRICT HALL 2383031 v1	Upper Mackay Creek Debris Basin - Inspection Report	Inspections	KWL	Jul-14

Report Number	Report Name	Report Type	Source	Date Published
	Creek Hydrology Floodplain Mapping and Bridge Hydraulic Assessment	Hazard Assessment	KWL	24-Oct-14
CDNV DISTRICT HALL 2480682 v1	Creek Restoration Works - Kilmer Creek Wellington Drive to Doran Road	Mitigation	KWL	18-Nov-14
CDNV DISTRICT HALL 2480683 v1	North Vancouver Flood Response - High Priority Short-term Works	Mitigation	KWL	18-Nov-14
CDNV DISTRICT HALL 2487358 v1	Mackay and Mosquito Watersheds ISMP Phase 1 Report	Inspections	KWL	20-Nov-14
	Creek Summary Drawings	Culverts	KWL	Feb-15
CDNV DISTRICT HALL 2609755 v1	North Vancouver Flood Response - Stream Bank Erosion Inspections	2014 Damages	KWL	30-Apr-15
	Proposed Kilmer Debris Basin Drawing	Mitigation	KWL	May-15
	Thames Creek Restoration - Short Term Works	Mitigation	KWL	30-Jun-15
CDNV DISTRICT HALL 2560451	North Vancouver Interface Inpsection and Capacity Assessment	Culverts	KWL	20-Jul-15
	Debris Flood Quantitative Risk Assessment, 2672 Panorama Drive	Risk Assessment	LaCas Consultants Inc.	11-Jun-15
2730890-2691	Gavles Creek Preliminary Assessment Report - Creek Hazard DPA	Hazard Assessment	LaCas Consultants Inc.	28-Aug-15
	Request for Proposal: Drainage System Model Development & Assessment	Hazard Assessment	NHC	13-Feb-15
GSC OF7677	A Profile of Earthquake Risk for the District of North Vancouver	Hazard Assessment	NRCAN	2015
	Request for Variance of a RAR SPEA, 2672 Panorama Drive	Environmental Assessment	Phoenix Environmental Services	14-May-15
	Kilmer Creek Debris Flow Hazard Assessment and Geotechnical Recommendations Related to Proposed Debris Basin	Hazard Assessment	Thurber	Jun-15
15022 S1	Deep Cove Residence Creek Channel Drawing S1	Mitigation	W Architecture	-

APPENDIX E
HYDROLOGICAL ASSESSMENT METHODOLOGY

E.1. INTRODUCTION

E.1.1. Rainfall-Runoff Modelling

A rainfall-runoff approach was used to provide a preliminary estimate of the hydrographs for the 20-year, 50-year and 200-year return periods for each of the steep creeks in the study. Hydrographs were used as inputs to define the frequency-magnitude relations described in Section 4.0. The Soil Conservation Service (SCS) unit hydrograph method (SCS, 1972) was implemented using the HEC-HMS (Version 4.1) program developed by the US Army Corps of Engineers (USACE, 2015). This method is widely used to derive synthetic unit hydrographs and applies a design storm event and physical watershed characteristics to predict peak flows.

Required inputs to the model include:

- The storm event hyetograph (rainfall distribution).
- The time of concentration (Tc) defined as the time taken for the storm runoff event to travel from the most remote point of a basin to the point of interest.
- A curve number (CN), an empirically derived relationship between soil type, land use, antecedent conditions and runoff used to establish initial soil moisture conditions and infiltration response. CN values for various hydrologic soil groups are provided in USACE (2000).

E.1.2. Storm Event Hyetograph

A SCS Type 1A storm event hyetograph was used for the rainfall-runoff simulation. This storm type has been shown to accurately generate flood runoff from watersheds within the region (Loukas, 1994). Rainfall totals of 167 mm, 194 mm and 235 mm were used to represent the storm event for the 20-year, 50-year and 200-year return periods over a 24-hour duration, respectively (Table E.1-1). These values are based on regional intensity-duration-frequency (IDF) rainfall curves previously generated by BGC for Metro Vancouver (BGC, 2009). In that study, publicly-available precipitation data from ninety-three local Environment Canada and Metro Vancouver stations were used to develop regional-scale representation of precipitation intensity and delineate the Metro Vancouver area into homogeneous regions of precipitation conditions.

Table E.1-1. 24-hour rainfall totals adapted from BGC (2009).

Duration	Rainfall for Given Return Periods (mm)		
	20-year	50-year	200-year
24 hr	167	194	235

E.1.3. Peak Flow Estimates

Recent floodplain mapping work conducted by KWL (2015) reported a 200-year return period unit peak flow in the range of 7.1 to 7.4 $m^3/s/km^2$ for undeveloped (i.e., forested) portions of the Mackay Creek, Mosquito Creek, Mission Creek, Thain Creek and Hasting Creek watersheds using HEC-HMS. Given that the NHC study is ongoing and preliminary results are not yet available, BGC has initially assumed that the 200-year unit peak instantaneous flow for the watershed areas above as well as Dyer Creek and Thames Creek is 7.4 $m^3/s/km^2$. By extension, unit peak flows of 4.2 $m^3/s/km^2$ and 5.3 $m^3/s/km^2$ have been adopted by BGC for return periods of 20 and 50 years.

East of Lynn Creek, the unit peak flow for each return period was adjusted to account for reduced orographic effects at lower elevations and increased orographic effects at higher elevations. In this manner, the peak flows were scaled for each watershed by the ratio of the mean elevation of the watershed area to that of the Kilmer watershed, while also recognizing that there is a precipitation limit to the scaling factor. That is, elevations near sea level on the North Shore receive about 40% less precipitation compared to the values reported in Table E.1-1 (BGC 2009).

Table E.1-2 outlines the mean elevations of the watersheds and unit flow adjustment factors, while Table E.1-3 shows the attendant peak flows at the watershed outlet for each creek.

Table E.1-2. Mean elevation of watersheds.

Watershed	Location	Mean Elevation (masl)	Mean Elevation Ratio[1]	Unit Flow Adjustment
Mission	North Shore	673	-	1
Thain	North Shore	465	-	1
Dyer	North Shore	508	-	1
Hastings	North Shore	582	-	1
Kilmer	North Shore	684	-	1
Kilmer Tributary	North Shore	430	0.63	0.85
Thames	North Shore	502	-	1
Canyon	East of Lynn	257	0.38	0.75
McCartney	East of Lynn	275	0.4	0.76
Taylor	East of Lynn	264	0.39	0.75
Gallant	Indian Arm	376	0.55	0.82
Panorama	Deep Cove	397	0.58	0.83
Kai	Deep Cove	120	0.18	0.67
Mathews Brook	Deep Cove	382	0.56	0.82
Gavles	Deep Cove	429	0.63	0.85

[1] Relative to the mean elevation of the Kilmer Creek Watershed.

Watershed	Location	Mean Elevation (masl)	Mean Elevation Ratio[1]	Unit Flow Adjustment
Cleopatra	Deep Cove	260	0.38	0.75
Cove	Deep Cove	428	0.63	0.85
Martin	Deep Cove	155	0.23	0.69
Francis	Deep Cove	586	0.86	0.94
Unnamed[2]	Indian Arm	181	0.26	0.71
Ward	Indian Arm	250	0.37	0.75
Ostler	Indian Arm	458	0.67	0.87
Allan	Indian Arm	634	0.93	0.97
Sunshine	Indian Arm	382	0.56	0.82
Scott Goldie	Indian Arm	837	1.22	1.09
Percy	Indian Arm	890	1.3	1.12
Vapour	Indian Arm	564	0.82	0.93
Gardner Brook	Indian Arm	207	0.3	0.72
Shone	Indian Arm	731	1.07	1.03
Underhill	Indian Arm	575	0.84	0.94
Holmden	Indian Arm	500	0.73	0.89
Ragland	Indian Arm	323	0.47	0.79
Coldwell	Indian Arm	812	1.19	1.07
Friar	Indian Arm	384	0.56	0.82

Table E.1-3. Peak flows at watershed outlet by watershed area and mean elevation.

Watershed	Location	Watershed Area (km^2)	Peak Flows (m^3/s)		
			10-30 year	30-100 year	100-300 year
Mackay (west)	North Shore	0.78	3.4	4.2	5.8
Mackay (east)	North Shore	0.58	2.5	3.1	4.3
Mission	North Shore	0.28	1.2	1.5	2.1
Thain	North Shore	0.52	2.2	2.8	3.8
Dyer	North Shore	0.76	3.3	4.1	5.6
Hastings	North Shore	0.35	1.5	1.9	2.6
Kilmer	North Shore	0.77	3.3	4.2	5.7
Kilmer Tributary	North Shore	0.16	0.7	0.9	1.2
Coleman[3]	North Shore	-	-	-	-
Thames	North Shore	0.53	2.2	2.8	3.9

[2] Unnamed creek immediately west of Ward Creek.
[3] Coleman does not have a watershed area upstream of DNV development.

Watershed	Location	Watershed Area (km^2)	Peak Flows (m^3/s)		
			10-30 year	30-100 year	100-300 year
Canyon	East of Lynn	0.72	2.3	2.9	4.0
McCartney	East of Lynn	1.57	5.3	6.6	8.8
Taylor	East of Lynn	0.59	1.9	2.4	3.3
Gallant	Deep Cove	1.14	4.1	5.1	6.9
Panorama	Deep Cove	0.70	2.5	3.2	4.3
Kai	Deep Cove	0.06	0.16	0.21	0.30
Mathews Brook	Deep Cove	0.30	1.1	1.3	1.9
Gavles	Deep Cove	0.56	2.0	2.6	3.5
Cleopatra	Deep Cove	0.24	0.8	1.0	1.4
Cove	Deep Cove	0.46	1.7	2.1	2.9
Martin	Deep Cove	0.16	0.4	0.6	0.8
Francis	Deep Cove	1.72	7.2	8.9	12.0
Unnamed[4]	Indian Arm	0.19	0.6	0.8	1.1
Ward	Indian Arm	0.16	0.5	0.6	0.9
Ostler	Indian Arm	0.82	3.1	3.9	5.3
Allan	Indian Arm	1.10	4.7	5.8	7.9
Sunshine	Indian Arm	1.15	4.1	5.2	7.0
Scott Goldie	Indian Arm	2.96	14	18	24
Percy	Indian Arm	1.99	10	12	17
Vapour	Indian Arm	0.62	2.5	3.1	4.2
Gardner Brook	Indian Arm	0.58	1.8	2.2	3.1
Shone	Indian Arm	2.73	13	16	21
Underhill	Indian Arm	0.27	1.1	1.4	1.9
Ragland	Indian Arm	0.37	1.2	1.6	2.2
Holmden	Indian Arm	2.03	8.0	10.0	13.0
Coldwell	Indian Arm	4.65	23	28	37
Friar	Indian Arm	0.43	1.5	1.9	2.6

Using these unit peak flows and employing HEC-HMS, BGC generated interim hydrographs for the study creeks for the three return period classes under consideration (10-30 years, 30-100 years, and 100-300 years). BGC obtained these unit peak flows by adjusting CN values in the HEC-HMS model.

[4] Unnamed creek immediately west of Ward Creek.

Downstream of the development interface, BGC estimated peak flows at DNV storm culverts and water mains to facilitate assessment of blockage potential in order to develop hazard scenarios for modelling.

In order to do so, BGC used the ratio of peak flows reported by KWL (2015) to scale the BGC estimates to culverts downstream of the development interface. In this manner, if KWL reported a flow of 2.3 m^3/s for the culvert at the watershed outlet (Culvert A) and 2.6 m^3/s for a culvert downstream (Culvert B), then in order to estimate the peak flow at Culvert B based on BGC's estimated peak flow at Culvert A, BGC multiplied the peak flow at Culvert A by the ratio 2.6/2.3. This method was employed where possible based on the available flow data reported by KWL (2015). An exception is the Mission Creek culvert at Newdale Court. For this culvert, BGC used estimates of Tetra Tech (2017), who developed a PC-SWMM model to predict the 200-year return period flow in Mission Creek using recent IDF curves developed for DNV.

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APPENDIX F
STORMWATER INFRASTRUCTURE PARAMETERS
(PROVIDED IN DIGITAL FORMAT)

APPENDIX F
STORMWATER INFRASTRUCTURE PARAMETERS
(PROVIDED IN DIGITAL FORMAT)

F.1. STORMWATER INFRASTRUCTURE PARAMETERS

Placeholder, will be provided digitally as a file geodatabase as part of the Final Report..

APPENDIX G
DEBRIS FLOOD METHODOLOGY

G.1. INTRODUCTION

This appendix describes the methodology for determining the peak flow and debris sediment volume of debris floods. Debris flows are addressed in Appendix K where applicable. Debris floods can be triggered by exceedance of a critical discharge threshold in an alluvial channel bed (i.e., Church 2013), or from a unique geomorphic event in the watershed, such as a landslide dam or glacial dam breach (i.e., Jakob et al. 2015). It appears that sediment was mobilized in Kilmer Creek through exceedance of a critical shear stress threshold that led to mass mobilization of the channel bed. Data collected at this creek were used to develop a methodology to assess debris flood hazards for creeks within the DNV.

Note that research into sediment transport in steep creeks is still relatively limited, in part because some creeks can show little to no sediment transport for several decades or more. As a result, issues related to the prediction of sediment transport remain an active and under-studied field of research.

G.2. DEBRIS FLOOD MAGNITUDE

Hazard and risk estimates (and ultimately potential mitigation) to existing development, require knowledge of the expected sediment volume at the urban development interface for a range of return periods. To estimate sediment volume for a rainstorms of variable return periods, the following steps were followed:

1. Determine the critical shear stress required for bed mobilization.
2. Use average channel dimensions and Manning's equation[1] to determine the debris flood discharge that corresponds with the critical shear stress.
3. Develop a hydrograph using HEC-HMS as outlined in Appendix E associated with a specific return period to calculate the amount of time that the flow exceeded the discharge threshold.
4. Select an appropriate sediment transport equation to calculate sediment discharge based on stream power.
5. Calculate sediment volume based on the estimated sediment discharge (4.) multiplied by the duration over which the critical shear stress occurs.

[1] Manning's equation is an empirical relationship used to determine the velocity of water in open channel flow. The equation is $Q = \left(\frac{1}{n}\right) AR^{\frac{2}{3}}\sqrt{S}$ where Q is the flow rate, n is the Manning's roughness coefficient, A is the flow area, R is the hydraulic radius and S is the channel slope.

G.2.1. Step 1 – Critical Shear Stress

Many widely used bedload sediment transport models are based on the concept that sediment transport begins at, or can be scaled by, a constant value of the non-dimensional bed shear stress (Lamb et al. 2008). This non-dimensional bed shear stress, Θ_c, is also known as Shields stress:

$$\theta_c = \frac{\tau_g}{(\rho_s - \rho)gD} \quad \text{[Eq. G-1]}$$

$$\tau_g = \rho g R S \quad \text{[Eq. G-2]}$$

where T_g = shear stress at the bed, D is the diameter of a particle, R is the hydraulic radius, S is the energy slope (typically assumed to be the channel gradient for steeper channels), g is the acceleration due to gravity, and ρ_s and ρ are the densities of sediment and fluid, respectively. Assuming a sediment density of 2.65 kg/m^3, Equation G-1 can be re-written as:

$$\theta_c = \frac{RS}{1.65D} \quad \text{[Eq. G-3]}$$

The above equations are based on the work of Shields (1936) who considered bedload movement a threshold phenomenon and established a diagram relating the dimensionless critical shear stress (Equation G-1) to the roughness Reynolds number, *Re**. Under most natural flow conditions (rough and turbulent flows), Shields stress at incipient motion is roughly constant (i.e., Θ_c = 0.045). Shields (1936) proposed an asymptotic value of 0.06 for Θ_c. However, more generally values in the range of 0.03 to 0.07 have been proposed (Buffington and Montgomery, 1997). Defining a critical threshold is achieved by considering that in most gravel-bed rivers Shields number Θ barely exceeds 20% of the Θ_c during floods and that for these flow conditions, transport rates increase by several orders of magnitude for very small changes in shear stress (Recking, 2009). As a result, bedload prediction can result in very large errors if Θ_c is incorrectly specified.

If Θ_c is a constant, then Equation G-1 indicates that smaller particles are more mobile[2], as they require less shear stress to move. However, most studies have shown that sediment is more equally mobile (i.e., all grain sizes tend to mobilize near the same shear stress) than that predicted by Equation G-1, because of the hiding and exposure effects of a non-uniform grain size distribution (i.e., larger particles tend protrude above the bed, while smaller particles tend to be shielded from the flow by larger particles). Particles larger than the D_{50} are relatively easier to move than the same particles in a uniform bed material because they project above the smaller size and experience a higher drag force (see Bathurst, 2013); the pivoting angle through which they need to be tipped to begin moving may also be smaller. Particles smaller than the reference size are more difficult to move than if they were in a uniform bed material, because they are hidden behind larger particles and the pivoting angle is larger.

[2] Mobility is used to describe the boundary shear stress necessary to initiate sediment movement and does not refer to bedload transport rate.

Incipient motion of a non-uniform bed can then be reasonably determined using a single value of Θ_c for the mixture with the representative grain diameter in Equation G-1 set to $D = D_{50}$, where D_{50} is the median grain size. Finer particles are considered to move at slightly lower shear stresses than coarser particles (e.g., Parker 1990; Ferguson 2003).

Critical Shear Stress and Slope

Equation G-3 indicates that Shields parameter has a dependence with the relative depth *R/D* (the ratio between the hydraulic radius R and the grain diameter D) and the slope *S*. Shields (1936) himself recognized a potential slope dependency on Θ_c, observing increasing Shields stress with increasing slope. This means that the steeper a creek is the higher the bed shear stress required to move particles, which is counter-intuitive as explained below. This observation has since been confirmed by a number of researchers by flume and field experiments (e.g., Bathurst 1987; Mueller et al.,2005; Lamb et al. 2008). An example of this relation is shown in Figure G.2-1.

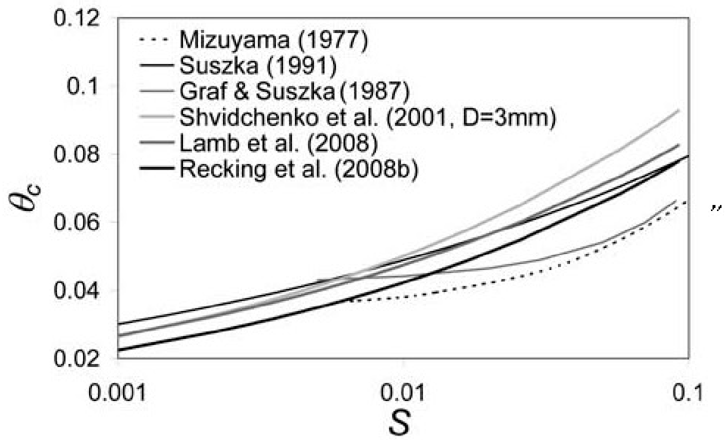

Figure G.2-1. Comparison between different critical Shields functions derived for rough turbulent flows in flume experiments (after Recking 2009).

The relation between critical Shields stress and channel slope was under-reported for a number of years because of two factors. First, a majority of studies in the past have been focused on lower gradient gravel-bed streams. This factor was recognized by Bathurst et al. (1982) who hypothesized that the traditional Shields approach (which assumes a constant value of 0.04 to 0.06 at high Reynolds numbers) could be based on the coincidence that most studies involved a relatively narrow range of shallow slopes such that the real variation of Θ_c with slope had been too small to appreciate. Second, increased sediment mobility with increasing slope could logically be expected due to the added gravitational force in the downstream direction. The fact that the opposite is observed has been attributed to a number of factors including:

- Some of the stress available for sediment transport is lost as fluid drag on larger particles (Mueller et al. 2005).
- Stabilizing bed structures are likely to develop in steep creeks (Church et al. 1998). This means that when big boulders get locked together, they provide a flow constriction that is very difficult to mobilize even during very high peak flows.

- Friction angles and grain emergence (Lamb et al. 2008).
- Changes to the vertical structure of flow velocity (Lamb et al. 2008).
- Turbulent fluctuations (Sumer et al. 2003).

Lamb et al. (2008) concluded that the local velocity acting on the grains must decrease with increasing channel slope, for the same shear stress and particle size. Extension to a non-uniform bed indicates that the coarse fraction becomes increasingly less mobile on steeper slopes.

Based on a compilation of previously published flume and field data, Lamb et al. (2008) provide the following relation:

$$\theta_c = 0.15S^{0.25} \qquad \text{[Eq. G-4]}$$

In the above relation, slope is defined as $S = \tan\beta$, where β is the bed slope angle from horizontal. The best fit line in a least squares sense has an r-square value of 0.41, but with a lack of data for channel gradients > 10% (Figure G.2-2). Lamb et al. (2008) attributed the scatter in the data due to differences in friction angles, drag from channel walls and morphologic structures on the bed, sediment shapes and size distribution.

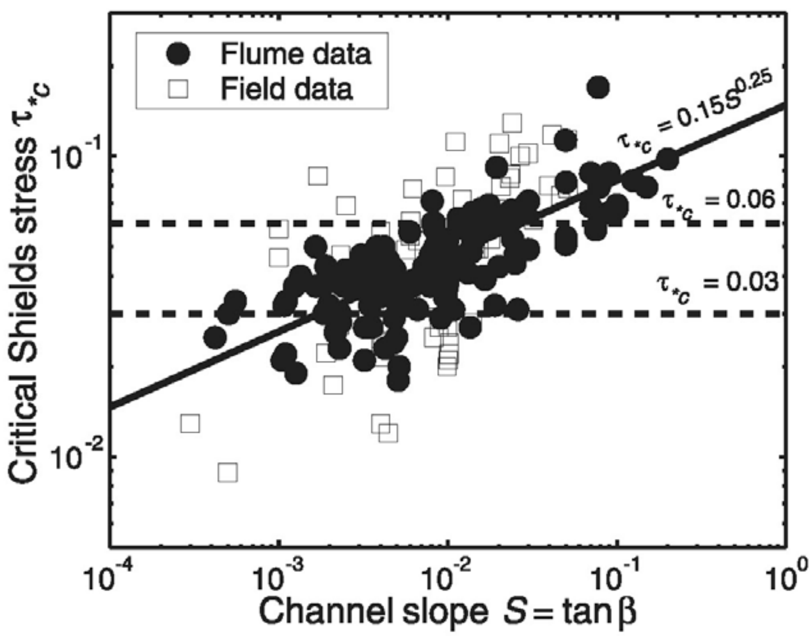

Figure G.2-2. Critical Shields stress versus channel slope (after Lamb et al. 2008).

Similar relations using D_{50} as the representative grain diameter have been developed by Recking (2009), Mueller et al. (2005), and Bunte et al. (2013) as illustrated by Equations G-5, G-6 and G-7, respectively. The Mueller et al. (2005) relation is also shown in Figure G.2-3.

$$\theta_{c50} = 1.32S + 0.037 \qquad \text{[Eq. G-5]}$$

$$\theta_{c50} = 2.18S + 0.021 \qquad \text{[Eq. G-6]}$$

$$\theta_{c50} = 1.74S + 0.037 \qquad \text{[Eq. G-7]}$$

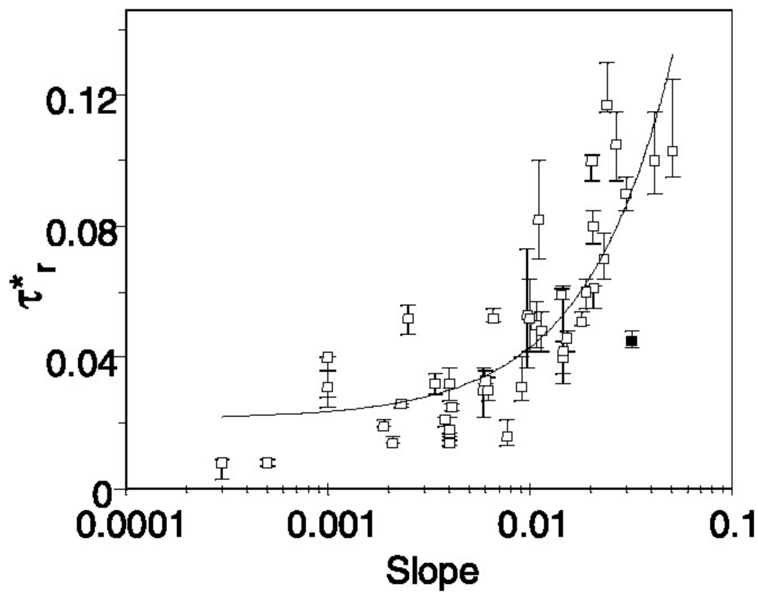

Figure G.2-3. Critical Shields stress versus channel slope (after Mueller et al. 2005).

Bunte et al. (2013) also provide a similar relation for mobilization of the D_{84}, as full bed mobilization is not always initiated by mobilization of the D_{50}.

$$\theta_{c84} = 0.71S + 0.021 \quad \text{[Eq. G-8]}$$

Critical Shear Stress and Bed Structure

Figure G.2-1 and Equations G-5 to G-7 demonstrate that while critical Shields stress and channel gradient have a well-defined relation, the various studies show scatter in their individual relations. Table G.2-1 shows the scatter in Θ_{c50} values associated with the various equations using channel gradients of 10% and 20%. This inter-study variability can not only be attributed to stream conditions (structural bed stability, bed material size composition, sediment supply, flow hydraulics and stream morphology), but also to differences in methodology applied by various researchers (Bunte et al. 2013).

Table G.2-1. Shields critical shear stress values using various equations.

Equation	Shields critical shear stress (Θ_{c50})	
	Slope = 10%	Slope = 20%
Lamb et al. (2008)	0.08	0.10
Recking (2009)	0.17	0.30
Mueller et al. (2005)	0.24	0.46
Bunte et al. (2013)	0.21	0.39

Indicators of low bed stability in steep coarse-bedded streams include an abundance of active gravel bars with particle sizes finer than the thalweg bed material, a high percentage of surface and subsurface sand and pea gravel, and a higher proportion of large particles that lie fully exposed on top of the bed (Bunte et al. 2013). High stability is inferred by the presence of algae and moss cover, particles that are stuck deeply in the gravel/cobble bed, imbrication and stone structures. Streams with average bed stability transport their subsurface D_{50} at bankfull flow; highly mobile streams transport their surface D_{84} at bankfull flow; and very stable streams transport their D_{16} sizes at bankfull flow (Bunte et al. 2013). Debris-flood prone creeks on the North Shore tend to be characterized by high bed stability, except for immediately following a debris-flood event when the bed is highly disturbed (e.g., Kilmer Creek). The choice of an appropriate relation for this study was informed by calibrating to the November 2014 event (see Section D.2).

G.2.2. Step 2 – Calculate Critical Discharge

Once a critical shear stress relation has been selected, Equation G-3 can be re-arranged to calculate the required hydraulic radius[3], R, for mobilization of the channel substrate:

$$R = \frac{1.65\emptyset_{c50}D_{50}}{S} \qquad \text{[Eq. G-9]}$$

The critical discharge can then be calculated using Manning's equation for a given cross-section. For Manning's equation, the methodology of Jarrett (1984) has been adopted to estimate Manning's n (see Equation A-1, Appendix A).

G.2.3. Step 3 – Exceedance of Critical Discharge

In a complementary study, Northwest Hydraulic Consultants (NHC) has been tasked with evaluating the adequacy and effectiveness of the existing drainage system in the DNV including pipe, culvert and channel capacities. An output of the work is a drainage model, which evaluates peak flows at all culverts for daily and sub-daily events under various return periods: 2-year, 10- year, 100-year and 200-year. Peak flow predictions under potential climate change conditions in 2030 and 2100 are a further deliverable of the work but have not been received at the time of this draft report

As a result, various return period hydrographs at the forested/development interface will be developed for a majority of the creeks being investigated by BGC. Because the NHC study is ongoing and preliminary results are not yet available, BGC has initially assumed that the 200-year unit peak instantaneous flow for all forested, undeveloped areas is 7.4 $m^3/s/km^2$. This value is a product of detailed rainfall-runoff modelling conducted by KWL (2014) as part of floodplain modelling conducted for Seymour River, Lynn Creek, Mosquito Creek and Mackay Creek. The KWL calibration was based on peak flow data from a hydrometric station on Mackay Creek

[3] The hydraulic radius is the ratio of the wetted cross-sectional area to the wetted perimeter.

(*Mackay Creek at Montroyal Boulevard*) operated by the Water Survey of Canada (WSC) since 1970. HEC-HMS was used by KWL for the rainfall-runoff modelling.

By extension, unit peak flows of 4.2 $m^3/s/km^2$ and 5.3 $m^3/s/km^2$ have been adopted by BGC for return periods of 20 and 50 years. Using these unit peak flows and employing HEC-HMS, BGC has generated interim hydrographs for the various creeks for the various return period classes (10 – 30 years, 30 – 100 years, and 100 – 300 years).

G.2.4. Step 4 – Sediment Transport

In recent years, research has focused on sediment transport in mountain streams and several models have been developed. Model examples include Topkapi ETH (Konz et al. 2011), TomSed (Chiari et al. 2010), SEDROUT (Hoey and Ferguson, 1994) and sedFlow (Heimann et al., 2015a). Sediment transport equations used by these models include those of Rickenmann (2001), Wilcock and Crowe (2003), Recking (2010), and Parker (1990). For this study, the equation of Rickenmann (2001) has been used, as it has been shown to be a reasonable predictor of sediment transport volumes for steep mountain streams in Switzerland (Nitsche et al. 2011, Heimann, et al. 2015b).

For steep slopes, i.e., 3% ≤ S ≤ 20%, the Rickenmann (2001) bedload transport rate q_b, is defined as:

$$q_b = 12.6 \frac{D_{90}}{D_{30}}^{0.2} \cdot (q - q_c) \cdot S^{2.0} \cdot (s-1)^{-1.6} \qquad \text{[Eq. G-10]}$$

where q_b is the bedload transport rate per unit channel width ($m^3/s/m$), q is unit discharge ($m^3/s/m$), q_c is the critical unit discharge at initiation of bedload transport, and s is the ratio of solid to fluid density. For simplification, setting $(D_{90}/D_{30})^{0.2}$ = 1.05 and s = 2.68 yields:

$$q_b = 5.8(q - q_c) \cdot S^{2.0} \qquad \text{[Eq. G-11]}$$

Equation G-11 is based on over 252 flume laboratory experiments. Observations on bedload transport in steep experimental streams are considered as a reference condition, which defines maximum transport rates ("transport capacity") for the idealized case of a uniform bed material, no morphological features, and hence no significant form roughness effects. Rickenmann (2001) then compared this empirical formula with bedload transport data from 19 mountain streams. This comparison showed that most of the smaller and steeper streams tended to have a lower bedload transport efficiency than larger streams. Rickenmann attributed this reduction in transport efficiency to an increase in flow resistance, as all the lower efficiency streams are grouped within the range of relative flow depths[4] smaller than 4 to 6. However, he also noted that lower efficiencies may be related to having flows near critical conditions for the beginning of sediment transport, which prevailed for many events analyzed in his study (i.e., only partial sediment transport occurred and full bed mobilization did not occur).

[4] Relative flow depth is defined as h/D_{90}, where h is flow depth and D_{90} is the grain size for which 90% of the surface bed material is finer by weight.

Given this variance from idealized conditions, Rickenmann (2001) provides the following alternative equation for bedload transport:

$$G_E = AS^{2.0}V_{re} \quad \text{[Eq. G-12]}$$

Where G_E is the total bedload volume per flood events and the effective runoff volume, V_{re}, is the integral of the discharge above the critical discharge at initiation of bedload motion (Q-Q_c). The parameter A represents bedload efficiency, which is defined by the deviation of observed transport rates from those predicted by Equation G-11.

G.2.5. Step 5 – Sediment Transport Volumes

Using the hydrographs developed in Step 3 and Equation G-12, sediment volumes for various return periods hydrographs can be determined.

G.3. CALIBRATION

G.3.1. Kilmer Creek

The November 3, 2014 debris flood on Kilmer Creek provided an opportunity to validate the proposed methodology outlined in the previous section. That debris flood transported a minimum of 300 m^3 of sediment to the Kilmer Diversion. Additional sediment may have been transported downstream of the diversion.

As a first step in the analysis of the November 2014 debris flood, a hydrograph of the flood event was developed using a HEC-HMS model developed by BGC. The model was initially "calibrated" to produce a unit peak flow of 7.4 $m^3/s/km^2$ for undeveloped, forested terrain and a 200-year rainfall event. Once calibrated, the input hyetograph to the model was the November 3, 2014 rainfall data collected at the Hastings rain gauge, which is operated by the DNV (Figure A.2-1, Appendix A). This rain gauge is located at an approximate elevation of 350 m, compared to a maximum elevation of about 1005 m in the Kilmer Creek watershed and will thus proportionally underestimate rainfall amounts at higher elevation due to orographic uplift effects.

In their calibration to Mackay Creek flow data, KWL (2014) used precipitation data from Metro Vancouver's Cleveland Dam climate station (DN82). This station has been active since 1962 and is located at an elevation of about 156 m. During the calibration process, KWL determined that an orographic rainfall factor of 2.08 should be applied to the DN82 rainfall for the forested sub-catchment of Mackay Creek. Rainfall volumes at the Hastings Creek rain gauge for the November 3, 2014 storm was about 65% greater than that recorded at lower elevations (see Table A.2-1, Appendix A). Therefore, the Hastings rainfall was adjusted upwards by 25% by BGC to provide an orographic factor of about 1.9 compared to lower elevations. This adjustment factor is slightly lower than that applied by KWL, but the mean elevation of the Kilmer Creek watershed is lower than the Mackay Creek watershed.

The resulting November 2014 hydrograph is shown in Figure G.3-1 for Kilmer Creek at Dempsey Road. The peak instantaneous flow estimate based using the HEC-HMS model and a watershed area of 0.77 km^2 is 4.6 m^3/s. This result is consistent with a peak flow estimate of 4.4 m^3/s derived from high water mark observations (Section A.4.5, Appendix A).

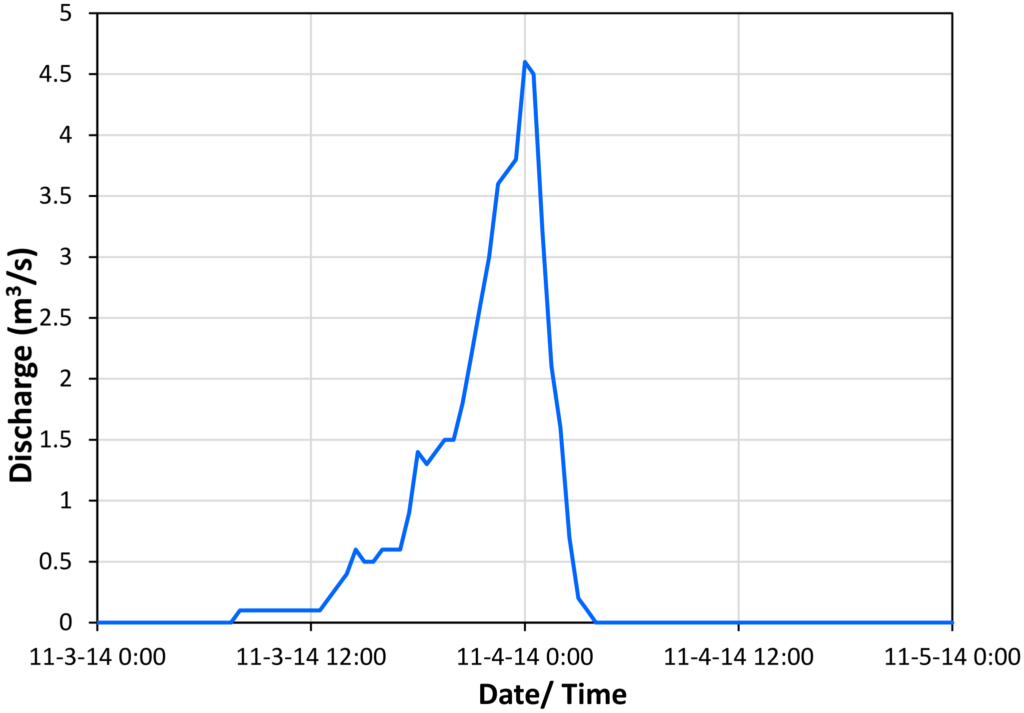

Figure G.3-1. November 3, 2014 hydrograph for Kilmer Creek at Dempsey Road.

In comparison, the 20-year, 50-year and 200-year return period peak instantaneous flows for Kilmer Creek at Dempsey Road are estimated at 3.3, 4.2 and 5.7 m^3/s. The November 3, 2014 event is associated with a return period of about 60 years. BGC is not aware of a major debris flood having initiated on Kilmer Creek since at least the 1960s, when the Kilmer Creek diversion was constructed. The critical discharge for debris flood initiation on Kilmer Creek is between 3.3 m^3/s and 4.2 m^3/s. For about a 200 m section of Kilmer Creek above Dempsey Road, the creek has a width of about 4.2 m and an average channel gradient of 18%. Using an idealized channel cross-section and Manning's equation, hydraulic radii of 0.57 m and 0.63 m, respectively, are back-calculated for peak flows of 3.3 m^3/s and 4.2 m^3/s.

Using a field-estimated D_{50} of 120 mm and a D_{84} of 300 mm, application of Equations G-5 to D-9 results in the following critical values:

Table G.3-1. Critical values for Shields stress, hydraulic radius and discharge using various relations.

Equation	Critical Shields Stress (Θ_c)	Critical R (m)	Critical Q (m^3/s)
Bunte et al. (2013), D_{50}	0.35	0.39	2.2
Bunte et al. (2013), D_{84}	0.15	0.46	3.1
Recking (2009)	0.27	0.30	1.3
Mueller et al. (2005)	0.41	0.45	3.1

Table G.3-1 indicates that Equations G-5 to G-8 predict that a critical threshold for sediment transport is reached at a discharge less than a 20-year return period. For full bed mobilization, these predictions are considered to be an over-estimate, as bedload removal at the Kilmer Diversion is not a regular maintenance issue for DNV maintenance crew. This result is expected given that Kilmer Creek is considerably steeper than the studies used by Bunte et al. (2013), Recking (2009), and Mueller et al. (2005) to generate the critical Shields stress versus channel slope relations. None of the reference creeks used by the authors has a channel gradient in excess of 10% (see Figure G.2-1 to Figure G.2-3). Also large boulders that are eroded from glacial till (i.e., lag deposits) as well as woody debris dams on Kilmer Creek provide considerably more resistance to sediment mobilization in comparison to the reference creeks. As a result, sediment transport is a relatively rare event on Kilmer Creek. The importance of bed stability has been noted by Bunte et al. (2013) who postulated that highly mobile streams transport their surface D_{84} at bankfull flow while very stable streams transport their D_{16} sizes at bankfull flow.

To account for this additional bed stability at Kilmer Creek, the critical Shields stress (θ_{c84}) calculated using Equation G-8 was multiplied by an adjustment factor of 1.1. The critical hydraulic radius and discharge for full bed mobilization is then estimated at 0.48 m and 3.5 m^3/s, placing the November 3, 2014 debris flood on Kilmer Creek between a 20 and 50-year return period event. Using D_{84} in these calculations is preferred over the D_{50} in that the D_{84} is an easier metric to measure in the field.

Using a critical discharge of 3.5 m^3/s, the hydrograph of Figure G.3-1, and Equation G-11, the volume of sediment transported past the Dempsey Road culvert on Kilmer Creek is estimated at 350 m^3 for the November 2014 flood event, which compares well with the minimum estimated volume of 300 m^3.

G.4. PREDICTIONS

As a final note, care must be taken in using Equation G-12 for predicting sediment volumes during debris flood events. Equation G-12 assumes that there is an unlimited supply of sediment for transport during a flood event. In reality, all of the debris-flood prone creeks assessed in this study are supply-limited. Most of the creeks have less than 0.5 to 1 m of sediment on the channel bottom (on average) below which a dense basal till is encountered. Variations in channel gradients also result in short reaches where sediment is preferably deposited, further constraining the reach length over which sediment can be mobilized and impact urbanized areas.

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APPENDIX L
CULVERT OVERFLOW ASSESSMENT METHODOLOGY

L.1. CULVERT OVERFLOW ASSESSMENT METHODOLOGY

BGC used the following parameters, individually and in combination, to systematically identify culverts and stormwater mains (collectively referred to as "culverts") susceptible to overtopping at a certain flow return period:

1. Overtopping of culverts due to exceedance of culvert flow capacity.
2. Blockage due to accumulation of debris at a "flat culvert inlet" as defined in section L.1.2.
3. Blockage due to accumulation of debris in a "flat culvert" as defined in section L.1.3.
4. Blockage due to boulders.
5. Blockage due to large woody debris (LWD).
6. Blockage due to undersized trash rack screen area.
7. Effectiveness of debris control structures (posts).

Each parameter is considered for the 20, 50, and 200-year return period events.

L.1.1. Overtopping due to exceedance of culvert capacity

When a culvert is undersized to pass the clear-water flood discharge, a portion of the flow spills out of the channel. In cases where culvert capacity is exceeded, the culvert may still remain functional. That is, the culvert will pass a flow rate up to its capacity, and the portion of flow exceeding that capacity will overtop the culvert.

L.1.2. Blockage due to accumulation of debris at a flat culvert inlet

Abrupt decreases in the channel slope (flat areas), especially if associated with channel widening, encourage sediment deposition. When these low gradient areas are located immediately upstream of the culvert inlet, the deposited sediment can block the culvert inlet over time or in a single event. Table L.1-1 shows the criteria used by BGC to identify culverts susceptible to blockage due to sediment or organic material. The gradient "immediately upstream"[1] of the culvert intake is used to assess whether bed material transported in a debris flood will begin to deposit upstream of or be transported through the culvert. This matrix has been calibrated by known events. An example of this calibration is shown in Table L.1-2.

[1] Gradient "immediately upstream" is defined as the average gradient between the culvert and a point upstream at a distance equal to 3 times the channel width

Table L.1-1.Multi-criteria matrix to identify culverts susceptible to blockage due to flat inlet.[2]

		Culvert Capacity	
		Flow > 50% capacity	**Flow < 50% capacity**
Gradient immediately upstream of culvert[3]	< 1/2 avg. effective u/s gradient[4]	*Blocks*	*Blocks*
	1/2 to 2/3 effective u/s gradient[4]	*Blocks*	*Does not block*
	>2/3 effective u/s gradient[4]	*Does not block*	*Does not block*

L.1.3. Blockage due to accumulation of debris in a flat culvert

Abrupt decreases in the channel slope encourage sediment deposition. As with low gradient culvert inlets, when the culvert itself has a shallow gradient, deposited sediment can block the culvert. A culvert with a gradient less than half the average upstream channel gradient is expected to block due to accumulation of debris. A culvert having a gradient between half and two-thirds the average upstream channel gradient is expected to block when the flow rate is greater than 50% the culvert capacity and debris is expected to be mobilized.

L.1.4. Blockage due to boulders

When boulders are mobilized and deposited in front of or within a culvert, the culvert may become blocked. If the effective grain size (maximum size of sediment expected to be mobilized in a flood, debris flood, or debris flow) is greater than half of the culvert diameter, the culvert is expected to block. The logic is that such boulder accumulation would lead to a positive feedback whereby other sediment is jammed into the larger clasts and a boulder dam or sediment wedge will form upstream, with a lower gradient than the channel itself. This sediment wedge will encourage more deposition; hence, further increasing the likelihood for culvert blockage.

L.1.5. Blockage due to large woody debris

Large woody debris such as root wads, portions of trees or entire trees, as well as large branches, can also accumulate at the culvert intake. This creates a flow obstruction that promotes additional blockage by organic and mineral debris before entering the culvert. As more material accumulates and restricts flow a sediment wedge will form upstream of the culvert inlet, as discussed in section L.1.4, further increasing the likelihood for culvert blockage. If large volumes of organic materials (trees/root wads/branches) are expected, and a debris control structure is not present, the culvert is expected to block. If a debris control structure or trash rack is present, its effectiveness at preventing blockage due to large woody debris is considered (see sections L.1.6 and L.1.7).

[2] Evaluated upstream of culvert over a distance equal to 3 times the channel width.
[3] Gradient immediately upstream is defined as the average gradient between the culvert and a point upstream at a distance equal to 3 times the channel width.
[4] Effective upstream gradient is defined as the average upstream channel gradient.

L.1.6. Blockage due to undersized trash rack screen area

Trash racks promote blockage of organic debris and sediment in front of a culvert. DNV staff report that many of the blockages during the November 2014 event were a result of blocked trash racks. Where the screening area of the trash rack is less than 10 times the cross-sectional area of the culvert (Bradley et al. 2005), we have assumed the trash rack will block during flood events.

L.1.7. Debris control structures (steel posts)

Debris control structures, such as steel posts, are located upstream of many culverts in the DNV, and are designed to prevent debris from reaching the culvert. The effectiveness of a debris control structure (DCS) is determined systematically by evaluating the height of the posts relative to the height of the culvert and the depth of the channel, the spacing between posts relative to the size of debris expected, and the width of the set of posts relative to the channel width. The distance between the culvert and the DCS, and the storage area upstream of the DCS are also considered in determining the effectiveness of a debris control structure; the larger the storage area, the more effective the DCS. If a DCS is determined to be effective the culvert may be categorized as *does not block (due to LWD or boulders).*

L.1.8. Application of culvert overflow assessment

Table L.1-2 shows the application of the overflow and debris blockage criteria to the November 2014 event on Thames Creek, which illustrates the calibration of the methodology. This culvert overflow assessment methodology was applied to all creeks, and results are included in Appendix O.

In Table L.1-2. the methodology correctly predicts the November 2014 culvert blockages at all assets. As outlined in Appendix A, the damage at Kilmer Road occurred due to the overtopping of the trash rack, rather than the blockage of the culvert.

A limitation of this analysis is that it classifies culvert overtopping as simply occurring or not occurring. In reality, a range in possibilities exist due to factors that cannot be fully predicted, such as blockage by woody debris. However, this simplification limits the number of possible hazard scenarios to a level that can be feasibly modelled and assessed. BGC assigned relatively conservative estimates to account for this uncertainty. Since climate change is likely going to increase runoff and thus sediment transport (see section 4.3 of the main report), such conservativism appears warranted.

Table L.1-2. Application of debris blockage criteria to Thames Creek.

Culvert Location	Gradient (%)			200-Year Flow (m^3/s)	Culvert Capacity (m^3/s)	Debris Control Structure	November 2014 event response	Matrix predicted response
	Avg. upstream	Immediately Upstream	Culvert					
Mountain Highway	29	33	14	3.9	4.3	No[5]	Partially blocked	Blocks
McNair Dr	14	4	7	5.2	3.9	No[5]	Partially blocked	Blocks
Valley Rd	19	11	3	5.4	14.8	No	Did not block	Does not block
Coleman St	7	9	12	6.1	7.4	Yes	Did not block	Does not block
Dempsey Rd	9	11	4	7.6	4.0	No	Partially blocked	Blocks
Kilmer Rd	6	7	N/A[6]	7.8	9.2	Yes	Trash rack blocked	Special considerat-ion of trash rack required

[5] ISL Engineering mitigation design implemented October of 2015.
[6] Enters Kilmer Diversion.

APPENDIX M
DEBRIS FLOW AND DEBRIS FLOOD MODELLING AND HAZARD INTENSITY MAPPING

M.1. INTRODUCTION

BGC modelled hydrogeomorphic processes numerically to estimate the extent and intensity of inundation associated with debris flow and debris flood hazard scenarios on a number of DNV creeks. The model outputs are used to develop interpreted hazard intensity maps, which in turn form the basis for the risk assessment and prioritization of creeks for mitigation.

This appendix describes the modelling methods and input parameters that were used to simulate debris floods on urban creeks and debris flows on Percy Creek. The interpreted hazard intensity maps are included in Drawings M-1 to M-40 at the end of this appendix. Also included are hazard zone maps for Indian Arm creeks delineated in the North Vancouver debris flow and debris flood quantitative risk assessment update (BGC 2009). No new modelling was completed for Scott-Goldie Creek or the Indian Arm creeks north of Percy Creek. Raw model results are available upon request.

Debris flood and flood modelling is first discussed, followed by a description of debris-flow modelling conducted for Percy Creek.

M.2. FLOOD AND DEBRIS FLOOD MODELLING

M.2.1. Introduction

Numerical modelling of debris floods on urban creeks within the DNV provided the basis for the estimation of spatial impact probabilities and corresponding debris-flood intensities, which serve as inputs to the quantitative risk assessment (QRA) described in Appendix N. This section describes the approach to modelling debris floods and model inputs, the selection of creeks to model, and model results.

M.2.2. Methodology and Input

FLO-2D, a U.S. Federal Emergency Management Agency (FEMA) approved two-dimensional hydraulic model, was used to model flood and debris-flood events. FLO-2D was selected because of its ability to model flood wave propagation processes and overland flow.

The minimum inputs required for FLO-2D to simulate a hydrogeomorphic event include a hydrograph at the point of flow initiation, a Digital Elevation Model (DEM), and the spatial distribution of Manning's n numbers that reflect the roughness of surface cover for potential inundation areas. For hydraulic structures, such as culverts, bridges and storm mains, the inlet and outlet locations, dimensions, and flow capacity are input to the model. The input parameters used are summarized in Table M.2-1.

Table M.2-1. Summary of input parameters for FLO-2D debris-flood modelling.

Input	Description
Inflow hydrograph	Inflow hydrographs for each scenario were generated using HEC-HMS models. These include hydrographs for creeks where modelling was considered, but assessed not to be required, including Thain Creek, Hastings Creek, Dyer Creek, Cove Creek, Martin Creek, Francis Creek and Ward Creek. The methodology used to develop peak flow estimates and hydrographs for this assessment is outlined in Appendix E[1].
DEM of modelling domain	LiDAR flown in spring and summer of 2013[2] was used to generate topographic inputs. The LiDAR data available cover an area of approximately 200 km^2 and extend from Capilano Lake in the west to Indian Arm with a resolution of approximately 1 m. FLO-2D's pre-processing program, GDS, was used to create a 2 m topographic grid. The model domain for each individual scenario was selected based on the anticipated area of interest and adjusted if shown to be too small.
Building effects	The ground surface elevations inside the footprint of the houses were artificially elevated for three meters to take into account the obstacle effect of the buildings along the flow path.
Surface roughness	The floodplain roughness in the model domain was estimated based on a land use map provided by DNV[3] that includes roads, residential areas, site photos, and aerial photographs. Roads were simulated in FLO-2D as shallow rectangular channels with a smooth bed. The Manning's n values assigned for the various land uses are shown in Table M.2-2. Buildings were accounted for by adding them to the topography based on the building footprint locations.
Hydraulic structures	Culverts and storm mains were entered into the model domain by inputting the inlet and outlet locations, culvert dimensions, and flow capacity. Undersized culverts, where overland flow is expected due to insufficient capacity, were entered into the model with the true dimensions. In contrast, where a culvert is expected to block due to sedimentation, large woody debris or material build up on the trash rack, the blockage was simulated by using the original ground elevation and assigning zero discharge capacity to the culvert.
Simulation time	A 24-hour hydrograph was input to FLO-2D for all debris flood and flood modelling scenarios.

[1] Note that this Appendix is an interim analysis that will be superseded by nhc results.
[2] The LiDAR was flown in three missions: 1) Urban area on April 23, 2013 from approximately 5:15pm to 8:00 pm PST; 2) Alpine area on July 2, 2013 from 3:05 pm to 6:40 pm PST; and, 3) Re-flight data from July 6, 2013 between approximately 11:35 am to 12:45 pm PST (K.Wong, email, February 18, 2016).
[3] The land use map was provided as part of the geodatabase package DNV provided to BGC in June 2015.

Table M.2-2. Manning's n values based on Chow (1959).

Land Cover	Manning's n
Forest	0.1
Creeks	0.04
Existing Residential Areas	0.045
Streets	0.016

Debris floods within urban areas of the DNV are estimated to have sediment concentrations of about 3 to 10%, though this has never been measured directly. To simulate the rheology of geomorphic events in FLO-2D, such as hyperconcentrated flows or debris flows, a minimum volumetric sediment concentration of 20% is required. Therefore, all debris-flood events were simulated as clear water. This assumption is reasonable in that the debris floods behave similar to clear water flood.

M.2.3. Hazard Scenarios

The criteria used to select which of the DNV creeks would require modelling were as follows:

1. A debris-flood event with a 200-year or lower return period is likely to lead to overland flow outside of the creek channel upstream of development or as a result of culvert blockage (Appendix L) within the developed area.
2. A flood event with a 200-year or lower return period is likely to result in full or partial culvert blockage (Appendix L) within the developed area.
3. Potential overland flooding is sufficiently complex, such that modelling is required. In some cases, the topography constrains the potential extent of overland flooding such that the potential inundation area could be delineated from existing LiDAR data and judgement. Sensitivity modelling by BGC indicated that, for the magnitude of debris floods considered in this study, overland flows are generally shallow (< 0.1 m). Therefore, debris-flood intensity for select cases can be approximated with confidence in the absence of detailed modelling.

According to the criteria above, several study creeks did not require modelling. These are listed with the associated rationale for not completing modelling in Table M.2-3.

For creeks where modelling was required, representative hazard scenarios were developed that consider the culvert blockage rating. This rating identifies the most frequent return period where blockage can be expected along with the associated reason for blockage (Appendix L), and the likely combinations of culverts on a given creek which could block during the same event. All model scenarios are outlined in Table M.2-4. These scenarios formed the basis for the risk assessment (Appendix N). Creeks where no representative hazard scenarios were identified and modelled or manually mapped were not included in the risk assessment.

Table M.2-3. Outline of creeks where modelling was not completed as part of this assignment and associated rationale.

Creek	Process	Rationale
Mosquito	Debris flood	Modelling was completed as part of 2013 assessment (BGC, 2013a).
Thain	Debris flood	Upslope of development, Thain Creek consists of several tributaries that coalesce in the vicinity of St. Albans Park. The western tributaries are generally weakly incised and not prone to debris floods. Minor debris flood activity could occur in the east tributary, but there is sufficient storage capacity where Thain Creek first reaches development at Prospect Road. Here, there are two circular concrete culverts at the base of the creek and a larger overflow arch culvert that is set at an elevation that is 1 to 1.5 m higher than the crown of the concrete culverts.  **Figure M.2-1. Thain Creek at Prospect Road, STMCUL00274 (photo taken by BGC, August 2015).**
Hastings	Debris flood	Immediately upstream of culvert at E Braemar Rd, there is a localized sedimentation area. While the culvert at E Braemer Rd could still become blocked, the topography is such that any overland flows would return to the creek corridor immediately below the road. Only one house has been identified at risk at this location. The extents of potential inundation were assessed based on field observations and manually mapped.
Dyer	Debris flood	Dyer Creek becomes relatively unconfined upstream of E Braemer Rd, providing a broad area for sediment deposition. Thus, the culvert at E Braemer Rd is unlikely to be blocked by sediment or LWD. In the unlikely event of a blockage, the topography is such that any overland flows would return to the creek corridor immediately below the road. The only infrastructure at risk for this unlikely occurrence is the road itself, which could be damaged by overland flows. The extents of potential inundation were assessed based on field observations and manually mapped.
Coleman	Flood	Coleman Creek has a very small watershed upstream of development and is not prone to debris floods. Within the developed area, no geomorphic activity has been identified that could impact channel hydraulics.
Canyon	Flood	The channel is confined to a valley upstream and downstream of Hyannis Drive. Any overflow of the culvert would result in overland flow returning to the channel due to the local topography of the road. Due to a low channel gradient, there is also no evidence that Canyon Creek is prone to debris floods.

Creek	Process	Rationale
McCartney	Flood	The watershed discharges into McCartney Creek Park and is well confined by valley walls. The projected flow would not be sufficient to transport large woody debris of sufficient size to block the culvert completely. Moreover, there is little to no infrastructure susceptible to damage downstream of the creek.
Kai	Flood	Kai Creek has a very small watershed area and does not experience sufficient flow to generate a credible hazard to downstream infrastructure. The creek is poorly defined upstream of development.
Cove	Flood	The channel is stabilized by boulder steps such that only partial sediment transport is likely below the 200-year return period.
Martin	Flood	Upstream of Indian River Drive, there is evidence of an historical avulsion path from Francis Creek into Martin Creek. The potential for future avulsions of Francis Creek into Martin was assessed during this investigation. The most probable avulsion point is at an outside meander bend where the channel is 7 m wide and the bank is 1.7 m high (49.341132, -122.943188). Based on the channel cross section at this location, the amount of flow required to overtop the channel is approximately 35 m^3/s. The peak flow estimates indicate that a 200 year storm event would result in peak flows of approximately 11 m^3/s. As such, avulsion of Francis Creek into Martin Creek is only anticipated to occur during storm events significantly exceeding the return periods investigated in this study. This does not preclude the possibility of progressive aggradation in the channel reach reducing the available freeboard in the channel. Regular inspections of the avulsion location to ensure no future changes are recommended.
Francis	Debris flood	Francis Creek discharges into the Seycove Marina lot at the far east end of Panorama Drive. The only building susceptible to damage is the Deep Cove North Shore Marina Building. The extents of potential inundation were assessed based on field observations and manually mapped.
Unnamed	Flood	Unnamed Creek is located west of Ward Creek. It was not included in the DNV's creeks and was identified during field investigations. The buildings susceptible to damage were also identified based on field observations along with LiDAR data thus the extents of potential inundation were manually mapped.
Sunshine	Debris flood	Sunshine Creek lies within a well-defined valley; any overland flow resulting from culvert blockage at Sasamat Lane or Sunshine Falls Lane would return to the main channel due to the local topography of the area. Moreover, the Sasamat Lane culvert has significant sediment storage capacity upstream of the culvert due to the culvert being recessed several meters below the road. Note that debris flows from Scott-Goldie creek may have the potential to avulse into the Sunshine Creek watershed (see Appendix K). Further debris flow hazard analysis and scenario modelling on Scott-Goldie creek would be required to quantify this hazard and associated risk.

Table M.2-4. Summary of modelled scenarios and key inputs.

Creek	Scenario	Run(s)	Initiation Point		Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width	Height	Capacity	Return Period	Notes
			Latitude	Longitude						(mm)	(mm)	(m^3/s)	(years)	
Mackay Creek	1	1	49.37190	-123.09615	Mackay West Avulsion Scenario (50 year)	Grouse Grind Trail	STMCUL00248	Blocked	Round	900		1.0	30-100	West avulsion scenario possible only as a result of a Mackay Creek debris flow, assuming the full debris flow avulses to the west and the flow is the afterflow. The return period reflects the most frequent debris flow return period as outlined in BGC's 2013 assessment (BGC, 2013b).
						Parking lot	STMCUL00249	Blocked	Round	1200		2.1		
						Grousewoods Dr	STMCUL00361	Blocked	Box	1800	1300	3.5		
						Parking lot ditch	STMCUL00362	Clear	Round	900		1.0		
						Grousewoods Dr	STMCUL00528	Clear	Box	2400	1500	6.0		
						Blue Grouse Way	STMCUL00527	Clear	Box	2400	1500	6.0		
						Cliffridge Ave	STMMN00392	Clear	Round	1350		3.5		
						Sonora Dr	STMCUL00533	Clear	Round [5]	1200		4.0		
Mackay Creek	2	2	49.37190	-123.09615	Mackay West Avulsion Scenario (100 year)	Grouse Grind Trail	STMCUL00248	Blocked	Round	900		1.0	100-300	West avulsion scenario possible only as a result of a Mackay Creek debris flow, assuming the full debris flow avulses to the west and the flow is the afterflow.
						Parking Lot	STMCUL00249	Blocked	Round	1200		2.1		
						Grousewoods Dr	STMCUL00361	Blocked	Box	1800	1300	3.5		
						Parking lot ditch	STMCUL00362	Clear	Round	900		1.0		
						Grousewoods Dr	STMCUL00528	Clear	Box	2400	1500	6.0		
						Blue Grouse Way	STMCUL00527	Clear	Box	2400	1500	6.0		
						Cliffridge Ave	STMMN00392	Clear	Round	1350		3.5		
						Sonora Dr	STMCUL00533	Clear	Round [5]	1200		4.0		
Mackay Creek	3	3	49.36811	-123.09527	Mackay East Scenario (50 year)	Grouse Mountain Overflow	STMCUL00364	Blocked	Round	1200		2.2	30-100	Assumes the full debris flow reports to the base of the Mackay watershed.
						Grousewoods Dr	STMMN00192 [STMMN00190, STMMN00189, STMMN00191, STMMN00212, STMMN09316]	Blocked	Round	600		0.7		
						Grousewoods Dr	STMCUL00622	Blocked	Round	900		1.9		
						Cliffridge Ave	STMMN00365	Blocked	Round	1200		3.8		
						Sonora Dr	STMCUL00533	Clear	Round [5]	1200		4.0		

[4] Where multiple culverts or storm mains are joined into one hydraulic structure, the most upstream asset is listed with the downstream assets in square brackets.
[5] Double barreled culvert, each 1200 mm in diameter.

Creek	Scenario	Run(s)	Initiation Point		Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width	Height	Capacity	Return Period	Notes
			Latitude	Longitude						(mm)	(mm)	(m^3/s)	(years)	
Mackay Creek	4	4	49.36811	-123.09527	Mackay East Scenario (200 year)	Grouse Overflow	STMCUL00364	Blocked	Round	1200		2.2	100-300	Assumes the full debris flow reports to the base of the Mackay watershed.
						Grousewoods Dr	STMMN00192 [STMMN00190, STMMN00189, STMMN00191, STMMN00212, STMMN09316]	Blocked	Round	600		0.7		
						Grousewoods Dr	STMCUL00622	Blocked	Round	900		1.9		
						Cliffridge Ave	STMMN00365	Blocked	Round	1200		3.8		
						Sonora Dr	STMCUL00533	Clear	Round [5]	1200		4.0		
Mosquito Creek	5	1	49.362	-123.072	-	-	-		-	-	-	-	200	Interpreted hazard intensity zones delineated based on model results from BGC (2013b).
Mosquito Creek	6	2	49.365	-123.071	-	-	-		-	-	-	-	5000	
Mosquito Creek	7	3	49.365	-123.071	-	-	-		-	-	-	-	2500	
Mission Creek	8	1-4	49.35339	-123.07569	Debris Flood	Prospect Rd	STMCUL00266	Blocked	Round	1050		1.5	100-300	
						Beaver Rd	STMCUL00267	Blocked	Round	1350		1		
						Beaver Rd	STMCUL00268	Clear	Round	1050		0.6		
						Newdale Crt	STMCUL00269	Blocked	Round	1050		1.3		
						Monteray Ave	STMCUL00270	Blocked	Round	1200		4.0[6]		
							STMCUL00271	Blocked	Round	1200				
						Montroyal Blvd	STMCUL00607	Clear	Box	2000	1300	4.5		
Mission Creek	9	3	49.35339	-123.07569	Newdale Crt Blocked Trash Rack	Prospect Rd	STMCUL00266	Clear	Round	1050		1.5	10-30	Modelled at the 200 year return period in order to determine the maximum probably overland flood extents
						Beaver Rd	STMCUL00267	Clear	Round	1350		1		
						Beaver Rd	STMCUL00268	Clear	Round	1050		0.6		
						Newdale Crt	STMCUL00269	Blocked	Round	1050		1.3		
						Monteray Ave	STMCUL00270	Clear	Round	1200		4.0[7]		
							STMCUL00271	Clear	Round	1200				
						Montroyal Blvd	STMCUL00607	Clear	Box	2000	1300	4.5		

[6] The capacity is the combined capacity of STMCUL00270 and STMCUL00271 at Monteray Ave.
[7] The capacity is the combined capacity of STMCUL00270 and STMCUL00271 at Monteray Ave.

Creek	Scenario	Run(s)	Initiation Point		Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width	Height	Capacity	Return Period	Notes
			Latitude	Longitude						(mm)	(mm)	(m^3/s)	(years)	
Mission Creek 2	10	5	49.35108	-123.07379	Prospect Rd Storm Main	Prospect Rd	STMMN01726	Blocked	Round	525		0.3	100-300	
Mission Creek 3	11	6	49.35250	-123.07390	Prospect Rd Storm Main	Prospect Rd	STMCUL00273	Clear	Round	1050		1.4	100-300	
							STMMN09114	Blocked	Round	600	-	0.5		
Hastings Creek	12	-	49.34561	-123.06003	Braemar Rd Blockage	E Braemar Rd	STMCUL00393	Blocked	Round	1200		2.9	100-300	Interpreted hazard intensity zones delineated based on field observations with experience and judgement
Dyer Creek							STMCUL00395	Blocked	Round	1200		2.1	100-300	
Kilmer Creek	13	1	49.34646	-123.04756	November 2014 Storm Event	South of Michener Way	STMMN04251	Blocked	Round	750		4.7 [8]	Nov 2014 event	
							STMMN08659	Blocked	Round	1350				
						Wellington Dr	STMCUL00043	Clear	Box	2400		11.7		
						Fromme Rd	STMCUL00172	Blocked	Arch	1800	1100	3.0		
						Frederick Rd	STMCUL00175	Blocked	Round [9]	900		3.0		
Kilmer Creek	14	2	49.34644	-123.04753	Kilmer Diversion	South of Michener Way	STMMN04251	Blocked	Round	750		4.7 [8]	100-300	
							STMMN08659	Blocked	Round	1350				
						Wellington Dr	STMCUL00043	Clear	Box	2400		11.7		
						Fromme Rd	STMCUL00172	Clear	Arch	1800	1100	3.0		
						Frederick Rd	STMCUL00175	Clear	Round [10]	900		3.0		
Thames Creek	15	1	49.35623	-123.03716	Mountain Hwy Blockage	Mountain Hwy	STMCUL00052	Blocked	Box	1800	1250	4.3	100-300	
							BGCSTMCUL00074	Blocked	Round	800		0.8		
						Dempsey Rd	STMCUL00409	Clear	Round	1200		4.0		
						Kilmer Rd	STMCUL00412	Clear	Box	600				
							STMMN09158	Clear	Round	1800		9.2		
Thames Creek	16	2, 3	49.34657	-123.03283	McNair Rd and Kilmer Rd Blockage	McNair Rd	STMCUL00152	Blocked	Round	1200		3.9	30-100	
						Dempsey Rd	STMCUL00409	Clear	Round	1200		4.0		
						Kilmer Rd	STMCUL00412	Blocked	Box	600				
							STMMN09158	Blocked	Round	1800		9.2		

[8] The capacity is the combined capacity of STMMN04251 and STMMN08659 at the Kilmer Diversion south of Michener Way.
[9] Triple barreled culvert, capacity reported is the combined total capacity.
[10] Triple barreled culvert, capacity reported is the combined total capacity.

Creek	Scenario	Run(s)	Initiation Point Latitude	Initiation Point Longitude	Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width (mm)	Height (mm)	Capacity (m^3/s)	Return Period (years)	Notes
Taylor Creek	17	1	49.317523	-122.942178	Mt Seymour Pkwy Blockage	Mt Seymour Pkwy	STMCUL00259	Blocked	Round [11]	1050		4.2	100-300	
Gallant Creek	18	1	49.32641	-122.95731	Deep Cove Blockage	Badger Rd	STMCUL00217	Blocked	Round	900		2.0	10-30	
						Deep Cove Rd	STMCUL00580	Blocked	Round	1250		2.5		
Gallant Creek	19	2	49.33327	-122.96053	Indian River Dr Overland Flow	Indian River Dr	STMCUL00181	Clear	Box	4500	1300	10.4	100-300	
						Indian River Dr	STMCUL00180	Clear	Box	4500	1400	11.7		
						Indian River Dr	BGCSTMCUL00010	Clear	Round	1270		2.6		
						Badger Rd	STMCUL00217	Blocked	Round	900		2.0		
						Deep Cove Rd	STMCUL00580	Blocked	Round	1250		2.5		
						Gallant Ave	STMMN09116	Clear	Round	1200		2.0		
						Panorama Dr	STMCUL00222	Clear	Round	600		0.35		
						Panorama Dr	STMCUL00221	Clear	Round	600		0.35		
						Panorama Park	STMCUL00681	Clear	Round	600		0.35		
						Panorama Park	STMCUL00446	Clear	Round	750		0.40		
						Panorama Park	STMCUL00445	Clear	Round	600		0.35		
						Badger Rd	STMMN07161 [STMMN07162] [12]	Clear	Round	525		0.26		
						Caledonia Ave	STMMN07181 [STMMN07182, STMMN07183]	Clear	Round	600		0.35		
						Panorama Dr	STMMN09093 [STMMN07142] [13]	Clear	Round	525		0.26		
						Badger Rd	STMMN07163 [STMMN07164] [14]	Clear	Round	375		0.11		
						Caledonia Ave	STMMN07192 [STMMN07193, STMMN07194, STMMN07195] [15]	Clear	Round	200		0.06		

[11] Triple barreled culvert, capacity reported is the combined total capacity.
[12] Minimum dimension and capacity are based on the downstream asset STMMN07162 due to the smaller size relative to the inlet.
[13] Minimum dimension and capacity are based on the downstream asset STMMN07142 due to the smaller size relative to the inlet.
[14] Minimum dimension and capacity are based on the downstream asset STMMN07164 due to the smaller size relative to the inlet.
[15] Minimum dimension and capacity are based on downstream asset dimensions due to the smaller size relative to the inlet.

Creek	Scenario	Run(s)	Initiation Point		Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width	Height	Capacity	Return Period	Notes
			Latitude	Longitude						(mm)	(mm)	(m^3/s)	(years)	
Panorama Creek	20	1	49.33125	-122.95068	Panorama Dr Blockage	Panorama Dr	STMCUL00447	Blocked	Round	1200		2.0	100-300	
Mathews Brook	21	1	49.33226	-122.9487	Channel Overflow	-	None	-	-	-	-	-	30-100	
Mathews Brook	22	2	49.33227	-122.9487	Channel Overflow	-	None	-	-	-	-	-	100-300	
Gavles Creek	23	1	49.33329	-122.94679	Panorama Dr Blockage	Panorama Dr	STMCUL00451 [STMCUL00452]	Blocked	Round	900		1.0	30-100	
Gavles Creek	24	2	49.3329	-122.94679	Panorama Dr Blockage	Panorama Dr	STMCUL00451 [STMCUL00452]	Blocked	Round	900		1.0	100-300	
Cleopatra Creek	25	1	49.33347	-122.9445	Panorama Dr Blockage	Panorama Dr	STMCUL00643 [STMCUL00644]	Blocked	Round	1200		2.0	30-100	
							STMCUL00233	Clear	Round	1200		2.0		
							STMCUL00234	Clear	Round	1200		2.0		
Cleopatra Creek	26	2	49.33347	-122.9445	Panorama Dr Blockage	Panorama Dr	STMCUL00643 [STMCUL00644]	Blocked	Round	1200		2.0	100-300	
							STMCUL00233	Clear	Round	1200		2.0		
							STMCUL00234	Clear	Round	1200		2.0		
Francis Creek	27	-	49.332369	-122.939348	Panorama Dr Blockage	Panorama Dr	BGCSTMCUL00009	Blocked	Box	1800	1400	4.7	100-300	Interpreted hazard intensity zones delineated based on field observations with experience and judgement
Unnamed Creek	28	-	49.338619	-122.927993	Fire Lane 2 Blockage	Fire Lane 2	BGCSTMCUL00094	Blocked	Round	800		0.7	30-100	Interpreted hazard intensity zones delineated based on field observations with experience and judgement
						Fire Lane 2	BGCSTMCUL00095	Blocked	Round	750		0.6		
Ward Creek	29	1	49.340992	-122.931723	Indian River Dr Blockage	Indian River Dr	STMCUL00662	Blocked	Round	900		1.0	100-300	
						Indian River Dr	STMCUL00663	Clear	Round	1000		1.4		
						Fire Lane 2	BGCSTMCUL00093	Clear	Round	900		1.0		
						Fire Lane 2	BGCSTMCUL00092	Clear	Round	800		0.7		
						Fire Lane 2	BGCSTMCUL00091	Clear	Round	1000		1.3		
Ostler Creek	30	1	49.34363	-122.93344	Indian River Dr	Sunshine Falls Ln	STMCUL00225	Clear	Round	1500		3.4	100-300	
						Indian River Dr	STMCUL00226	Blocked	Round	2100		8.0		

Creek	Scenario	Run(s)	Initiation Point		Blockage Scenario	Street Location	Modelled Asset ID(s) [4]	Blockage Status	Culvert Shape	Diameter/ Width	Height	Capacity	Return Period	Notes
			Latitude	Longitude						(mm)	(mm)	(m^3/s)	(years)	
Allan Creek	31	1	49.3425	-122.92744	Indian River Dr – 20 year	Sunshine Falls Ln	STMCUL00320	Clear	Round	1500		3.4	30-100	
						Sunshine Falls Ln	STMCUL00587	Clear	Round	1500		3.4		
						Indian River Dr	STMCUL00227 [16] (BGCSTMCUL00080)	Blocked	Round	1500		3.4		
						Indian River Dr	STMCUL00228 [17] (BGCSTMCUL00073)	Blocked	Round	1200		1.9		
						Indian River Dr	BGCVEHBRG00001	Clear	Box	32000	100	5.1		
Allan Creek	32	2	49.3425	-122.92744	Indian River Dr – 200 year	Sunshine Falls Ln	STMCUL00320	Clear	Round	1500		3.4	100-300	
						Sunshine Falls Ln	STMCUL00587	Clear	Round	1500		3.4		
						Indian River Dr	STMCUL00227 (BGCSTMCUL00080)	Blocked	Round	1500		3.4		
						Indian River Dr	STMCUL00228 (BGCSTMCUL00073)	Blocked	Round	1200		1.9		
						Indian River Dr	BGCVEHBRG00001	Clear	Box	3200	100	5.1		
						Sunshine Falls Ln	STMCUL00320	Clear	Round	1500		3.4		
Allan Creek	33	3	49.3425	-122.92744	Fence blocking bridge	Sunshine Falls Ln	STMCUL00320	Clear	Round	1500		3.4	100-300	
						Sunshine Falls Ln	STMCUL00587	Clear	Round	1500		3.4		
						Indian River Dr	STMCUL00227 (BGCSTMCUL00080)	Clear	Round	1500		3.4		
						Indian River Dr	STMCUL00228 (BGCSTMCUL00073)	Clear	Round	1200		1.9		
						Indian River Dr	BGCVEHBRG00001	Blocked	Box	3200	100	5.1		

[16] STMCUL00227 is recorded at the incorrect location, BGCSTMCUL00080 is shown on DNVHIT at the correct location of the culvert.
[17] STMCUL000228 is recorded at the incorrect location, BGCSTMCUL00073 is shown on DNVHIT at the correct location of the culvert.

M.2.4. Results

As described in Section 5.3 of the main report, the model results are displayed with a "flow intensity index" (I_{DF}). The flow intensity index is calculated as flow depth multiplied by the square of flow velocity. The index is directly proportional to flow impact pressure (Zanchetta et al. 2004, Kang and Kim 2016) and can be empirically related to building damage (Jakob et al. 2012). It is not appropriate for estimating damages associated with low velocity flooding (e.g., approximately < 1 m/s), where values of I_{DF} will approach zero irrespective of flood depth.

M.2.5. Limitations

Debris-flood inundation involves complex and dynamic physical processes that are variable in space and time. It is unlikely that two debris floods, even with identical volumes, result in the same avulsions, bank erosion and channel bed aggradation due to differences in the composition of the sediment transport, the number of surges and poorly predictable avulsions due to intermittent debris deposition or channel erosion.

Given the impracticality of modelling all conceivable hydrograph shapes, several simplifying assumptions were made. As such, uncertainties exist that influence the model outcome, including uncertainties in the resolution of the topographic input, the location of the culverts, culvert capacities, and the detailed effects of buildings and streets on flow behavior. Moreover, modeling is based on the current channel, culvert and development configurations. Any future alterations are not included in the model.

Also, FLO-2D does not account for fan surface erosion, sediment transport and deposition processes, the influence of deep standing or flowing water along the path, or displacement waves that could occur in such standing water upon impact by the flow front. The velocity estimates are approximations and may vary according to microtopography and various flow obstacles or channelization that occurs during the debris flood. These cannot be captured in a rigorous manner at the scale of modelling.

It should further be understood that the intent of modelling is to indicate the potential extent of overland flow in the event of culvert or channel blockages resulting from sediment and large woody debris transport. For developed reaches, the precision of the LiDAR data and the grid size used in FLO-2D (2 m) is insufficient to accurately model channel hydraulics. Detailed surveys would be required to conduct hydraulic modelling at this scale.

Given these uncertainties, the raw model results should not be used directly to estimate consequences and associated economic or safety risk. Additional interpretation and judgment is required to prepare hazard intensity maps for each scenario that are suitable for risk analysis. Such analysis would also consider the combined risk from different scenarios for areas impacted by more than one hazard scenario.

M.3. PERCY CREEK DEBRIS FLOW MODELLING

M.3.1. Introduction

Numerical modelling of debris flows at Percy Creek provided the basis for the estimation of spatial impact probabilities and corresponding debris-flow intensities. These serve as inputs to the quantitative risk assessment (QRA) described in Section 6 of the main report, and in Appendix N. This section describes the debris-flow modelling approach, input and results.

M.3.2. Methodology and Input

Debris-flow modelling for Percy Creek was carried out using the three-dimensional numerical model *DAN3D* (McDougall and Hungr 2004). *DAN3D* was developed specifically for the analysis of rapid landslide motion across complex 3D terrain and is well-suited to the simulation of coarse debris flows that deposit on relatively steep slopes, like Percy Creek fan. BGC has used *DAN3D* for the same purposes on other projects.

The model simulates landslide motion from initiation to deposition and requires the following inputs, as described in detail below:

- A digital elevation model (DEM) of the topography in the study area, which defines the sliding surface across which the simulated landslide travels
- A corresponding DEM that delineates the extent and thickness of the initial landslide
- A corresponding DEM that delineates the extent and thickness of erodible material along the path that could be entrained by the landslide as it passes
- A user-specified entrainment rate that determines how much of the available erodible material is picked up by the landslide
- User-specified flow resistance parameters that control how fast and how far the simulated landslide travels.

M.3.2.1. Sliding Surface

The sliding surface that was used for debris-flow modelling was based on the bare earth LiDAR DEM provided by DNV and acquired in spring/summer 2013. The LiDAR data were resampled to 5 x 5 m grid spacing and smoothed to reduce surface roughness and improve numerical model stability. Resampling to 5 x 5 m spacing is standard procedure for debris-flow analyses at this scale to ensure that the model input parameter values that are selected are comparable with previous similar analyses. This generalization results in some loss of topographic details (e.g., large boulders or channel constrictions that could locally affect the flow path and flow depth), but does not substantively affect the debris-flow modelling results.

M.3.2.2. Debris Flow Volumes and Source Locations

Debris-flow modelling was based on the ‘best estimate’ frequency-magnitude curve described in Appendix H. Four debris-flow volume classes were modelled corresponding to 30-100 year, 100-300 year, 300-1,000 year and 1,000-3,000 year return period events, with sub-scenarios modelled within each volume class as described below. In all four cases, constant entrainment rates were specified between the source area and the fan apex to achieve the desired final 'best estimate' volumes. The initial and final debris-flow volumes that were modelled are summarized in Table M.3-1.

Table M.3-1. Summary of modelled debris flow volumes.

Volume Class	Return Period (years)	Modelled Initial Volume[1] (m^3)	Modelled Final Volume Reaching Fan[1] (m^3)
1	30-100	500	5,000
2	100-300	1,600	9,600
3	300-1,000	2,200	12,200
4	1,000-3,000	6,000	16,000

Note:
1. Volumes are based on the 'best estimate' frequency-magnitude curve described in Appendix H.

All debris flows were assumed to initiate from source location 'B' described in Appendix H. Based on BGC's experience modelling debris flows on similar creeks in the region, the modelled results below the fan apex are not expected to be sensitive to this assumption.

All debris flows were modelled as single events (as opposed to events involving multiple source failures and/or surges that result in the same total event volume). This approach likely results in relatively conservative estimates of flow depth and velocity, and in turn vulnerability.

M.3.2.3. Avulsion Scenarios

Flow avulsions out of the active creek channel can be caused by obstructions that develop during a debris flow, for example, due to tree jams, deposition of coarse debris lobes and levees, or channel bank collapses. These processes cannot be simulated automatically in *DAN3D*. Potential avulsion scenarios were therefore simulated manually by adjusting the local elevation of the sliding surface to mimic channel blockages.

Channel blockages or bank overtopping could occur on Percy Creek near the fan apex, at the bridge, or at a number of other random locations. However, it is not practical to model all potential avulsions that could occur. Therefore, in addition to a baseline scenario in which the entire flow stays in the active channel, three representative avulsion scenarios (A-C) were considered:

A. Avulsion towards the south fan sector (Zone A, adjacent to the active creek channel).
B. Avulsion towards the north fan sector (Zone B).
C. Avulsion towards the mid-fan sector (Zone C).

In each avulsion scenario listed above, the possibility that some proportion of the flow would stay in the active channel was considered (i.e. it is considered relatively unlikely that the entire flow would leave the channel).

Probabilities associated with each of these avulsion scenarios were estimated, as summarized in Table M.3-2, based on the following general pattern:

- Smaller events are more likely to stay in the active channel
- The probability of an avulsion into Zone A is higher than the probability of an avulsion into Zone B or Zone C, due to the proximity of Zone A to the active channel
- The probability of an avulsion into Zone B is higher than the probability of an avulsion into Zone C, because Zone B has a concave cross-slope profile that would tend to attract flows, whereas Zone C has a convex cross-slope profile that would tend to divert flows away.

Table M.3-2. Estimated avulsion scenario probabilities.

Volume Class[1]	Avulsion Scenario Probability			
	No Avulsion (entire flow stays in active channel)	Partial Avulsion Into Zone A (some flow stays in active channel)	Partial Avulsion Into Zone B (some flow stays in active channel)	Partial Avulsion Into Zone C (some flow stays in active channel)
1	0.88	0.1	0.01	0.01
2	0.79	0.15	0.05	0.01
3	0.65	0.2	0.1	0.05
4	0.5	0.25	0.15	0.1

Note:
1. See Table for volume class descriptions.

M.3.2.4. Resistance Parameters

To simulate debris flows on Percy Creek, the Voellmy flow resistance model was used. The Voellmy model is governed by two parameters: 1) a friction coefficient, f, which determines the slope angle on which material begins to deposit (i.e., if the friction coefficient is higher than the local slope gradient, material will decelerate and begin to deposit); and 2) a turbulence parameter, ξ, which produces a velocity-dependent resistance that tends to limit flow velocities (similar to air drag acting on a falling object).

A single set of resistance parameter values was used, f = 0.2 and ξ = 500 m/s^2, to simulate representative “high mobility” debris flows on Percy Creek (i.e. flows that would likely reach homes near the water line). These values are consistent with values obtained through back-analysis of recent debris flows in southwestern BC. The friction coefficient of 0.2 also approximates the fan gradient near the water line, and is therefore consistent with the behaviour of events that have deposited in that area in the past. The turbulence parameter of 500 m/s^2 generally limits the simulated flow velocities on the fan to less than 10 m/s, which is in the range of peak velocities that have been estimated for local historical debris flows (Thurber 1983).

For comparison, GEO (2011) also recommends using a friction coefficient of 0.2 and a turbulence parameter of 500 m/s^2 to model typical saturated, channelized debris flows in Hong Kong. Similar input parameter combinations have also been used by BGC on other projects in BC.

M.3.2.5. Summary of Debris Flow Model Scenarios

The debris-flow model scenarios described in the preceding sections are summarized in Table M.3-3.

Table M.3-3. Summary of debris-flow model scenarios.

Scenario	Run(s)	Avulsion	Blockage Scenario	Return Period (years)
34	1	A	South Avulsion	30-100
34	1	B	North Avulsion	30-100
34	1	C	Mid-fan Avulsion	30-100
35	2	A	South Avulsion	100-300
35	2	B	North Avulsion	100-300
35	2	C	Mid-fan Avulsion	100-300
36	3	A	South Avulsion	300-1000
36	3	B	North Avulsion	300-1000
36	3	C	Mid-fan Avulsion	300-1000
37	4	A	South Avulsion	1000-3000
37	4	B	North Avulsion	1000-3000
37	4	C	Mid-fan Avulsion	1000-3000

M.3.3. Results

The *DAN3D* results indicate the maximum simulated debris-flow intensity index (Jakob et al. 2012) within the modelled inundation area on the fan. This index indicates the simulated destructive potential of a flow, and is calculated as flow depth multiplied by the square of flow velocity.

In general, larger modelled flow volumes resulted in longer modelled runout distances, larger modelled inundation areas and higher modelled intensities. Relatively high intensities were also generally modelled within the active channel, as channel confinement resulted in relatively high modelled flow depths and velocities.

M.3.4. Limitations

Most of the model limitations described in Section M.2.5 also apply to the *DAN3D* debris flow modelling described above. Therefore, the raw *DAN3D* model results should similarly not be used directly to estimate consequences and associated economic or safety risk. Additional interpretation and judgment is required to prepare hazard intensity maps that are suitable for risk analysis.

M.4. INTERPRETED HAZARD INTENSITY MAPS

Drawings M-02 to M-40 show the hazard intensity maps prepared for each hazard scenario. These maps form the hazard basis for risk analyses and show the extent and "intensity", or destructive potential, of a flood, debris flood or debris flow. They are based on the hazard modelling results, supplemented by judgement and generalized to account for model uncertainty. BGC completed fieldwork on February 23, 2016 to verify each hazard scenario based on judgment.

The maps display flow intensity categorized according to the divisions used for vulnerability estimation. Table M.4-1 describes the categories shown on the hazard intensity maps in terms of potential levels of building damage. Numerical criteria used to quantify vulnerability (building damage and probability of loss of life) are provided in Appendix N.

Table M.4-1. Hazard intensity index (I_{DF}) categories related to building damage. Adapted from Jakob et al. (2012) with additional data from Kang and Kim (2016).

Hazard Intensity Index (Range)	Destructive Potential	
	Category	Description
<1	Sedimentation	Flood and sediment-related damage.
1-10	Some damage	High likelihood of moderate to major building structure damage due to impact pressure. Certain severe sediment and water damage. Building repairs required, possibly including some structural elements.
10-100	Major damage	High likelihood of major to severe building structure damage due to impact pressure. Certain severe sediment and water damage. Major building repairs required including to structural elements.
100+	Destruction	Very high likelihood of severe building structure damage or collapse. Complete building replacement required.

For urban creeks, each hazard intensity map shows areas assumed as certain to be impacted by a given scenario. Areas outside the hazard extent are not considered impacted. This approach is considered appropriate for debris flood creeks where the objective is to describe typical scenarios for risk estimation supporting risk control prioritization.

At Percy Creek, a wider range of flow characteristics is possible for a given debris flow magnitude, with more watery flows expected to run out further than those with higher sediment concentration. Interpretation was therefore required to delineate runout exceedance probability contour lines (isolines associated with a certain conditional probability that debris flows of a given volume class will travel beyond the position of the line). The probabilities that debris flows of a given volume class would reach the water line in each zone were estimated, as summarized in Table M.4-2 based on the following general pattern:

- Larger events are more likely to reach the water line
- Events that stay in the active channel are more likely to reach the water line (this tends to occur because unconfined debris flows are characterized by lower flow depths and faster water drainage from the debris mass, both of which increase frictional resistance)
- Considering that approximately 1/3 of the Percy Creek fan extends underwater, the probability of unchannelized flows of any size reaching the water line likely averages roughly 30%.

Table M.4-2. Estimated runout exceedance probabilities.

Volume Class[1]	Runout Exceedance Probability at Water Line			
	Active Channel	Zone A	Zone B	Zone C
1	0.6	0.1	0.1	0.1
2	0.7	0.2	0.2	0.2
3	0.8	0.3	0.3	0.3
4	0.9	0.4	0.4	0.4

Note:
1. See Table for volume class descriptions.

Note that the raw model results that were used as the basis for the interpreted hazard intensity maps at Percy Creek were based on simulations of coarse debris-flows using *DAN3D*. *DAN3D* does not simulate the finer, more fluid afterflow phase that typically follows coarse debris-flow surges and often travels beyond the limit of the coarse debris deposits. The afterflow phase is represented on the interpreted hazard intensity maps by areas of lower flow intensity extending further than the raw model results.

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INTERPRETED HAZARD INTENSITY MAPS

FIGURE NO	FIGURE NAME – HAZARD INTENSITY MAPS
M-02	SCENARIO 1 - MACKAY CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-03	SCENARIO 2 - MACKAY CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-04	SCENARIO 3 - MACKAY CREEK, RUN 4 - INTERPRETED HAZARD INTENSITY MAP
M-05	SCENARIO 4 - MACKAY CREEK, RUN 5 - INTERPRETED HAZARD INTENSITY MAP
M-06	SCENARIO 5 - MOSQUITO CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-07	SCENARIO 6 - MOSQUITO CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-08	SCENARIO 7 - MOSQUITO CREEK, RUN 3 - INTERPRETED HAZARD INTENSITY MAP
M-09	SCENARIO 8 - MISSION CREEK, RUN 1-4 - INTERPRETED HAZARD INTENSITY MAP
M-10	SCENARIO 10 - MISSION CREEK 2, RUN 5 - INTERPRETED HAZARD INTENSITY MAP
M-11	SCENARIO 11 - MISSION CREEK 3, RUN 6 - INTERPRETED HAZARD INTENSITY MAP
M-12	SCENARIO 12 - HASTINGS AND DYER CREEKS - INTERPRETED HAZARD INTENSITY MAP
M-13	SCENARIO 13 - KILMER CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-14	SCENARIO 14 - KILMER CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-15	SCENARIO 15 - THAMES CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-16	SCENARIO 16 - THAMES CREEK, RUN 2, 3 - INTERPRETED HAZARD INTENSITY MAP
M-17	SCENARIO 17 - TAYLOR CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-18	SCENARIO 18 - GALLANT CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-19	SCENARIO 19 - GALLANT CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-20	SCENARIO 20 - PANORAMA CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-21	SCENARIO 21 - MATHEWS BROOK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-22	SCENARIO 22 - MATHEWS BROOK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-23	SCENARIO 23 - GAVLES CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-24	SCENARIO 24 - GAVLES CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-25	SCENARIO 25 - CLEOPATRA CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-26	SCENARIO 26 - CLEOPATRA CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-27	SCENARIO 27 - FRANCIS CREEK - INTERPRETED HAZARD INTENSITY MAP
M-28	SCENARIO 28 - UNNAMED CREEK - INTERPRETED HAZARD INTENSITY MAP
M-29	SCENARIO 29 - WARD CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-30	SCENARIO 30 - OSTLER CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-31	SCENARIO 31 - ALLAN CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-32	SCENARIO 32 - ALLAN CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-33	SCENARIO 33 - ALLAN CREEK, RUN 3 - INTERPRETED HAZARD INTENSITY MAP
M-34	SCENARIO 34 - PERCY CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP
M-35	SCENARIO 35 - PERCY CREEK, RUN 2 - INTERPRETED HAZARD INTENSITY MAP
M-36	SCENARIO 36 - PERCY CREEK, RUN 3 - INTERPRETED HAZARD INTENSITY MAP
M-37	SCENARIO 37 - PERCY CREEK, RUN 4 - INTERPRETED HAZARD INTENSITY MAP
M-38	SCENARIO 38 - SCOTT-GOLDIE CREEK - HAZARD ZONES MAP
M-39	SCENARIO 39 - SHONE AND UNDERHILL CREEKS - HAZARD ZONES MAP
M-40	SCENARIO 40 - COLDWELL AND FRIAR CREEKS - HAZARD ZONES MAP

NOTES:

1. REGULATORY, CADASTRAL, AND BUILDINGS WERE OBTAINED FROM DNV.
2. ROADS, TRANSPORTATION AND TRAILS WERE OBTAINED FROM DNV. PEDESTRIAN AND VEHICLE BRIDGE LOCATIONS WERE PROVIDED BY DNV AND SUPPLEMENTED WITH FIELD OBSERVATIONS FROM BGC.
3. WATERCOURSE AND WATERBODIES EXCEPTING STUDY CREEKS WERE OBTAINED FROM DNV.
4. STUDY CREEKS WERE DIGITIZED BY BGC BASED ON LiDAR AND FIELD OBSERVATIONS. STUDY CREEK CHAINAGE WAS ASSIGNED BY BGC WITH THE CHAINAGE INCREASING FROM 0 DOWNSTREAM FROM THE HEADWATERS.
5. STORMWATER INFRASTRUCTURE WAS OBTAINED FROM DNV. UPSTREAM OF THE DEVELOPED AREA, BGC CHARACTERIZED ADDITIONAL CULVERTS DURING FIELD AND OFFICE INVESTIGATIONS.
6. DEBRIS CONTROL STRUCTURES WERE IDENTIFIED AND CHARACTERIZED BY BGC DURING FIELD INVESTIGATIONS. THOSE SHOWN ONLY REPRESENT SITES VISITED BY BGC AND ARE NOT THE FULL INVENTORY OF DEBRIS CONTROL STRUCTURES WITHIN DNV.
7. INTERPRETED IMPACT INTENSITY ZONES WERE DELINEATED BY BGC BASED ON THE RAW MODEL OUTPUTS CALIBRATED WITH FIELD OBSERVATIONS AND GROUND TRUTHING.
8. KWL (2003) HAZARD ZONES SHOWN ON DRAWINGS M-38 to M-40 ARE BASED ON THE REPORTS FOR EACH CREEK FROM THE “DEBRIS FLOW STUDY AND RISK MITIGATION ALTERNATIVES” REPORT SUITE.
9. WATERSHED BOUNDARIES WERE DRAWN BY BGC FOR THE PURPOSE OF FLOW FREQUENCY-MAGNITUDE ESTIMATION ABOVE THE DEVELOPMENT INTERFACE AND DO NOT NECESSARILY INCLUDE THE ENTIRE DOWNSTREAM WATERSHED. THEY SHOULD NOT BE USED TO DETERMINE HAZARD EXTENTS.

LEGEND

IMPACT INTENSITY (INTERPRETED)

- <=1
- >1 to <=10
- >10 to <=100
- >100

KWL (2003) HAZARD ZONES

- DIRECT IMPACT ZONE
- INDIRECT IMPACT ZONE
- SEDIMENTATION ZONE
- BUILDINGS
- PROPERTY BOUNDARIES (PARCELS)
- STORM MAIN (STUDY CREEK)
- STORM MAIN
- STORM CULVERT (STUDY CREEK)
- STORM CULVERT
- STORM CULVERT BGC CHARACTERIZED
- WATERSHED
- EXISTING DEBRIS CONTROL STRUCTURE
- PEDESTRIAN BRIDGE
- VEHICLE BRIDGE
- TRAIL
- ROAD

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DATE:	MAY 2017
DRAWN:	MIB
CHECKED:	LCH, MTL, KH
APPROVED:	KH, HW

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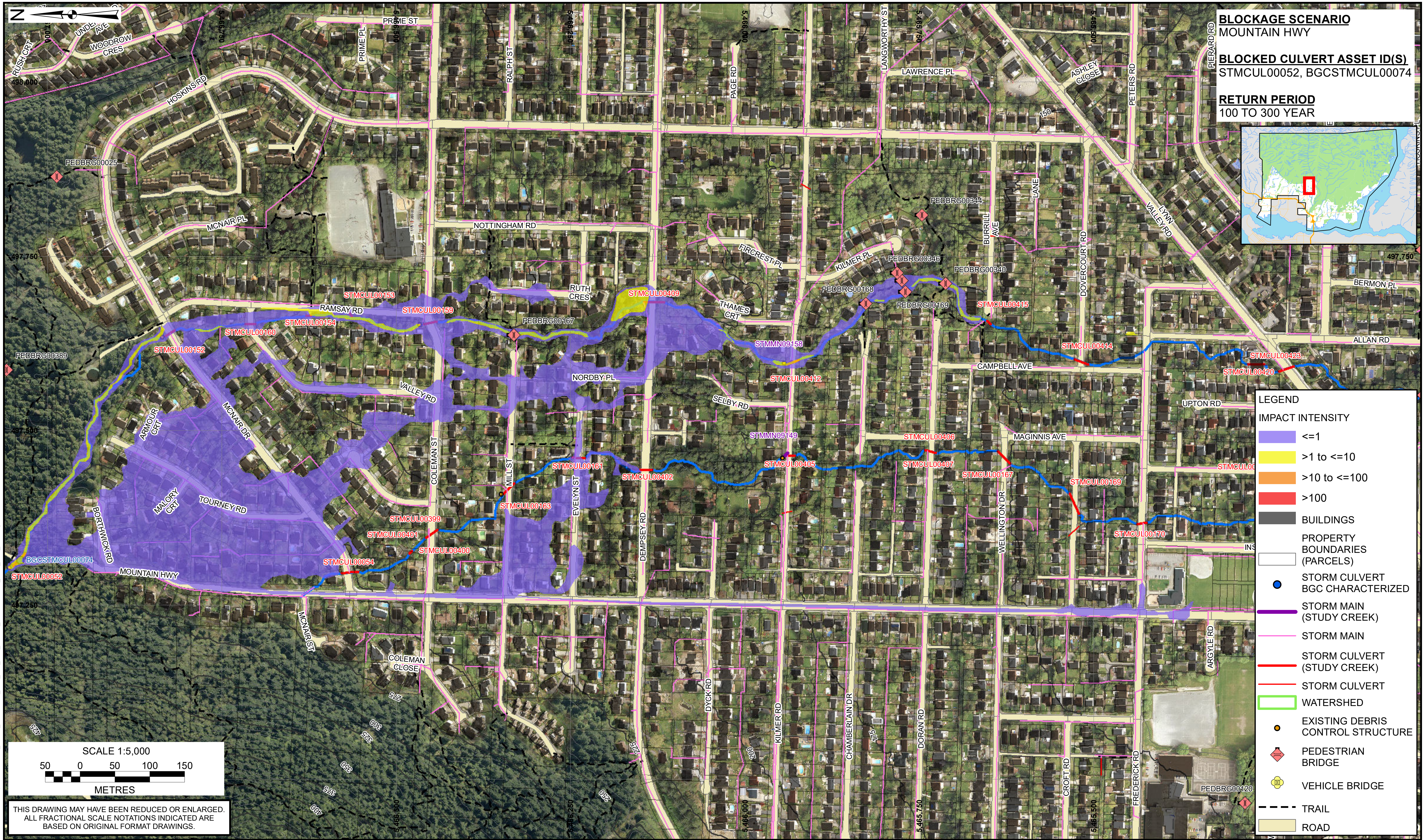
CLIENT: NORTH VANCOUVER

PROJECT: DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT

TITLE: DRAWING LIST AND LEGEND

PROJECT No.: 0404-054

DWG No: M-01



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10. THE RETURN PERIOD SHOWN CORRESPONDS TO THE MODELLED RETURN PERIOD EVENT. FOR THE INDIVIDUAL CULVERT BLOCKAGE RATING, REFER TO APPENDIX F OR THE DNVHIT SITE.

SCALE:	1:5,000
DATE:	MAY 2017
DRAWN:	MIB
CHECKED:	LCH, MTL, KH
APPROVED:	KH, HW

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CLIENT: NORTH VANCOUVER DISTRICT

PROJECT: DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT

TITLE: SCENARIO 15 - THAMES CREEK, RUN 1 - INTERPRETED HAZARD INTENSITY MAP

PROJECT No.: 0404-054

DWG No: M-15

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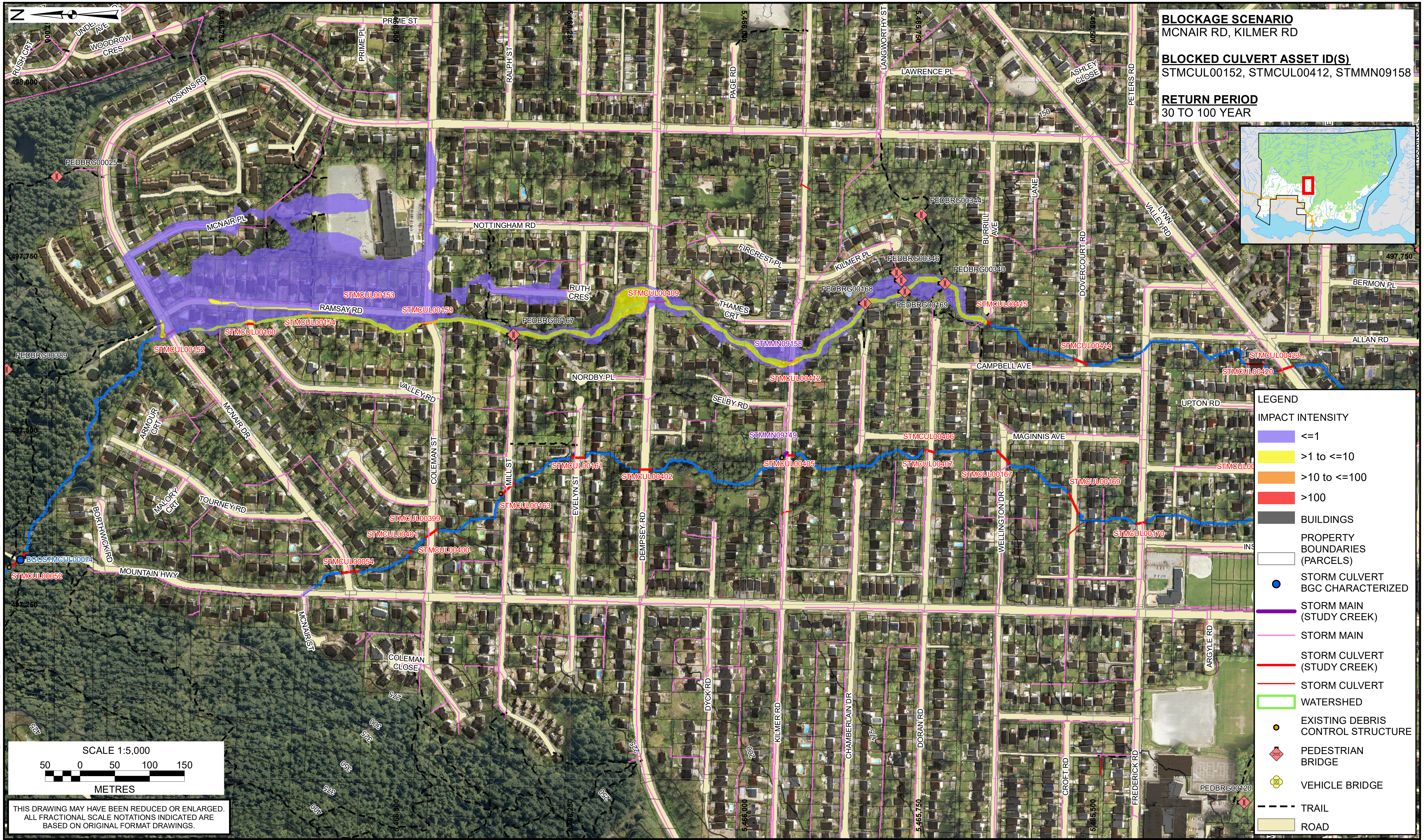

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
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3. FOR FULL EXPLANATION OF DNV INFRASTRUCTURE AND HYDROLOGY SYMBOLS REFER TO DRAWING M-01.
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5. THIS MAP REPRESENTS A SNAPSHOT IN TIME. FUTURE CHANGES (DEVELOPMENT, MITIGATION WORKS, GEOHAZARD EVENTS) MAY WARRANT THE RE-DRAWING OF CERTAIN AREAS.
6. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY DNV FLOWN IN SPRING/SUMMER 2013 - PROVIDED JUNE 2015.
7. THIS MAP REPRESENTS THE INTERPRETED EXTENT AND INTENSITY OF A PARTICULAR HAZARD SCENARIO. OTHER SCENARIOS ARE POSSIBLE. HAZARD INTENSITY IS ONLY INTERPRETED DOWNSTREAM OF THE DEVELOPMENT INTERFACE.
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SCALE:	1:5,000
DATE:	MAY 2017
DRAWN:	MIB
CHECKED:	LCH, MTL, KH
APPROVED:	KH, HW

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PROJECT: DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT

TITLE: SCENARIO 16 - THAMES CREEK, RUN 2, 3 - INTERPRETED HAZARD INTENSITY MAP

PROJECT No.:	DWG No:
0404-054	M-16

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APPENDIX N
RISK ASSESSMENT METHODS AND RESULTS

N.1. INTRODUCTION

Risk assessment involves estimation of the likelihood that a debris-flood or debris-flow scenario will occur, impact elements at risk, and cause particular types and severities of consequences. BGC assessed risk for creeks with identified hazard scenarios. Section 5.2.1, Table 5-1 of the main body of the report describes the hazard scenarios considered in the assessment.

BGC assessed the following types of risks in this assessment:

- Safety risk: annual probability of loss of life for persons within buildings.
- Economic risk: direct building damage[1] costs.

Section 6.0 of the Main Report described the objectives of BGCs assessment, provided an overview of methodologies used, and describes the risk tolerance criteria used to evaluate safety risk. This appendix provides additional details on the methodology used for risk analyses.

BGC used both quantitative risk assessment (QRA) and semi-quantitative risk assessment (semi-QRA) methods to estimate risk. QRA estimates an annual likelihood of some consequence (economic or safety risk), considering all hazard scenarios modelled for a given creek. In contrast, semi-QRA provides a relative, numerical risk rating and considers a single hazard scenario.

Section N.2 describes QRA methods, which were used to estimate safety and economic risk for each creek system as a whole. The results of QRA support risk reduction prioritization for each creek (e.g. should creek “X” be higher priority, from a risk perspective, than creek “Y”).

Section N.3 describes semi-QRA methods, which were used to assign economic risk ratings to individual culverts or storm water mains. The economic risk rating for individual culverts addresses the question, “what is the probability that a particular culvert blocks and results in some level of direct building damages? The results of semi-QRA support risk reduction prioritization for individual culverts (e.g. should culvert “X” be higher priority, from a risk perspective, than culvert “Y”). Safety risk ratings were not assigned to individual culverts.

N.1.1. Safety Risk

Safety risk (risk to life) was estimated separately for individuals and groups. Individual risk assessment considered the probability that hazard scenarios result in loss of life for a particular individual, referred to as Probability of Death of an Individual (PDI). Individual risk levels are independent of the number of persons exposed to risk. In contrast, group risk assessment considered the cumulative probability that hazard scenarios result in at least a certain number of fatalities. Unlike individual risk, group risk is proportional to the number of persons exposed to hazard: all else being equal, a greater number of persons exposed to hazard results in higher risk. It is possible that individual risk is considered tolerable, but group risk is not.

[1] Direct building damage: quantitative estimation of damage to building structure or contents directly resulting from debris flood or debris flow impact.

Group risk was represented graphically on an F-N curve, as shown in Figure N.1-1. The Y-axis shows the annual cumulative frequency, f_i, of each hazard scenario, and the X-axis shows the estimated number of fatalities, N_i, where:

$$f_i = \sum_{i=1}^{n} P(H)_i P(S:H)_i P(T:S)_i \quad [1]$$

and N_i is represented by equation [3] (see Section 0).

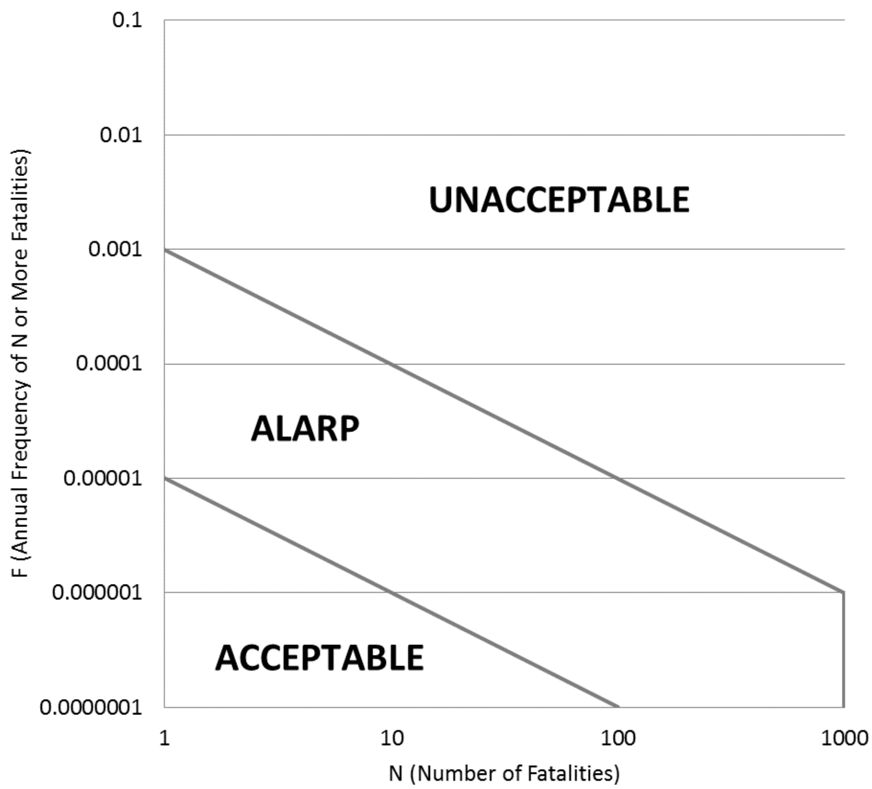

Figure N.1-1. F-N Curve showing group risk tolerance criteria as defined by GEO (1998).

N.1.2. Economic Risk

Direct building damages were calculated as total annualized damage considering all scenarios, as well as direct damage costs for individual hazard scenarios.

Direct building damage is calculated as a damage cost per m^2 of building area, assessed separately for building structure damage and building contents damage. Total estimated damage is the sum of building structure damage and building contents damage. Section N.2.4 provides further details on methods to estimate the vulnerability of buildings to direct damages by debris floods or debris flows.

In all cases, damage costs reflect current conditions and do not consider future changes (e.g. inflation, changes in building replacement costs over time)

N.2. QUANTITATIVE RISK ANALYSIS

Risk (P_E) was estimated using the following equation:

$$P_E = \sum_{i=1}^{n} P(H)_i P(S:H)_i P(T:S)_i \, N \qquad [2]$$

where:

$P(H)_i$ is the annual hazard probability of debris-flow or debris-flood scenario i of n, where *n* is the total number of scenarios. It addresses the question, "how likely is the event"?

$P(S:H)_i$ is the spatial probability that the event would reach the element at risk. It addresses the question, "what is the chance that the event will reach an element at risk"?

$P(T:S)_i$ is the temporal probability that the element at risk would be in the impact zone at the time of impact. It answers the question, "what is the chance of someone or something being in the area affected by the hazard when it occurs"

$N = V_i E_i$ describes the consequences [3]

where:

V_i is the vulnerability, which is the probability elements at risk will suffer consequences given hazard impact with a certain severity. For persons, vulnerability is defined as the likelihood of fatality given flow impact. For buildings, it is defined as the level of damage, measured as a proportion of the building replacement cost or as an absolute cost.

E_i is a measure of the elements at risk, quantifying the value of the elements that could potentially suffer damage or loss (e.g. number of persons, building value).

Methods used to estimate each variable in Equation [1] are described below.

N.2.1. Hazard Probability, $P(H)$

Hazard probability, $P(H)_i$, corresponds to the annual probability of occurrence of each hazard scenario, which are defined as annual probability ranges (see Section 5 of the main report). The bounds of a given range are "exceedance" probabilities, corresponding to the probability that an event of *at least* a certain magnitude will occur. As such, for a scenario with the annual probability range P_{min} to P_{max}, the probability of events within this range corresponds to:

$$P(H)_i = P_{max} - P_{min} \qquad [4]$$

For example, for the 1:30 – 1:100 year range, this would correspond to:

$$P(H)_i = \frac{1}{30} - \frac{1}{100} = \frac{1}{43} \qquad [5]$$

In the example above, there is a 1 in 43 year chance that an event greater than the 1:30 year event, but not larger than the 1:100 year event, will occur.

N.2.2. Spatial Probability, $P(S:H)$

For creeks subject to shallow depth flood or debris floods, overall flow extents are shown by interpreted model results. However, not all homes within these zones will necessarily be impacted by a single event. Uncertainty exist in predicting the spatial probability of impact due to the random nature of individual events and the influence of micro-topography (e.g. influence of road curbs) on shallow flows that cannot be fully captured in the modelling. BGC conservatively assumed that flows with intensities exceeding 1 (I_{DF}>1) are certain to impact all buildings ($P(S:H)$=1) within these areas. For low intensity flows (I_{DF}<1), BGC used judgement calibrated with November 2014 building damage records to assign spatial probabilities of building impact, with lower values assigned to areas where modelled flow intensities were very low (I_{DF}<0.1). This is consistent with recorded events (e.g. Nov. 2014) where not all buildings within an affected area were impacted. Buildings outside the hazard zone were not considered subject to impact by a particular scenario ($P(S:H)$=0).

Compared to creeks prone to floods and debris floods, a wider range of flow characteristics is possible for a given debris-flow magnitude at Percy Creek. Specifically, more watery flows are expected to run out further than those with higher sediment concentration. Moreover, flow avulsions near the fan apex can result in flow trajectories primarily towards a certain sector of the fan. For a given hazard scenario, both of these factors influence the spatial probability of debris-flow impact. As described in Appendix M (see Section M.3) and shown on the Percy Creek interpreted hazard intensity maps (Drawings M02-M41), BGC divided the fan into 4 avulsion sectors and interpreted runout exceedance probability contours for each volume class. For a given building location, the spatial probability of debris-flow impact is calculated as:

$$P(S:H) = P(S:H)_1 \times P(S:H)_2 \qquad [6]$$

where

$P(S:H)_1$ is the estimated avulsion scenario probability

$P(S:H)_2$ is the estimated flow runout exceedance probability

Values of $P(S:H)_1$ for a given volume class are shown in Table M.3.2 of Appendix M. The flow runout exceedance probabilities are shown as contours on Percy Creek Drawings M35-M38.

N.2.3. Temporal Probability, $P(T:S)$

For assessment of risk to buildings, temporal probability, $P(T:S)$, was assigned as 1 (certain) based on the assumption that all buildings considered are permanent structures.

For assessment of safety risk, the value of $P(T:S)$ corresponds to the estimated proportion of time spent by persons within a building.

For persons in residential buildings, an average value of 0.5 was assigned for analysis of risk to groups, implying that about half of the residents will be in their homes during a debris flow or debris flood. A more conservative value of 0.9 was used for estimation of individual risk, corresponding to a person spending the greatest proportion of time at home, such as a young child, stay-at-home person, or an elderly person. These values are consistent with those used by BGC on previous safety risk assessments for DNV (e.g. BGC, 2009).

N.2.4. Vulnerability

This section describes methods to estimate the vulnerability of buildings and persons within those buildings to impact by debris-flood or debris-flow scenarios. The following sections refer to debris-flood and debris-flow impacts as “flows”. Vulnerability estimates are based on flow intensity, I_{DF}, which is calculated as follows:

$$I_{DF} = (d)(v^2) \quad [6]$$

where:

I_{DF} is the intensity index.

d is the modelled flow depth.

v is the modelled flow velocity.

N.2.4.1. Low intensity flows ($I_{DF} < 1$)

Lower intensity flows are defined as flows where intensity index (I_{DF}) was less than one. Damages associated with these low intensity flows (very slow or very shallow flows) are typically limited to water damage and sedimentation (as opposed to high impact forces for higher intensity flows that can lead to structural failures). While the possibility of fatalities cannot be entirely ruled out, it is likely due to unpredictable human behavior that cannot be quantified in a risk assessment. For example, several drownings occurred during attempts to recover valuable items from flooded basements during recent (June 2016) flooding in small urban creeks in southern Germany[2].

This section describes criteria used to estimate direct building damage costs as a proportion of estimated building replacement values. These criteria were applied to low intensity flows on urban debris-flood creeks.

BGC estimated building damages for lower intensity flows using flood stage-damage curves developed for Alberta Environment and Parks (AEP) following the damaging floods in southwestern Alberta in June, 2013. The flood stage-damage curves used for this estimation were developed by IBI Group. BGC selected these curves for analysis because they are the only curves that, to BGC’s knowledge, have been developed specifically for residential development in Canada.

The stage-damage curves developed for AEP are based on an inventory of residential and commercial units in Southern Alberta that were flooded in June 2013, as well as other Alberta flood events dating back to the early 1980s. Damage costs are estimated as a cost per unit floor area, for a given flood depth and building type, and exclude garages.

[2] http://www.merkur.de/bayern/hochwasser-drama-simbach-diesem-haus-starben-drei-frauen-meta-6454029.html

BGC assigned DNV buildings with the categories used in the stage-damage criteria (Table N.2-1 and Table), based on estimated building area and building types previously assigned to DNV buildings by Journeay et al. (2015).

Table N.2-1. Residential building classes for stage-damage criteria.

Stage-damage Building Class	Hazus Building Class	Number of Stories[1]	Building Footprint Area (m^2)[1]	Description
A1	RES1	1	> 223	Single Family Dwelling
A2	RES1	2	> 223	Single Family Dwelling
B1	RES1	1	112-223	Single Family Dwelling
B2	RES1	2	112-223	Single Family Dwelling
C1	RES1	1	< 112	Single Family Dwelling
C2	RES1	2	< 112	Single Family Dwelling
D	RES2	-	-	Manufactured Housing
B3	RES3A	-	-	Duplex
B3	RES3B	-	-	Triplex
MW	RES3C	-	-	Multiple Unit Dwellings (5 to 9 units)
MW	RES3D	-	-	Multiple Unit Dwellings (10 to 19 units)
MW	RES3E	-	-	Multiple Unit Dwellings (20 to 49 units)
MA	RES3F	-	-	Multiple Unit Dwellings (50+ units)
S3	RES4	-	-	Temporary Lodging

[1]Number of stories and building footprint area are used as parameters to assign building class for single family dwellings only.

Table N.2-2. Commercial building classes for stage-damage criteria.

Stage-damage Building Class	Stage-damage Content Class	Hazus Building Class	Description
S1	C6	COM1	Retail Trade
S1	J1	COM3	Personal and Repair
S1	A1	COM4	Professional/Technical
S1	K1	COM5	Banks
S1	M1	COM8	Entertainment
S5	N2	EDU1	Grade Schools
S5	N2	EDU2	Colleges/Universities
S5	N1	GOV1	General Government Services
S5	N1	GOV2	Emergency Response Services
S2	L1	IND2	Light Industry
S5	N1	REL1	Religious Buildings

The stage-damage criteria relate flood damages to a particular depth of inundation. However, for the typically shallow flows and sloping ground encountered throughout most of the study area, the modelled flow depths may not simulate the actual depth of water inside an impacted building. Actual water depths will be affected by structural details of the building envelope and the location of water entry and exit points, which cannot be captured at the level of detail of this study. For simplicity, BGC assumed that all buildings contain basements, that ground and first-floor elevation were similar on the uphill side of a building, and that low intensity flows inundate all impacted basements and first-floors to depths of 0.9 m and 0.1 m, respectively. These simplifications are considered reasonable for relative risk estimation given that all impacted buildings are located on slopes and were subject to modelled flows of less than 30 cm depth. Table N.2-3 and Table N.2-4 show the criteria relating flow depth to building structure or contents damage cost per unit floor area, for a given building type and the above assumed inundation depths. Separate costs/areas are listed for the basement and first floor areas of a given building type; these are summed to provide a total loss estimate for the respective building.

For damage cost calculation, BGC estimated the interior area of each floor as encompassing about 40% of the total rooftop footprint. This estimate accounts for the proportion of the footprint covered by a typical roof overhang, interior walls and an unfinished standard garage. It is approximate and does not account for site-specific differences between houses, which could not be captured at the level of detail of assessment. However, as an assumption applied consistently across the study area; this uncertainty is not expected to affect the relative ranking of sites for risk reduction prioritization.

Table N.2-3. Residential stage-damage criteria

Stage-Damage Building Class	Main Floor Structure Damage Cost ($/m^2)	Main Floor Content Damage Cost ($/m^2)	Basement Structure Damage Cost ($/m^2)	Basement Content Damage Cost ($/m^2)
A1	$588	$373	$299	$778
A2	$665	$343	$406	$437
B1	$400	$221	$312	$401
B2	$524	$235	$385	$324
C1	$210	$108	$0	$0
C2	$467	$240	$356	$418
C3	$599	$204	$399	$264
D	$245	$117	$0	$0
MW	$362	$243	$0	$0
MA	$822	$260	$0	$0

Table N.2-4. Commercial stage-damage criteria

Stage-Damage Building Class	Structure Damage Cost ($/m²)	Stage Damage Content Class	Content Damage Cost ($/m²)
A1	$121	S1	$105
B1	$150	S2	$16
C1	$200	S3	$113
C2	$187	S4	$79
C3	$352	S5	$68
C4	$96		
C5	$142		
C6	$209		
C7	$182		
D1	$138		
E1	$148		
F1	$50		
G1	$46		
H1	$20		
I1	$72		
J1	$37		
K1	$121		
L1	$173		
M1	$0		
N1	$59		
N2	$72		

N.2.4.2. High intensity flows (I_{DF} >1)

Higher intensity flows are defined as modelled flows where I_{DF} exceeds 1. These flows have greater potential to result in structural building damage due to dynamic and static impact pressure, and are considered to have credible potential to cause loss of life. Vulnerability ratings for these flows consider the likelihood of fatalities as an indirect consequence of building damage or collapse.

BGC assigned vulnerability ratings using criteria developed from judgement with reference to Jakob et al. (2011). The values used are also consistent with those used by BGC to quantify debris-flood and debris-flow risk for alluvial fans in the Town of Canmore and Municipal District of Bighorn, which were calibrated to damaging flood, debris-flood and debris-flow events in June 2013 (e.g. BGC Engineering 2014, 2015a-e, 2016). The vulnerability estimates contain uncertainty due to factors that cannot be captured at the scale of assessment, such as variations in the structure and contents of a given building and the location of persons within the building at the time of impact.

Table N.2- shows the building vulnerability ratings used for flows where I_{DF} >1. The table also lists the building vulnerability rating applied to low intensity debris-flow zones (I_{DF} <1) at Percy Creek (RFDAM criteria are not applicable to debris flows).

Table N.2-5. Vulnerability criteria for buildings.

Hazard Intensity Index (Range)	Building Damage Description		Building Vulnerability[1]
	Category	Description	Best Estimate
<1	Slight	Low likelihood of building structure damage due to impact pressure. High likelihood of major sediment and/or water damage. Damage level and cost primarily a function of flood-related damages.	0.25[4]
1-10	Moderate	High likelihood of moderate to major building structure damage due to impact pressure. Certain severe sediment and water damage. Building repairs required, possibly including some structural elements.	0.5
10-100	Major	High likelihood of major to severe building structure damage due to impact pressure. Certain severe sediment and water damage. Major building repairs required including to structural elements.	0.8
100-400	Complete	Very high likelihood of severe building structure damage or collapse. Complete building replacement required.	1

Notes:

[1]Value indicate estimated proportion of building replacement value
[4]Applied only to residences impacted by debris flows at Percy Creek. RFDAM stage-damage criteria were applied for low intensity flows (I_{DF}<1) at flood and debris-flood creeks.

Table N.2-6 shows the criteria used to estimate the vulnerability of persons within buildings to debris-flow or debris-flood impact, where vulnerability is primarily an indirect outcome of building damage or collapse. Based on field inspection, BGC also identified several buildings considered more vulnerable to debris flood impact due to their close proximity to the channel. These are listed in Table N.2-7. Based on judgement, BGC assigned slightly more conservative ratings to these homes, as listed in Table N.2-8.Table N.2-6. Vulnerability criteria for persons within buildings

Hazard Intensity Index (Range)	Debris Flows[2]			Debris Floods[2]		
	Lower Bound	Best Estimate	Upper Bound	Lower Bound	Best Estimate	Upper Bound
<1	~0	~0	~0	~0	~0	~0
1-10	~0	0.2	0.4	~0	0.01	0.02
10-100	0.4	0.6	0.8	0.05	0.1	0.2
100-400	0.80	0.9	1	0.2	0.5	1

Notes:
[2]Values indicate estimated probability of loss of life given impact

Table N.2-7. Buildings identified as more highly vulnerable to debris flood impact

Creek	Building AssetID	Address
Mission Creek 1	BLDG02481	310 Newdale Ct.
Gallant Creek 1	BLDG14092	2150 Badger Rd.
Panorama Creek 1	BLDG12516	2525 Panorama Dr.
Mathews Brook 1	BLDG12267	2603 Panorama Dr.
Gavles Creek 1	Not available	2672 Panorama Dr.
Gavles Creek 1	BLDG12061	2679 Panorama Dr.
Gavles Creek 1	BLDG12034	2683 Panorama Dr.
Cleopatra Creek 1	BLDG11848	2755 Panorama Dr.

Table N.2-8. Vulnerability criteria for persons within buildings listed in Table N.2-7

Hazard Intensity Index (Range)	Debris Floods[2]		
	Lower Bound	Best Estimate	Upper Bound
0-1	~0	0.001	0.002
1-10	0.002	0.01	0.02

Notes:
[2]Values indicate estimated probability of loss of life given impact.

N.3. SEMI-QUANTITATIVE RISK ANALYSIS

BGC estimated baseline risk for existing conditions at each culvert, and residual risk assuming implementation of the risk control measures described in Appendix O. Culvert risk ratings were only assigned to culverts on creeks with identified hazard scenarios (see Section 5.2 of the main report).

Table N.3-1 displays the matrix used to determine relative economic risk ratings for individual culverts. The relative ratings range from 1 to 7, with 7 being highest risk.

The hazard rating in the matrix corresponds to the culvert blockage rating (see Appendix L). Ideally, the consequence rating would quantify the damages the culvert blockage is “responsible for”, considering each culvert in isolation. In practice, however, it is too simplistic to consider culverts in isolation along any creek drained by multiple culverts because the culverts behave as a system. For example, culvert blockage will be affected by any blockages upstream, and, in turn, a blockage will affect subsequent blockages downstream. Moreover, flow avulsions can also partially return to the stream channel, only to avulse again further downstream, while some portion of the flow also remains outside the stream channel. Avulsion paths from multiple culverts can also overlap, which makes it impossible to distinguish one flow path from another.

Given the above complexities, BGC made a simplifying assumption for baseline culvert risk estimation: all else being equal, culvert blockages higher in elevation in a creek system are likely to result in greater consequences, due to the proportionally greater number of buildings exposed to hazard downstream. Given this assumption, BGC assigned consequence levels to a given culvert based on the maximum, total consequences estimated downstream of the culvert. Although conservative, it represents an objective way to place an upper bound on consequences that can be consistently applied across all culverts. When combined with overall creek risk ratings, the approach represents a practical method to assign risk ratings supporting risk reduction prioritization.

BGC also estimated the level of risk reduction (residual risk) achieved by implementing the risk reduction measures described in Appendix O. Systematic re-modelling of hazard scenarios and re-analysis of risk with risk control measures was outside the scope of work. As such, BGC reduced either the hazard or consequence ratings, or both, based on two key assumptions. First, in most cases, the residual risk rating assumes that risk control measures, once implemented, will reduce the culvert overflow rating to an annual return period greater than 200 years. Where the proposed risk control measure would reduce consequences (e.g. by increasing the likelihood that flow avulsions, if occurring, re-enter the creek downstream of the road or otherwise reduce building damages), the consequence rating was reduced by 90% from the baseline estimate.

Table N.3-1. Economic Risk Matrix

Hazard Rating (Probability Hazard Scenario Occurs and Impacts Elements at Risk)			**Economic Risk Rating**				
Classification	**Hazard Scenario Probability**	**Culvert Overflow Rating (Years)**					
Very Low	0.001-0.0003	n/a	**1**	**1**	**2**	**3**	**4**
Low	0.003-0.001	>200	**1**	**2**	**3**	**4**	**5**
Moderate	0.01-0.003	200	**2**	**3**	**4**	**5**	**6**
High	0.03-0.01	50	**3**	**4**	**5**	**6**	**7**
Very High	0.1-0.03	20	**4**	**5**	**6**	**7**	**7**
Consequence Rating	**Indices**		Very Low	Low	Moderate	High	Very High
	Direct Damage Cost ($M)		<0.1	<0.5	0.5-1	1-10	>10

N.4. REFERENCES

BGC Engineering Inc. (BGC). 2014. Cougar Creek Debris Flood Risk Assessment. Report prepared for the Town of Canmore dated June 11, 2014.

BGC Engineering Inc. (BGC). 2015a. Harvie Heights Creek Debris-Flood Risk Assessment. Report prepared for the Town of Canmore dated May 27, 2015.

BGC Engineering Inc. (BGC). 2015b. Heart Creek Debris-Flood Risk Assessment. Report prepared for the Town of Canmore dated April 27, 2015. Report prepared for the Town of Canmore dated January 16, 2015.

BGC Engineering Inc. (BGC). 2015c. Stone Creek Debris-Flow Risk Assessment. Report prepared for the Town of Canmore dated January 13, 2015.

BGC Engineering Inc. (BGC). 2015d. Three Sisters Creek Debris Flood Risk Assessment. Report prepared for the Town of Canmore dated January 20, 2015.

BGC Engineering Inc. (BGC). 2015e. Exshaw Creek and Jura Creek Debris Flood Risk Assessment. Report prepared for the Municipal District of Bighorn dated March 1, 2015.

BGC Engineering Inc. (BGC), 2016. Stoneworks Creek Debris Flood Risk Assessment. Draft Report Prepared for the Town of Canmore dated May 17, 2016.

Jakob, M., Stein, D., and Ulmi, M., 2011. Vulnerability of buildings to debris flow impact. Natural Hazards, Vol. 60, Issue 2, pp. 241-261.

APPENDIX O
INDIVIDUAL CREEK SUMMARIES

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

DISTRICT OF NORTH VANCOUVER

DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT (PO #96726)

INDIVIDUAL CREEK SUMMARIES (APPENDIX O)

FINAL

PROJECT NO:	0404054	DISTRIBUTION:	
DATE:	May 31, 2017	CLIENT:	e-copy
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O.1. INTRODUCTION

O.1.1. Overview

This appendix summarizes the site-specific hazard, risk, and risk-control assessments completed for each creek, including the following for each creek:

- Hazard assessment – creek characteristics, peak discharge, and mobilized sediment volume at various return periods, and estimated frequency of overflow at culverts.
- Risk assessment – estimation of potential direct damages associated with various return period hazard scenarios.
- Risk control assessment – conceptual-level design options, costs, and potential risk reduction that could be achieved at identified sites that do not currently meet DNV's safety risk tolerance criteria or DNV's design standard of passing the 200-year instantaneous flow.

This introductory chapter provides context and definitions needed to understand the site-specific information that is presented in subsequent chapters, which are organized by creek. This appendix is intended to be read along with the general description of assessment methods and risk control options presented in the main report, and with reference to the information presented on DNVHIT. BGC also notes that the data shown on DNVHIT and provided in database format (Appendix F) could be presented in additional formats that are outside the current scope of work, but that could be completed on request.

O.1.2. Hazard Assessment

The hazard assessment summarizes creek characteristics that were used to estimate the frequency of flood overflow at culverts and to model flood flows.

Peak discharge estimates presented throughout this appendix were developed using regional flood frequency analysis (FFA), based on gauged local watersheds (Appendix E). As noted in the main report, NHC is concurrently working on an updated hydrologic analysis, the results of which are not yet available. Debris flood sediment volume estimates were developed as outlined in Table 4-2 of the main report and in detail in Appendix G. Modelling results and hazard intensity maps are provided in Appendix M. The following definitions are needed to understand the hazard assessment summaries:

- Development Interface - All assessed creeks originate in an undeveloped watershed and flow into developed areas. The upstream-most point at which the main creek stem crosses a road is considered the development interface. The assessment assumes that sediment and debris originate primarily from the above the development interface.
- Controlling reach – The controlling reach is the creek reach immediately upstream of the development interface that delivers debris to developed areas. The controlling reach length is governed by the creek characteristics and is typically 50 m to 200 m in length.
- Critical grainsize - The largest estimated grainsize mobilized during the 200-year peak discharge. The sediment volume that arrives at the development interface will be

comprised of sand, gravel, and cobble-sized particles up to the critical grainsize. The estimated sediment volume is highly sensitive to the critical grainsize value.

Table O.1-1 summarizes key characteristics of each creek that were estimated by the hazard assessment. It is a reference for comparing the hazard characteristics of the different creeks.

Table O.1-1. Tabulation of study creek hazard characteristics.

Creek	Location	Process	Above development [1]		Controlling reach [2]		Peak discharge estimates by return period [3] (m^3/s)					Sediment volume estimates by return period [4] (m^3)				
			Watershed area (km^2)	Average slope (m/m)	Slope (m/m)	Critical grainsize (m)	10 - 30 years Q_{20}	30 - 100 years Q_{50}	100 - 300 years Q_{200}	300 - 1000 years Q_{500}	1000 - 3000 years Q_{2500}	10 - 30 years V_{20}	30 - 100 years V_{50}	100 - 300 years V_{200}	300 - 1000 years V_{500}	1000 - 3000 years V_{2500}
Mackay [5]	West of Lynn	Debris flow	0.78	0.48	N/A	N/A	3.4	4.2	5.8	-	-	-	500	5,000	-	-
Mackay (east)	West of Lynn	Debris flow	0.58	-	N/A	N/A	2.5	3.1	4.3	-	-	-	-	-	-	-
Mosquito [6]	West of Lynn	Debris flood	4.74	0.17	-	-	18	20	120	310	600	-	-	10,000	-	-
Mission	West of Lynn	Debris flood	0.28	0.29	0.31	0.28	1.2	1.5	2.1	-	-	<10	200	500	-	-
Thain	West of Lynn	Debris flood	0.51	0.25	0.11	0.30	2.2	2.8	3.8	-	-	<10	70	75	-	-
Hastings	West of Lynn	Debris flood	0.35	0.25	0.12	0.27	1.5	1.9	2.6	-	-	<5	10	220	-	-
Dyer	West of Lynn	Debris flood	0.76	0.21	0.14	0.30	3.3	4.1	5.6	-	-	<10	40	600	-	-
Kilmer	West of Lynn	Debris flood	0.77	0.20	0.19	0.31	3.3	4.2	5.7	-	-	<10	345	600	-	-
Coleman	West of Lynn	Flood	N/A	N/A	-	-	-	-	-	-	-	-	-	-	-	-
Thames	West of Lynn	Debris flood	0.53	0.23	0.23	0.33	2.2	2.8	3.9	-	-	<10	185	930	-	-
Canyon	East of Lynn	Flood	0.72	0.10	0.04	0.10	2.3	2.9	4.0	-	-	-	-	-	-	-
McCartney	East of Lynn	Flood	1.57	0.11	0.04	0.15	5.3	6.6	8.8	-	-	-	-	-	-	-
Taylor [7]	East of Lynn	Flood	0.59	0.15	0.12	0.20	1.9	2.4	3.3	-	-	-	-	-	-	-
Gallant	Deep Cove	Debris flood	1.14	0.16	0.14	0.33	4.1	5.1	6.9	-	-	<10	150	750	-	-
Panorama	Deep Cove	Flood	0.70	0.24	0.34	0.39	2.5	3.2	4.3	-	-	-	-	-	-	-
Kai	Deep Cove	Flood	0.06	0.33	0.23	0.10	0.2	0.2	0.3	-	-	-	-	-	-	-
Mathews Brook	Deep Cove	Debris flood	0.30	0.28	0.27	0.24	1.1	1.3	1.9	-	-	<10	160	440	-	-
Gavles	Deep Cove	Debris flood	0.56	0.27	0.27	0.35	2.0	2.6	3.5	-	-	<10	190	675	-	-
Cove	Deep Cove	Flood	0.46	0.27	0.24	0.44	1.7	2.1	2.9	-	-	-	-	-	-	-
Cleopatra	Deep Cove	Debris flood	0.24	0.26	0.40	0.29	1.1	1.5	1.9	-	-	<10	<10	200	-	-
Martin	Deep Cove	Flood	0.16	0.25	0.41	0.22	0.4	0.6	0.8	-	-	-	-	-	-	-
Francis	Deep Cove	Debris flood	1.72	0.26	0.60	0.58	7.2	8.9	12.0	-	-	120	150	190	-	-
Ward	Indian Arm	Flood	0.16	0.24	0.24	0.25	0.5	0.6	0.9	-	-	-	-	-	-	-

[1] Above development characteristics calculated based on 2013 LiDAR provided by the District of North Vancouver in June 2015 supplemented with field observations and information from previous assessments.
[2] The controlling reach is the creek reach immediately upstream of the development interface. The controlling reach length is governed by the creek characteristics and is typically 50-200 m.
[3] Peak discharge estimates are preliminary and will be updated based on information to be provided by nhc. This is the peak discharge at the development interface. Peak discharge values do not include an allowance for climate change or other uncertainties.
[4] Sediment volumes were estimated based on the methodology outlined in Appendices G, H and K for debris flood prone creeks, Percy Creek and Indian Arm Creeks, respectively. Reported sediment volumes are the best estimate of sediment volume. A range of volumes is possible.
[5] Mackay Creek estimated debris flow volumes are based on BGC's (2014a) assessment of the debris flow frequency-magnitude.
[6] Mosquito Creek was investigated as part of separate scope of work (BGC, 2013); the risk assessment relied on data from these investigations.
[7] The peak flow estimates for Taylor Creek are reported at Anne MacDonald Way. The additional watershed area to Mt Seymour Pkwy increases the peak flow estimates to 5.5 m^3/s, 6.8 m^3/s, and 9.2 m^3/s for the Q_{20}, Q_{50}, and Q_{200}, respectively.
[8] Indian Arm creeks east of Allan Creek, excepting Percy Creek were not hiked as part of the current assessment due to existing hazard assessments. The controlling reach characteristics from those creeks were therefore not included. The peak flow estimates for Indian Arm flood prone creeks are based on the regional analysis as outlined in Appendix E. The peak flow estimates for Indian Arm debris flood prone creeks are based on the regional analysis and bulked by a factor of 3 (Jakob and Jordan, 2001). On the debris flow prone creeks, the peak flows are based on the upper end of the return period range. As a conservative approach, the sediment volumes for all Indian Arm creeks are based on the upper end of the return period range.
[9] There is uncertainty in the sediment volume estimate at Scott-Goldie Creek due to the small size of the fan, see Appendix K for more details.

Creek	Location	Process	Above development [1]		Controlling reach [2]		Peak discharge estimates by return period [3] (m^3/s)					Sediment volume estimates by return period [4] (m^3)				
			Watershed area (km^2)	Average slope (m/m)	Slope (m/m)	Critical grainsize (m)	10 - 30 years	30 - 100 years	100 - 300 years	300 - 1000 years	1000 - 3000 years	10 - 30 years	30 - 100 years	100 - 300 years	300 - 1000 years	1000 - 3000 years
							Q_{20}	Q_{50}	Q_{200}	Q_{500}	Q_{2500}	V_{20}	V_{50}	V_{200}	V_{500}	V_{2500}
Unnamed		Flood	0.19	0.32	0.24	0.25	0.6	0.7	1.0	-	-	-	-	-	-	-
Ostler		Debris flood	0.83	0.27	0.31	0.65	3.1	3.9	5.3	-	-	<5	<10	790	-	-
Allan		Debris flood	1.10	0.31	0.33	0.40	4.7	5.8	7.9	-	-	30	240	620	-	-
Sunshine [8]		Debris flood	1.15	0.32	-	-	4.1	5.2	7.0	-	-	90	120	140	170	190
Scott-Goldie [8, 9]		Debris flood	2.96	0.20	Not applicable to debris flow creeks		15	54	72	85	-	-	-	3,100	3,100	-
Percy		Debris flow	1.99	0.24			10	110	200	250	310	-	5,000	10,000	12,000	16,000
Vapour [8]		Debris flood / Debris flow	0.62	0.37	-	-	7	9	13	60	-	190	250	310	2,600	-
Gardner Brook [8]		Debris flood	0.58	0.27	-	-	5	7	9	160	-	620	810	980	1,180	-
Shone [8]		Debris flow	2.73	0.22	-	-	13	180	270	370	-	-	9,000	16,000	23,000	-
Underhill [8]		Debris flow	0.27	0.47	N/A		1.1	1.4	1.9	-	-	-	-	-	-	-
Ragland [8]		Flood	0.37	0.42	-	-	1.2	1.6	2.2	-	-	-	-	-	-	
Holmden [8]		Debris flow	2.03	0.32	N/A		20	70	90	120	-	600	2,000	4,000	6,000	-
Coldwell [8]		Debris flood	4.65	0.24	-	-	68	84	111	130	-	300	400	500	600	-
Friar [8]		Debris flow	0.43	0.43	N/A		5	15	20	30	-	100	500	800	1,000	-
Clegg [8]		Debris flow	-	-	-	-	50	170	260	350	-	2,000	9,000	15,000	21,000	-

1 Above development characteristics calculated based on 2013 LiDAR provided by the District of North Vancouver in June 2015 supplemented with field observations and information from previous assessments.

2 The controlling reach is the creek reach immediately upstream of the development interface. The controlling reach length is governed by the creek characteristics and is typically 50-200 m.

3 Peak discharge estimates are preliminary and will be updated based on information to be provided by nhc. This is the peak discharge at the development interface. Peak discharge values do not include an allowance for climate change or other uncertainties.

4 Sediment volumes were estimated based on the methodology outlined in Appendices G, H and K for debris flood prone creeks, Percy Creek and Indian Arm Creeks, respectively. Debris flood prone creek sediment volumes are reported herein as ranges. The best estimate sediment volume for each creek is reported in the corresponding individual creek hazard summary.

5 Mackay Creek estimated debris flow volumes are based on BGC's (2014a) assessment of the debris flow frequency-magnitude.

6 Mosquito Creek was investigated as part of separate scope of work (BGC, 2013); the risk assessment relied on data from these investigations.

7 The peak flow estimates for Taylor Creek are reported at Anne MacDonald Way. The additional watershed area to Mt Seymour Pkwy increases the peak flow estimates to 5.5 m^3/s, 6.8 m^3/s, and 9.2 m^3/s for the Q_{20}, Q_{50}, and Q_{200}, respectively.

8 Indian Arm creeks east of Allan Creek, excepting Percy Creek were not hiked as part of the current assessment due to existing hazard assessments. The controlling reach characteristics from those creeks were therefore not included. The peak flow estimates for Indian Arm flood prone creeks are based on the regional analysis as outlined in Appendix E. The peak flow estimates for Indian Arm debris flood prone creeks are based on the regional analysis and bulked by a factor of 3 (Jakob and Jordan, 2001). On the debris flow prone creeks, the peak flows are based on the upper end of the return period range. As a conservative approach, the sediment volumes for all Indian Arm creeks are based on the upper end of the return period range.

9 There is uncertainty in the sediment volume estimate at Scott-Goldie Creek due to the small size of the fan, see Appendix K for more details.

O.1.3. Risk Assessment

BGC estimated risk from two different perspectives: each creek system as a whole, and for individual culverts or storm water mains. The creek-level estimate supports risk reduction prioritization for each creek (e.g., should creek "X" be higher priority, from a risk perspective, than creek "Y"). The latter supports prioritization of individual risk control measures (e.g., should culvert "X" be higher priority, from a risk perspective, than culvert "Y").

Section 6.4 of the main body of the report summarizes the creek-level risk results for urban creeks, including identification of buildings that exceed DNV's individual risk tolerance thresholds, and a ranking of study creeks in order of estimated annualized direct damage costs. This section should be referenced to support district-wide prioritization of creeks for risk reduction planning.

The risk assessment results in this appendix are separated for individual creeks and include the following:

- Direct building damage costs for each hazard scenario and an annualized cost considering all scenarios together
- Safety risk estimates for parcels identified as exceeding DNV individual risk tolerance thresholds (if any)
- Economic risk ratings for each culvert or stormwater main listed in the risk control summaries
- Residual risk ratings for each culvert or stormwater main listed in the risk control summaries, assuming implementation of the recommended risk control measures.

Creek-level risk results are provided for creeks with identified hazard scenarios. This includes all creeks except for Coleman, Canyon, McCartney, Kai, Cove, and Martin Creeks, which were estimated to have limited to no potential for flow avulsion at culverts resulting in downstream impacts to infrastructure. In addition, no hazard scenarios were modelled for Thain Creek due to the sediment storage available at Prospect Road.

To assist with risk-based prioritization of individual stormwater management assets, economic risk ratings were assigned to all in-scope culverts, including those without a modelled hazard scenario. In these cases, the consequence rating was assigned based on judgement and inspection of consequence rating at adjacent sites, and typically results in a low risk rating.

O.1.4. Risk Control Assessment

O.1.4.1. Site-Specific Risk Control Design Basis

The site-specific information provided in this appendix summarizes our current understanding of possible risk control options, and supports further assessment and risk control design by DNV. Site-specific information provided herein is not intended to be the sole basis for final risk control design. In all cases, further work will be needed to assess and complete final design of risk control at each creek.

Table O.1-2 summarizes the basis of the risk control assessments presented in this appendix. The items listed describe the constraints of the current design stage, and were used to weigh and compare technically feasible risk control options.

The assessment assumes that each culvert and drainage element in the upper watershed (e.g., Mountain Highway, Thames Creek watershed) maintains its current function. Potential consequences of making changes to the upper watershed surface water drainage infrastructure should be assessed prior to making such changes. A program that inspects and reviews surface water drainage elements in the upper watersheds is recommended.

Table O.1-2. Site-specific risk control design basis summary.

Item		Design Notes
1	Conceptual Design Level	Conceptual-level designs are presented. The intent of the conceptual design stage is to present technically feasible design options that are capable of meeting the project design criteria and risk reduction targets. Details of the design options, including final dimensioning and layout of design elements and budgetary level cost estimates, are beyond the scope of this conceptual design phase.
2	Risk Reduction Target	Except where safety risk is intolerable, designs are dimensioned for the 200-year instantaneous flow and sediment volume. For this assessment, it is assumed that economic risks associated with larger flows are tolerable.
3	Risk Transfer	Creek system design approach has been used in which functional chains of risk control elements along the creek are considered together in the options assessments. The functional chain of risk control elements seeks to avoid risk transfer to any individual home or building.
4	Hydrogeomorphic Process Type	Designs are intended to address the dominant hydrogeomorphic process type (e.g., debris flood, debris flow) at each creek. See Table 1-2 of the main report for dominant process types at each creek.
5	Design Life	Short-term measures are proposed that reduce the consequences of flooding, but that may not meet DNV's risk reduction target. Short-term measures tend to be lower cost and can be implemented relatively quickly. Short-term measures are intended to be replaced with long-term measures that meet DNV's risk reduction targets.
6	Site Access and Maintenance	It is assumed that existing roads can be used for permanent access, and that construction of new access roads is possible, but undesirable. It is assumed that at least annual maintenance will occur, including sediment removal with a backhoe. Options that minimize maintenance are typically preferred.
7	Cost Estimates	Costs are order of magnitude estimates intended for option comparison.
8	Social & Environmental Impacts	Social and environmental impacts have been considered qualitatively in options assessments. More complete assessment of impacts is typically needed during future design stages.
9	Design Confidence	Confidence that designs will perform as intended has been assessed qualitatively. Functional chains of risk control elements are typically proposed to increase redundancy where performance of risk control measures is uncertain.

Item		Design Notes
10	Geotechnical & Topographic Parameters	Design parameters are assumed based on surface observations only. Generally, it is assumed that design elements are founded on very dense till or coarse-grained soil. Bedrock and the water table are assumed to be below, and not interacting with design elements.
11	Priority Order	A priority order is given for proposed long-term risk control measures. Priority order was assigned based on BGC's interpretation of the total risk reduction achieved by the design element relative to other elements along the same creek. Priority level "1" is considered highest priority for implementation by DNV.

O.1.4.2. Risk Reduction Target

DNV has established risk reduction targets for safety risk, but have not established economic risk reduction targets. BGC estimates that the safety risk threshold for both risk to individuals and groups is exceeded at Percy Creek. Risk control designs options presented for Percy Creek are intended to reduce the safety risk to a tolerable level. At all other creeks, the 200-year event has been adopted as the 'design event' for this stage of design, which is consistent with DNV bylaws[10]. This decision implies that DNV considers the economic risk associated with larger events to be tolerable. DNV should review this assumption during future design stages, and may wish to select an economic risk threshold in terms of tolerable economic impact or tolerable Economic Risk Rating (see Table O.1-3). The economic risk threshold could be used to select and justify a 'design event' that is greater magnitude than the 200-year event referenced for this conceptual design. The general design concepts presented herein are expected to be valid for greater magnitude events, although potentially at an increased cost.

The 'current risk' and 'residual risk' referenced for individual assets and risk control elements refer to the economic risk matrix shown in Table O.1-3. 'Current risk' is an assessment of risk associated with each asset or design element at the time this report was written. 'Residual risk' describes the estimated risk that would remain after the proposed risk control elements are installed. The 'residual risk' estimates consider the effects of risk control elements installed at the asset location, as well as elements installed upstream of the asset.

The economic risk matrix considers a 200-year return period event to be representative of a "Moderate" likelihood event (100 to 300-year return period; Table O.1-3). Risk control measures that are designed to mitigate the 200-year event may be overwhelmed by higher magnitude, "Low" likelihood events (e.g., 300 to 1000-year return period). Therefore, the maximum residual risk rating is L5 at sites designed to pass the 200-year event (Low Hazard Rating, Very High Consequence). This implies that L5 economic risk rating is tolerable to DNV.

[10] This choice of design event is subject to confirmation following completion of the safety risk assessment for all urban debris flood and flood creeks.

Table O.1-3. Economic risk matrix.

Hazard Rating (Probability Hazard Scenario Occurs and Impacts Elements at Risk)			Economic Risk Rating				
Classification	**Return Period (Years)**	**Annual Probability**					
Very Low	1000-3000	0.001-0.0003	1	1	2	3	4
Low	300-1000	0.003-0.001	1	2	3	4	5
Moderate	100-300	0.01-0.003	2	3	4	5	6
High	30-100	0.03-0.01	3	4	5	6	7
Very High	10-30	0.1-0.03	4	5	6	7	7
Consequence Rating	**Indices**		Very Low	Low	Moderate	High	Very High
	Direct Damage Cost ($M)		<0.1	<0.5	0.5-1	1-10	>10

O.1.4.3. Risk Control Options

Risk control options were developed with consideration of each creek as a system, and proposed risk control elements are intended to operate as a functional chain of elements. The effect of risk controls elements on downstream infrastructure should be considered when selecting the order in which elements are designed and constructed. The priority rating ('1' is highest priority) provided for the long-term risk control concepts indicate BGC's interpretation of which risk control elements are most important for creek system risk reduction.

The complete combination of risk control elements provided at each creek is considered to be the "preferred" option proposed by BGC. Where appropriate, multiple options are provided that fulfill a similar function (for example, sediment capture and channel stabilization can be competing options for sediment control). In general, creek system designs that contain multiple, small-scale elements that offer redundancy are preferred (e.g., series of small basins and check dams) over a single large-scale risk control element (e.g., single large debris basin). Small-scale elements are also preferred because the typically steep, narrow channel width, and difficult site access for creeks in the DNV often preclude large structures.

O.1.4.4. Short-Term and Long-Term Risk Control

Site-specific risk control options are organized in terms of "Short-term" measures that can be implemented as soon as possible, and "Long-term" measures that meet DNV's risk reduction target, but are often more costly or disruptive to implement. Limited resources and funding may prevent DNV from immediately implementing long-term risk control measures at all sites. Short-term measures can often be implemented within budget and resource limits to marginally reduce risks in the interim period until a long-term measure is implemented.

O.1.4.5. Conceptual-Level Cost Estimates

Cost estimates for each risk control design option have been estimated to an order of magnitude and are described qualitatively (Table O.1-4). The estimates are for construction costs, including engineering design, material supply, and installation. These costs were developed primarily to support a comparison between options, and should not be used to set final budgets for risk control works. More detailed cost estimates should be developed for the selected risk control option during subsequent design and assessment phases.

The qualitative cost indicator that was assigned to each proposed risk control element was selected based on the example costs compiled in Table O.1-5.

Table O.1-4. Cost indicator definition of terms.

Term	Approximate Capital Cost ($)	Approximate Annual Maintenance Effort
Low (L)	Less than $100,000	Annual inspection and debris removal by hand
Moderate (M)	$100,000 to $500,000	Frequent inspection and debris removal by hand
High (H)	$500,000 to $1 Million	Annual sediment removal with a backhoe
Very High (VH)	Greater than $1 Million	Frequent sediment removal with a backhoe or annual removal of large volumes of sediment

Note: Capital cost is the cost to design and construct the risk control option, and does not include annual maintenance costs.

Table O.1-5. Example cost estimates for risk control elements.

Element	Unit	Dimensions	Cost Estimate	Source
Channel Stabilization	Per check dam	H: 1.5 m W:10 m	$15k	Contractor estimate, 2015
Debris Capture	Per debris basin	1000 m^3	$500k	KWL estimate, 2015, Kilmer
		3000 m^3	$700k	BGC estimate, 2013, Lions Bay
		4000 m^3	$1.4M	BGC estimate, 2013, Lions Bay
Culvert Inlet Debris Control	Per debris barrier	H: 1 m W: 8 m	$30k	DNV contractor bids, 2015
Trash rack modifications or replacement	Per culvert inlet area	0.5 m headwall height addition; 10 m^2 screen	$40k	DNV contractor bids, 2015
		New construction, 12 m^2 screen	$350k	BGC estimate, 2015, Exshaw Creek Debris Barrier, Alberta
Replace Culvert	Per culvert	D: 900 mm L: 15 m	$200k	DNV estimate, 2014, Asset Management Plan
		D: 3050 mm L: 20 m	$500k	
Improve hydraulics (headwall modifications)	Per headwall	0.5 m headwall height addition	$30k	DNV contractor bids, 2015
Channel modifications and erosion protection	Per channel	H: 1.5 m L:40 m	$100k	DNV contractor bids, 2015, Gallant
Swale across road	Per swale	W: 20 m L: 20 m	$150k	BGC estimate, 2014, Mackay
Berm to divert shallow flow	Per berm	H: 1 m L: 40 m	$70k	BGC estimate, 2015, Harvie Heights, Alberta
		H: 1-2 m L: 600 m	$360k	

Notes: Conceptual cost estimate for design and construction. Where individual elements are extracted from previous cost estimates or contractor bids, a 50% contingency has been added to account for engineering, permitting, and contractor mobilization.

O.1.4.6. Design Methods and Assumptions

The following items describe specific design methods and assumptions that were used in preparing the conceptual designs and assessing residual risk:

- Culvert replacement – Residual risk estimates assume that replacement culverts are installed as near as possible to the channel gradient and alignment, have appropriate headwalls and culvert inlet debris protection, and include additional capacity that accounts for expected sediment deposition within the culvert and climate change.
- Check dams – Check dams are assumed to be constructed from log cribs. The number of check dams shown in the conceptual design is approximate and requires review during detailed design. The location, number, spacing, and extent of check dams in the conceptual design is based on the suitability of the terrain for check dam installation, sediment volume that can be stored at each dam, and the critical channel gradient that results in sediment mobilization. The number and extent of check dams needed depends heavily on if sediment basins are also constructed.
- Sediment basins – Sediment basins are sized to store the expected mobilized sediment volume for the 200-year flood event. The proposed locations consider ease of access and channel geometry, favoring wide, shallow-gradient locations near existing roads.

O.8. THAMES CREEK SUMMARY

O.8.1. Thames Creek: Hazard Summary

Thames Creek is a debris-flood prone creek located west of Lynn Creek (Table O.8-1). Figure O.8-1 shows the assessed culverts and stormwater mains along Thames Creek. Each "site" represents a location where one or more culverts exist.

On November 3, 2014, a storm event resulted in blockage at McNair Road and Kilmer Road (site B and G in Figure O.8-1) resulting in overland flooding and property damage to homes on Ramsey Road and the home immediately downstream of Kilmer Road. Following the storm event, the DNV constructed risk control measures along Thames Creek in 2015, including:

- Debris control structure upstream of Mountain Highway (site A)
- Headwall modifications and debris control structure upstream of McNair Drive (site B)
- Trash rack and debris control structures upstream of Kilmer Rd (site G).

Photographs of the pre-construction and post-construction condition at each site are accessible on DNVHIT. The recommendations and findings outlined herein reflect the sites in the condition at the time of writing including the newly constructed debris barriers. More details on the November 2014 storm event, which was used to calibrate BGC's hazard analysis, are included in Appendix A.

The frequency-magnitude relationship for Thames Creek is shown in Table O.8-2. The reported peak flows and sediment volumes are estimated at the development interface at McNair Drive (site B). Downstream of the development interface, BGC assessed culverts and stormwater mains to identify the locations where flow is likely to avulse during a storm event. Details about each culvert are outlined in Table O.8-3 along with the estimated frequency of overflow.

Based on the information in Table O.8-3, BGC developed and modeled flows associated with two representative hazard scenarios (Appendix M), including:

- Blockage at Mountain Hwy (site A) during a 200-year return period event;
- Blockage at McNair Rd (site B) and Kilmer Rd (site G) during a 50-year return period event.

Table O.8-1. Thames Creek – Summary of creek characteristics.

Creek	Location	Process	Above development characteristics		Controlling reach characteristics	
			Watershed area (km^2)	Average slope (%)	Slope (%)	Critical grainsize (mm)
Thames	West of Lynn	Debris flood	0.53	23	23	330

Table O.8-2. Thames Creek – Frequency-magnitude estimates of peak flow and sediment volume at the development interface (site B – McNair Rd).

Return Period (years)	Flood Quantile	Peak Discharge (m^3/s)	Sediment Volume (m^3)
10-30	Q_{20}	2.2	<10
30-100	Q_{50}	2.8	185
100-300	Q_{200}	3.9	930

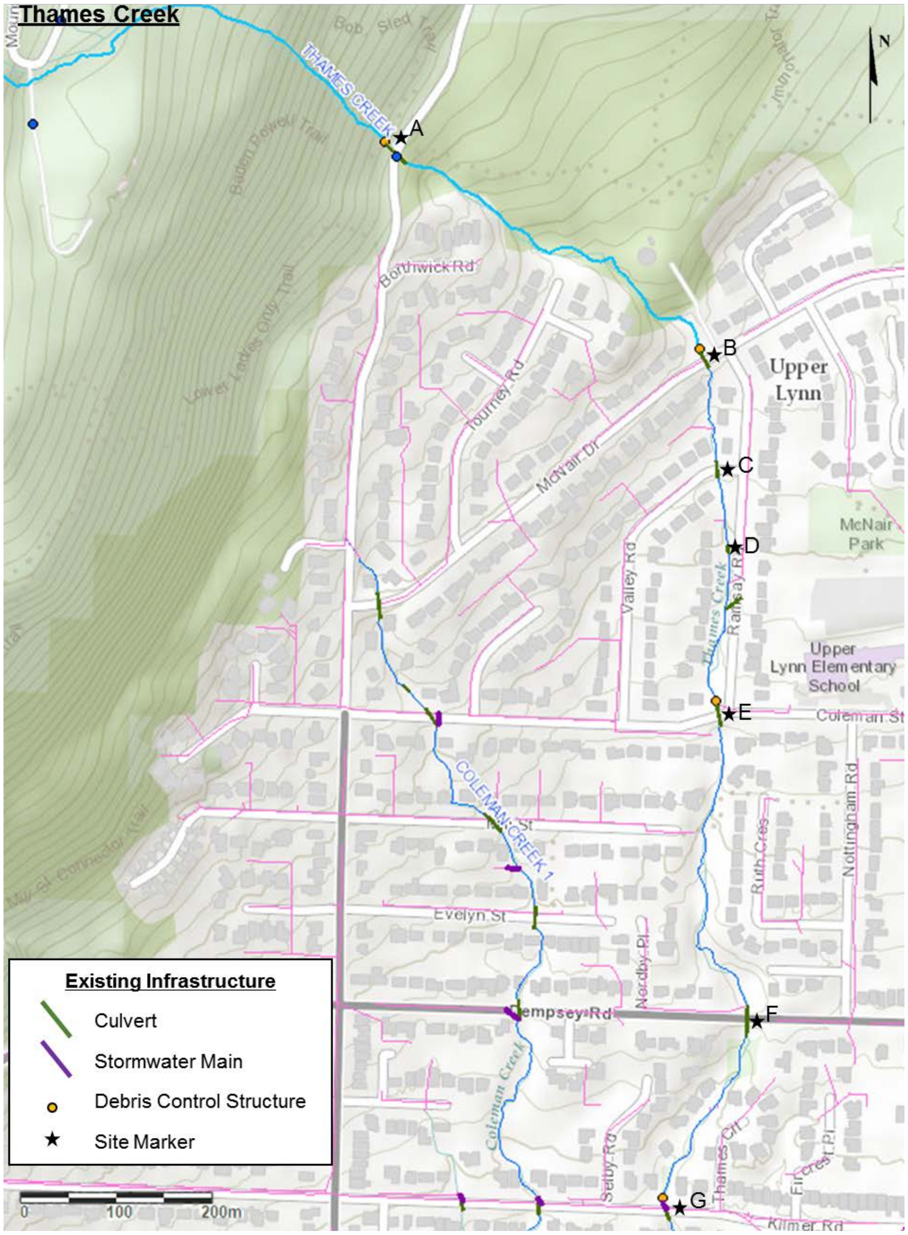

Figure O.8-1. Thames Creek – Culverts and stormwater mains.

Table O.8-3. Thames Creek – Summary of assessed culverts and stormwater mains.

Site	Asset ID	Description	Location	Meets 200-year Design Requirement[1]	Reason Does Not Meet 200-year Design Requirement	Overflow Return Period (years)	Risk Rating
A	STMCUL00052	Box Culvert	Mountain Hwy	No	Sediment and debris deposition, flat culvert inlet	50	6
	BGCSTMCUL00074	Round Culvert		No	Sediment and debris deposition, flat culvert inlet	50	6
B	STMCUL00152	Round Culvert (x2)	McNair Dr	No	Sediment and debris deposition, flat culvert inlet, flat culvert, undersized culvert	50	6
C	STMCUL00160	Box Culvert	Valley Rd	Yes	Meets Requirement	>200	4
D	STMCUL00154	Arch Culvert	4605 Ramsay Rd	Yes	Meets Requirement	>200	4
E	STMCUL00159	Box Culvert	Coleman St	Yes	Meets Requirement	>200	4
F	STMCUL00409	Round Culvert	Dempsey Rd	No[2]	Large woody debris, undersized culvert, flat culvert, sediment and debris deposition	50[2]	4
G	STMMN09158	Round Storm Main	Kilmer Rd	No	Undersized trash rack (approx. 3 times culvert area), sediment and debris deposition	50	4
	STMCUL00412	Box Culvert		No	Undersized trash rack (approx. 3 times culvert area), sediment and debris deposition	50	4

Notes:

1. DNV's Development Servicing Bylaw No. 7388, Schedule D.1, Section C9.5 specifies that all culverts be designed to convey the 200-year instantaneous flow with the design headwater not exceeding the top of culvert.
2. DNV's Development Servicing Bylaw No. 7388, Schedule D.1, Section C9.5 specifies that all culverts be designed to convey the 200-year instantaneous flow with the design headwater not exceeding the top of culvert. STMCUL00409 does not have sufficient capacity to pass the 20-year flow with the headwater at the top of the culvert; however, the actual available headwater depth is about 3 times the culvert diameter, which results in sufficient capacity to pass the 200-year flow. In this case, the overflow return period is controlled by the potential for large woody debris.

O.8.2. Thames Creek: Risk Summary

Table O.8-4 lists estimated direct building damage costs as an annualized figure and separately for each hazard scenario.

Table O.8-4. Thames Creek – Damage costs by scenario.

Creek	Scenario (Appendix M Drawing Number)	Hazard Scenarios Assessed (Annual Return Period Range)	Total # Homes In Hazard Zones	"Effective" Number of Buildings Impacted[1]	"Effective" Total Damage Cost[2]	Annualized Damage[2]
Thames Creek	15 (M-15)	100-300	150	75	$4,900,000	$79,000
	16 (M-16)	30-100	43	22	$1,300,000	

Notes:
1. Count of buildings in impact zones assuming 50% chance of building impact where intensity ≤1.
2. Values are rounded to the nearest $100 or $1,000 if exceeding $1k or $10k respectively.

There were no buildings on Thames Creek where BGC's best estimate of debris flood risk exceeded DNV risk tolerance thresholds of either 1:100,000 or 1:10,000 annual risk to life.

O.8.3. Thames Creek: Risk Control Summary

This section describes proposed short-term and long-term risk control options for Thames Creek:

- Short-term risk control: Table O.8-5, Table O.8-6, and Figure O.8-2.
- Long-term risk control: Table O.8-7, Table O.8-8, Figure O.8-3, and Figure O.8-4.

BGC is currently completing detailed design of risk control measures on Kilmer Creek. Once completed, the results of the detailed study will supersede some aspects of this risk control summary.

Table O.8-5. Thames Creek – Short-term risk control elements and costs.

Risk Control Elements	Description	Capital Cost	Annual Maintenance Effort
Routine maintenance	Routinely excavate and remove sediment and debris from all culvert inlet areas with a backhoe.	-	H
Emergency response	Prepare emergency response plans to implement during forecasted high flow events.	-	-
Return flow to channel	Create swale in dirt road at site A (Mountain Highway) to direct excess flow back into channel.	L - M	L

Table O.8-6. Thames Creek – Short-term risk reduction.

Site	Asset ID	Elements contributing to risk reduction at each asset[1]	Current Risk	Residual Risk
A	STMCUL00052	At Asset: Create swale in dirt road at Mountain Highway to direct excess flow back into channel, routine maintenance	6	4
	BGCSTMCUL00074		6	4
B	STMCUL00152	At Asset: Emergency response, routine maintenance	6	6
C	STMCUL00160	At Asset: Routine maintenance	4	4
D	STMCUL00154	At Asset: Routine maintenance	4	4
E	STMCUL00159	At Asset: Routine maintenance	4	4
F	STMCUL00409	At Asset: Routine maintenance	4	4
G	STMMN09158	At Asset: Emergency response, routine maintenance	4	4
	STMCUL00412	At Asset: Emergency response, routine maintenance	4	4

Note:

1. Routine maintenance contributes to risk reduction at each asset. The assigned residual risk rating assumes that routine inspection and maintenance is carried out at each asset.

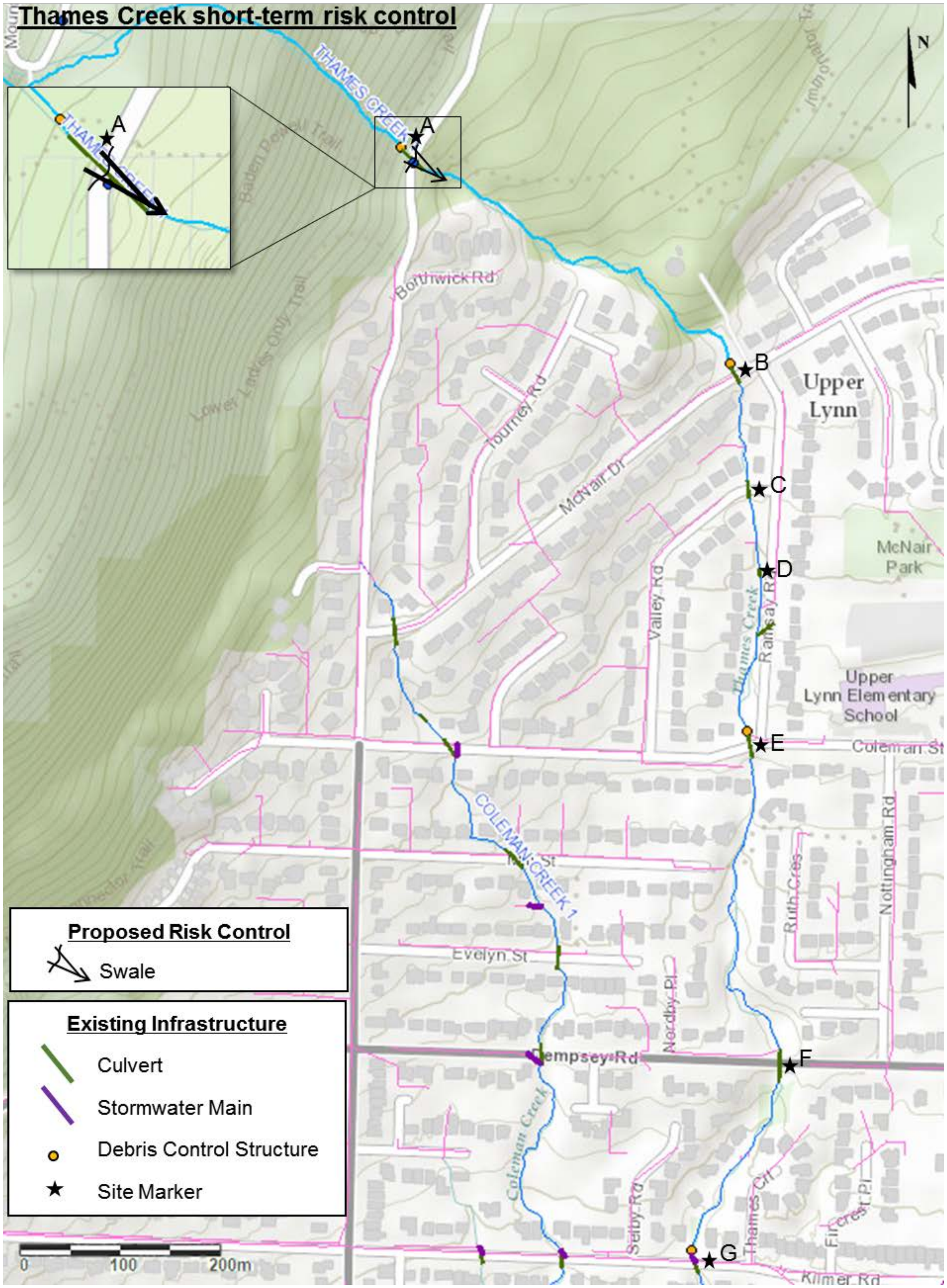

Figure O.8-2. Thames Creek – Short-term risk control.

Table O.8-7. Thames Creek – Long-term risk control elements and costs.

Risk Control Elements	Description	Capital Cost	Annual Maintenance Effort	Priority Order
Return flow to channel	Berm in upper watershed to prevent avulsion of Kilmer Creek into Thames Creek; 1 to 2 m tall, and approx. 40 m long. Maintenance of surface water drainage elements at Mountain Highway, upper watershed.	L	L	1
Channel Stabilization	Log-crib check dams upstream of development; 1.5 to 2 m tall, 15 to 20 m wide, 15 to 20 m spacing.	L	L	2
Return flow to channel	Create swale in dirt road at Mountain Highway to direct excess flow back into channel.	L-M	L	1
Debris Capture	Sediment basin upstream of site B with 500 to 1000 m^3 storage capacity.	M-H	VH	1
Increase culvert size	Replace existing culverts at site B with a large box culvert.	M	L	2
	Replace existing culvert at site F with a large box culvert[1].	M	L	3[1]
Trash rack modifications	Replace trash rack at site G. Screening area should be at least 30 m^2 (double the size of current trash rack). Concrete headwall modifications will need to be made to accommodate the larger trash rack.	M	H	2
Debris control structure	Debris control structure (posts) upstream of culvert at site F to protect against large woody debris. Set top of debris control structure equal to or greater than the elevation of the crown of the culvert at the inlet.	L	L	2
Routine maintenance	Routinely excavate and remove sediment and debris from all culvert inlet areas and sediment basins with a backhoe, and routinely inspect log-crib check dams.	-	H	1

Note:

1. DNV's Development Servicing Bylaw No. 7388, Schedule D.1, Section C9.5 specifies that all culverts be designed to convey the 200-year instantaneous flow with the design headwater not exceeding the top of culvert. STMCUL00409 does not have sufficient capacity to pass the 20-year flow with the headwater at the top of the culvert; however, the actual available headwater depth is about 3 times the culvert diameter, which results in sufficient capacity to pass the 200-year flow. Therefore, the culvert should eventually be replaced in order meet the DNV bylaw; however, it is considered low priority.

Table O.8-8. Thames Creek – Long-term risk reduction.

Site	Asset ID	Elements contributing to risk reduction at each asset[1]	Current Risk	Residual Risk
A	STMCUL00052	At Asset: Create swale in dirt road at Mountain Highway to direct excess flow back into channel Upstream: Log-crib check dams	6	4
	BGCSTMCUL00074		6	4
B	STMCUL00152	At Asset: Replace culvert Upstream: Sediment basin, log-crib check dams	6	4
C	STMCUL00160	At Asset: None Upstream: Sediment basin, log-crib check dams	4	4
D	STMCUL00154	At Asset: None Upstream: Sediment basin, log-crib check dams	4	4
E	STMCUL00159	At Asset: None Upstream: Sediment basin, log-crib check dams	4	4

Site	Asset ID	Elements contributing to risk reduction at each asset[1]	Current Risk	Residual Risk
F	STMCUL00409	At Asset: Replace culvert, debris control structure (posts) Upstream: Sediment basin, log-crib check dams	4	2
G	STMMN09158	At Asset: Replace trash rack Upstream: Sediment basin, log-crib check dams	4	2
	STMCUL00412	At Asset: Replace trash rack Upstream: Sediment basin, log-crib check dams	4	2

Note:
1. Routine maintenance contributes to risk reduction at each asset. The assigned residual risk rating assumes that routine inspection and maintenance is carried out at each asset.

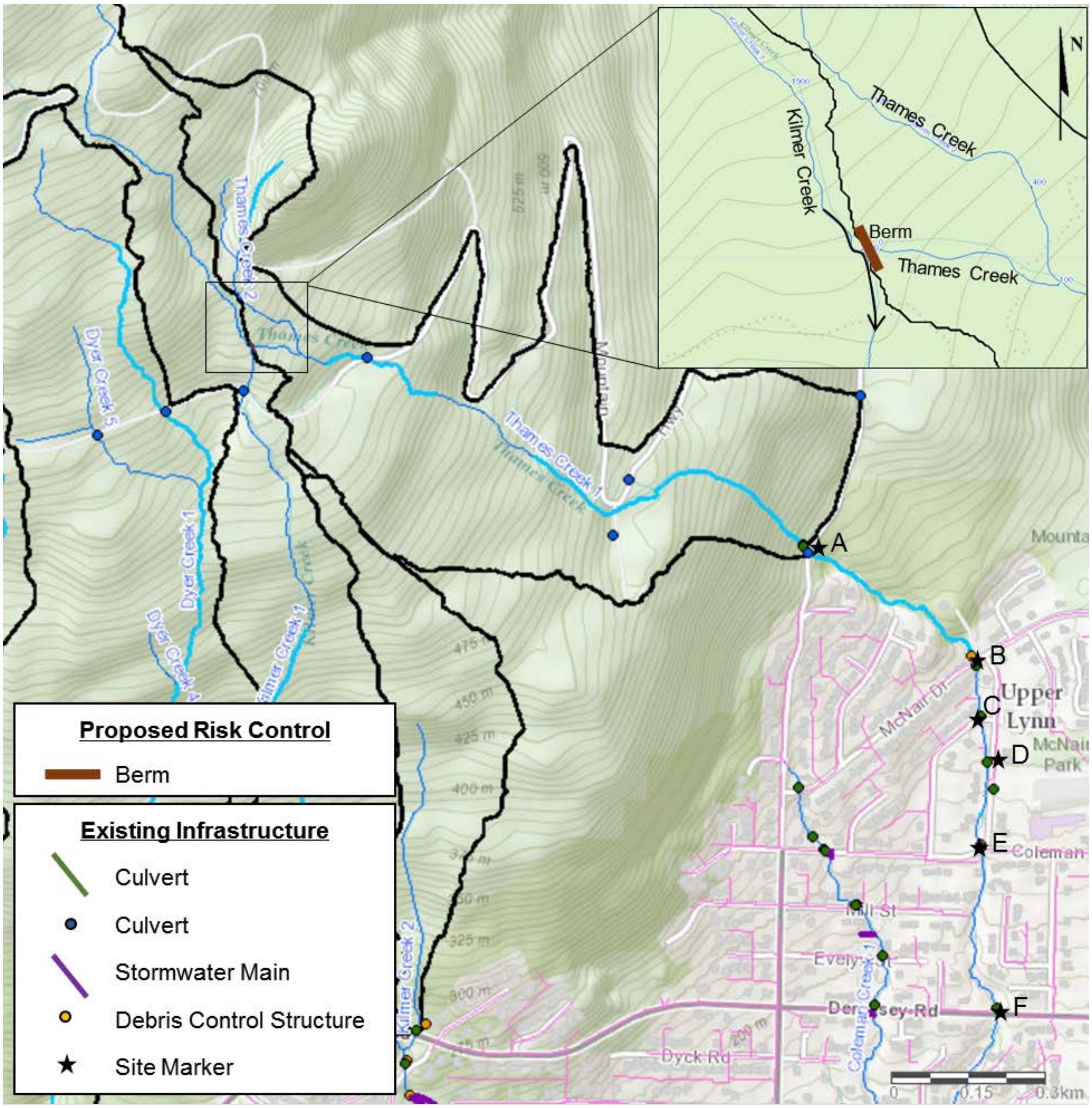

Figure O.8-3. Thames Creek – Long-term risk control – Berm in upper watershed.

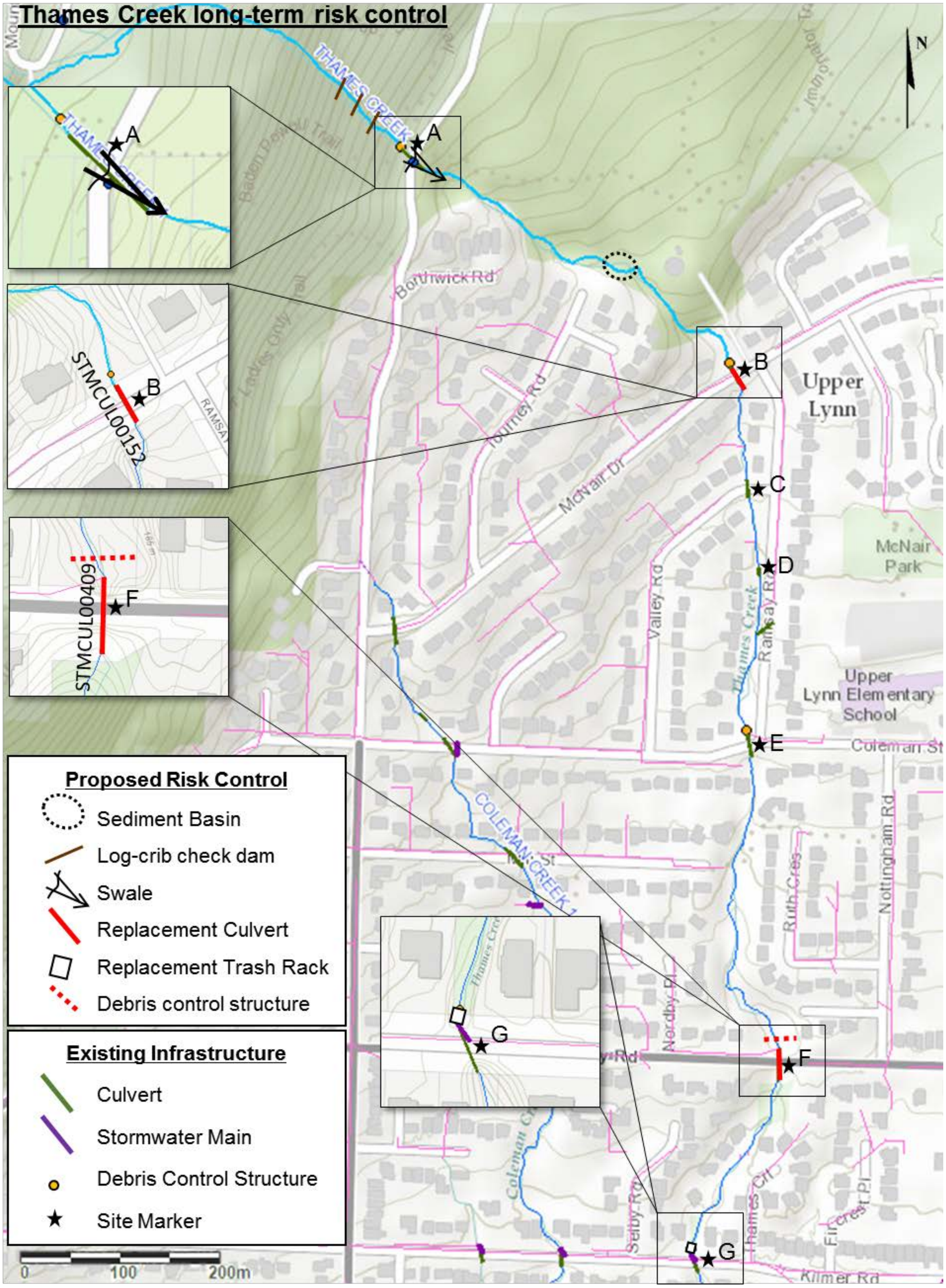

Figure O.8-4. Thames Creek – Long-term risk control.

Thames Creek Risk Control Summary

Proposed risk control elements at Thames Creek serve the following objectives:

- Reduce likelihood of Kilmer Creek avulsing into Thames Creek in the upper watershed
- Reduce likelihood of flow down Mountain Highway from site A
- Reduce the volume of sediment that arrives at site B and continues downstream
- Increase the capacity of downstream culverts to convey the design discharge.

A small berm at the watershed boundary between Thames and Kilmer Creek (see Creek Observation Point near Kilmer station 2000 in DNVHIT) is recommended to discourage Kilmer Creek from avulsing into the Thames Creek watershed. If this avulsion does occur, the peak flows in Thames Creek could increase by a factor two or more. The current configuration at this location is a flow split, where most of the streamflow is contained within Kilmer Creek but an undetermined portion enters into the upper reaches of Thames Creek. Site visits by BGC in July 2015 and August 2016 confirmed this flow split (Figure O.9-5). Berm construction has the advantage of reducing flows in Thames Creek and reducing uncertainty in flow estimates, as Thames Creek currently receives an unknown proportion of the flow from the watershed above the flow split. The berm would also prevent a large avulsion – and dramatically increased peak flows – in Thames Creek in the case of a blockage of the Kilmer Creek channel at the flow split. Berm construction would, however, increase flows in Kilmer Creek relative to current conditions, leading to possible channel adjustment. Before proceeding with berm construction, the DNV should consider whether potentially increased flows into Kilmer Creek are acceptable from a hazard and risk perspective.

Figure O.8-5. Flow is split between Thames (left) and Kilmer (right) Creek. Photograph of July 3, 2015 looking downstream.

Similarly, redirection of surface water drainage along Mountain Highway in the upper watershed could increase discharge received by Thames Creek. Routine inspection and maintenance of drainage infrastructure along Mountain Highway and trails in the upper watershed is recommended to reduce the potential for unintended surface water diversion.

The culvert at site A is susceptible to blockage by sediment and debris. The existing debris barrier at site A is expected to be effective at protecting the culvert inlet from boulders, but will not prevent build-up of finer sediment at the culvert inlet. Currently, if the culvert blocks, water that overflows the culvert is expected to flow down Mountain Highway, and may not return to the Thames Creek channel. The proposed swale across Mountain Highway is intended to return any overtopping flows back to the Thames Creek channel.

Check dams located within 100 m upstream of site A are proposed to improve channel stability in that reach and reduce the sediment volume that arrives at site A and downstream. The channel geometry and alluvial soils in this reach are expected to be favorable for check dam construction. A sediment basin is also recommended between sites A and B in wide, shallow-sloping area with easy access near the end of Tourney Road. This sediment basin would require routine sediment removal, and reduce the volume of sediment that arrives downstream.

The proposed risk control increases the flow capacity of downstream culverts by replacing undersized culverts at sites B and F, installing a debris control structure at site F, and increasing the trash rack screen area at site G to reduce the likelihood that the trash rack is blocked.

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APPENDIX P
DNV DEVELOPMENT SERVICING BYLAW 7388
SCHEDULE D

C9 MAJOR FLOW ROUTING AND FLOOD CONTROL

C9.1 Major Flow Routing

Unless the storm sewer system is oversized to accommodate the major flow (i.e. 100 year return period storm), provision for surface flow is required wherever the overland flows in excess of 0.05 cubic metres per second (m^3/s) are anticipated. Major flow routing is generally accommodated along roadways, swales and watercourses. These designated flow paths will be protected by restrictive covenants or rights-of-ways and clearly identified in the Stormwater Management Plan.

The quantity of flow to be conveyed by the surface flow path is the total major flow less the capacity of the minor system. The design of the major flow routing will ensure to the satisfaction of the District that no endangering of public safety nor any substantial property damage will occur under the major flow conditions.

C9.2 Roadway Surface Drainage

Roadways with barrier curbs and gutters can be designed as wide channels to convey major surface flow. The required freeboard between the water elevation at maximum ponding/flow and the lowest minimum building elevation of the adjacent buildings is specified in subsection C8.15. The maximum depths of flow will not exceed 150 mm above the gutter line. Flow velocities greater than 2.5 m/s must be acceptable to the District.

The Consulting Engineer will consider the impact of surface routing on the major flow hydraulic grade line (HGL) of adjacent lateral roads. Existing lateral roads designed with the major HGL below surface may preclude using surface flow routing on the road being designed.

Routing of major surface flow on roads with rollover curbs is discouraged. The Consulting Engineer will submit calculations to verify that the surface flow is maintained within the road right-of-way and the water elevation at maximum ponding/flow is at least 0.35 metres below the lowest flood construction level (FCL) of adjacent buildings.

The design of the intersections will ensure that the surface flow can continue along the designated path crossing over lateral roads. Similar considerations are required if a change of surface flow direction is required at an intersection.

C9.3 Ditches

Properly engineered ditches may be acceptable for permanent servicing of land development projects in urban areas of the District to reduce the storage required for stormwater management. Ditches adjacent to roadways will conform to the following criteria:

.1	maximum depth	1.0 m
.2	minimum bottom width	0.5 m
.3	maximum side slope	2.0(H):1(V)
.4	minimum grade	0.5%
.5	maximum velocity (Unlined ditch)	1.0 m/s

Where soil conditions are suitable or where erosion protection is provided, higher velocities may be permitted. If grades are excessive, rip-rap lined bottoms and sides of ditches, erosion control structures or complete ditch enclosure may be required.

The minimum right-of-way width for a ditch will be 5 metres where the ditch crosses private property. The ditch will be offset in the right-of-way to permit a 3 metre wide access for maintenance vehicles. Additional right-of-way may be required to facilitate the ditch construction and access. The top of the ditch adjacent to the property line will be a minimum 0.5 metres away from that property line. Ditches will be designed to maximize infiltration.

C9.4 Creeks

Natural creeks are integral components of the drainage system and the ecological system. If the process of development or drainage design involves in-stream works, the Consulting Engineer will refer to the latest version of the "*Land Development Guidelines for the Protection of Aquatic Habitat*" prepared by the Department of Fisheries and Oceans (DFO) & the B.C. Ministry of Water, Land and Air Protection (MWLAP), and Section 9 of the Water Act.

Any activity within a creek corridor or wetland is subject to the provisions of the District's *Environmental Protection and Preservation Bylaw #6515*.

All proposals for works within a creek corridor must be forwarded (by the Consulting Engineer) to the District's Sustainability, Planning and Building Services Division, who will liaise with the federal and provincial government agencies:

C9.5 Culverts

Culverts on creeks will be designed to convey the major flow (200-year return period instantaneous flow) or greater with the design headwater not exceeding the top of the culvert. The Consulting Engineer will determine whether the culvert will operate under inlet or outlet control at design conditions.

Concrete culverts are preferred for general uses. Corrugated steel culverts may be considered under special circumstances when their use can be justified.

The minimum diameter of culverts on creeks is 450 mm. The minimum diameter of driveway culverts that form part of the minor system is 300 mm. The average water velocity in culverts should not exceed:

- 1.2 m/s for lengths up to 24.4 metres
- 0.9 m/s for lengths greater than 24.4 metres

The minimum depth of cover over culverts is 0.3 metres, subject to the correct pipe loading criteria.

Inlet and outlet structures are required for all culverts designed for the 200-year return period instantaneous flow. Considerations for the installation of energy dissipation and erosion control will be included in the design.

Culverts on fish-bearing creeks must meet special conditions as specified by the District's Environment, Parks and Engineering Department, Fisheries and Oceans Canada, and the BC Ministry of Water, Land and Air Protection (MWLAP). Such culverts will be required to be passable to fish. Habitat restoration works will generally be required. The Consulting Engineer will consult the District to determine the requirement for individual projects.

Driveway culverts that form part of the minor system will have capacity for the runoff from the 10-year return period storm with the design headwater not to exceed the top of the culvert. All new driveway culverts will be sized to ensure that there is no adverse impact on adjacent properties under the 100-year return period runoff conditions.

Trash racks and/or debris barriers are required upstream of culvert installations. Refer to the Supplementary Standard Drawings

C9.6 Inlet and Outlet Structures

Refer to Supplementary Standard Drawings for the design of inlet and outlet structures for pipes up to 1200 mm diameter. Pipes larger than 1200 mm diameter and non-circular culverts require specially designed inlet and outlet structures. Outlets having discharge velocities in excess of 1 m/s require riprap and/or energy dissipating structures for erosion control.

Trash racks are required at the inlets and outlets of all pipes over 450 mm in diameter and exceeding 30 m in length (except large culverts in major watercourses). Trash racks may also be required on smaller diameter storm sewers at the discretion of the District. See Supplementary Standard Drawings for trash rack details.

C9.7 Flood Control and Debris Flow Hazards

Flood control provisions apply to development on sites subject to flood-related hazards. This section defines some general flood control provisions. The need for site specific provisions will be determined by the District in consultation with the B.C. Ministry of Water, Land and Air Protection (MWLAP), and/or the District's Chief Building Official. Owners and Consulting Engineers are directed to District MRLs #SPE106, *Flood Hazard Report* and #SPE107, *Creek Hazard Report*, with respect to flood hazard reports which may need to be submitted in the case of properties located within identified flood and debris hazardous areas.

Steep creeks within the District may be subject to debris flow or debris flood hazards, which are generally defined in the *Overview Report on Debris Flow Hazards*, prepared by Kerr Wood Leidal Associates Ltd and EBA Engineering Consultants, dated April 1999. Development in such areas will require satisfactory mitigation of the respective hazards.

Creeks and rivers may also give rise to flood and erosion hazards which must be mitigated through implementation of flood construction levels (FCLs') and building setbacks as follows:

.1 Proposed buildings that are subject to flood hazards require a specified FCL, which is the minimum elevation for main habitable floor areas. The FCL ensures that buildings are elevated sufficiently high that flood inundation will not occur up to the design flood condition. The FCL applies to the underside of wood floor systems, or the top of the concrete floor systems.

.2 FCLs are to be determined for any of the following conditions:
   i. 200-year return period flow for creeks and rivers (including 0.6 m freeboard).
   ii. 100-year flow plus hydraulic gradeline (normally 0.3 metres unless acceptable to the District).
   iii. The Seymour River FCLs as shown on the floodplain map produced by MWLAP.

.3 The need for creek setbacks over and above the environmental protection requirements (in order to ensure safe building sites) will be determined on a site specific basis.

A gravity connection to the District's storm drainage system may be made only where the habitable portion of a dwelling is above the major system hydraulic grade line.

C9.8 Limitations and Precautions to Implementing Source Controls in Hazardous Areas with Potential Slope Instability

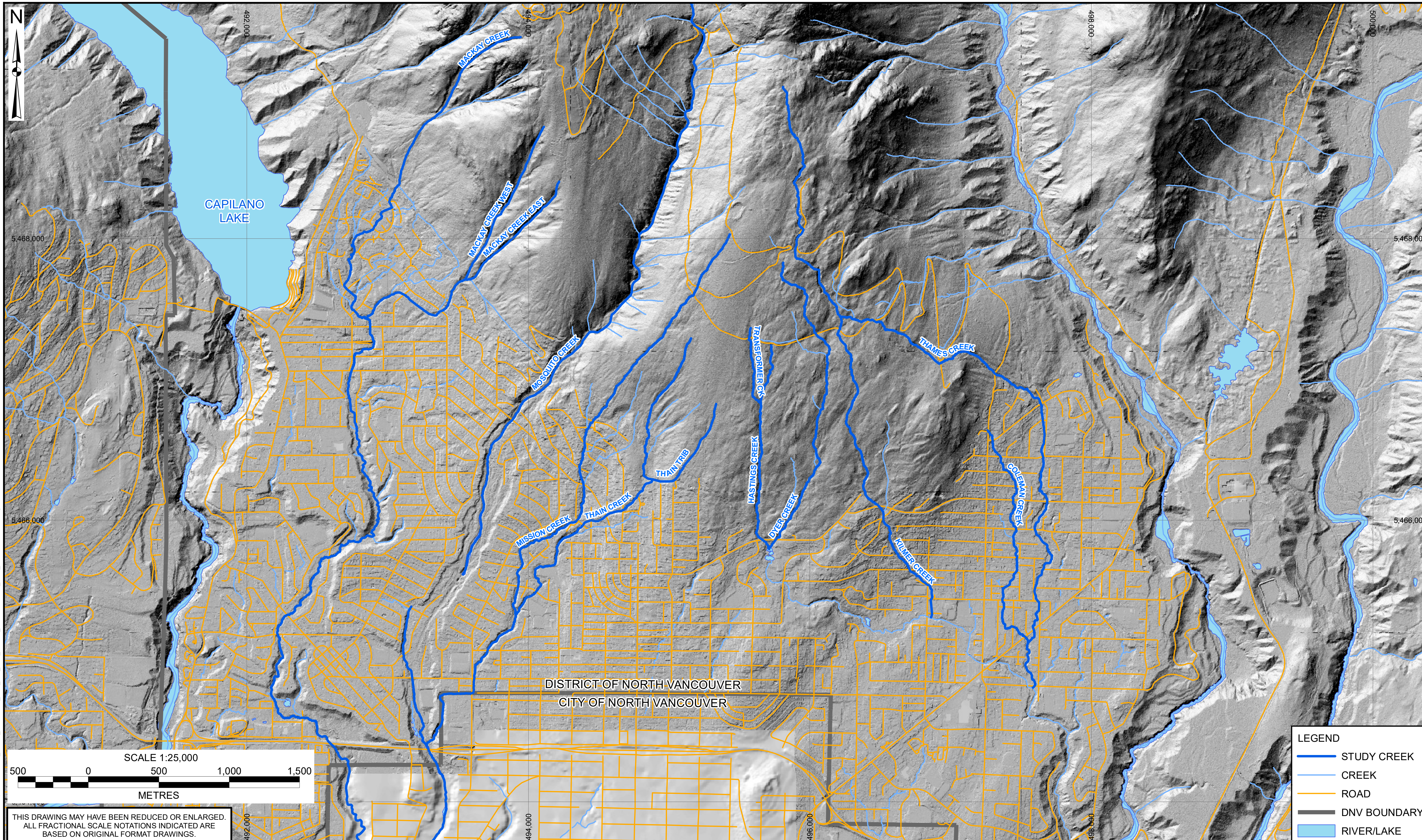
The implementation of source controls is prohibited in potential slope instability areas. Source controls encourage infiltration that saturate soils and further reduces the stability of these hazardous slopes. Adequate setbacks from the top of these slopes must be delineated by a qualified professional geotechnical engineer.

C9.9 Groundwater Downslope Impact

A hydrogeologist must be retained to assess the fate of infiltrated water to confirm that it does not pose an increased saturation/flooding risk to down slope areas and/or adjacent developed or undeveloped sites.

C9.10 Overflows

As with all drainage works, source controls must be designed to ensure that facility overflows and interflows drain to the municipal minor/major drainage system or natural drainage path, and do not discharge to, or through, adjacent sites. Emergency overflows must be designed into all source controls.



NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT", AND DATED MAY 2017.
3. BASE TOPOGRAPHIC DATA BASED ON LIDAR PROVIDED BY DNV FLOWN IN SPRING/SUMMER 2013 - PROVIDED JUNE 2015.
4. PROJECTION IS UTM 10 NAD83.
5. UNLESS BGC AGREES OTHERWISE IN WRITING, THIS DRAWING SHALL NOT BE MODIFIED OR USED FOR ANY PURPOSE OTHER THAN THE PURPOSE FOR WHICH BGC GENERATED IT. BGC SHALL HAVE NO LIABILITY FOR ANY DAMAGES OR LOSS ARISING IN ANY WAY FROM ANY USE OR MODIFICATION OF THIS DOCUMENT NOT AUTHORIZED BY BGC. ANY USE OF OR RELIANCE UPON THIS DOCUMENT OR ITS CONTENT BY THIRD PARTIES SHALL BE AT SUCH THIRD PARTIES' SOLE RISK.

SCALE:	1:25,000
DATE:	MAY 2017
DRAWN:	JCS
CHECKED:	LCH, MTL
APPROVED:	KH

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:
NORTH VANCOUVER

PROJECT:	DEBRIS GEOHAZARD RISK AND RISK CONTROL ASSESSMENT
TITLE:	DNV WEST CREEKS
PROJECT No.: 0404-054	DWG No: 01

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