Town of CANMORE

alpinfra
engineering competence

MOUNTAIN CREEK HAZARD MITIGATION
DESIGN OF MITIGATION MEASURES

COUGAR CREEK

Interim Report 03

Final Option Analysis

Prepared by:	alpinfra consulting + engineering gmbh
Edition:	R03 / January 2015
Print:	13th of January 2015
Distribution:	Recipient: 1 digital copy alpinfra: 1 digital copy/1 hardcopy

G:\Eng_Planning\Capital Projects\2014\1445-Cougar Creek Debris Flood Mitigation\Technical\Alpinfra\Option Analysis Phase\CCLTM Option Analysis Alpinfra FINAL REPORT 150113.docx

Revisions

Revision Nr.:	Description	Date
01	Update of Option Analysis based upon the first Coordination from July 2014	18th of August 2014
02	Adaptions	18th of September 2014
03	Final Adaptions	12th of December 2014
04	Final Revisions	13th of January 2015

00 TABLE OF CONTENT

00.01 List of Figures

00.02 List of Tables

00.03 List of Drawings

Drawing Nr.:	Content	Type	Scale / Size
Option A			
16494-OPT.A-001	Overview Map	Site Map	1:1,000 279x1064mm
16494-OPT.A-010	Flood Retention Dam, Station KM 2+900	Site Map	1:500 279x864mm
16494-OPT.A-011	Flood Retention Dam, Station KM 2+900	Cross Section 01	1:250 432x1295mm
16494-OPT.A-012	Flood Retention Dam, Station KM 2+900	Cross Section 02	1:250 432x1095mm
16494-OPT.A-013	Diverting Structure – Station KM 2+360	Site Map Length Section	1:500, 1:250 279x1095mm
16494-OPT.C-020	Ground Sills (same as for Option C)	Regular Drawing Cross Section Length Section	1:200 279x1016mm
Option B			
16494-OPT.B-001	Overview Map	Site Map	1:1,000 432x864mm
16494-OPT.B-010	Flood Retention Dam, Station KM 2+900	Site Map	1:500 559x 1064mm
16494-OPT.B-011	Flood Retention Dam, Station KM 2+900	Cross Section 01	1:250 432x 1295mm
16494-OPT.B-013	Diverting Structure, Station KM 2+360	Site Map Length Section	1:500, 1:250 279x1095mm
16494-OPT.C-020	Ground Sills (same as for Option C)	Regular Drawing Cross Section Length Section	1:200 279x1016mm
Option C			
16494-OPT.C-001	Overview Map	Site Map	1:1,250 559x2592mm
16494-OPT.C-010	Gravel Retention Structure, Station KM 2+450 Diverting Structure, Station KM 2+360	Site Map	1:1,000 279x736mm
16494-OPT.C-011	Gravel Retention Structure, Station KM 2+450	Length Section	1:500 279x918mm
16494-OPT.C-012	Gravel Retention Structure, Station KM 2+450	Cross Section 01 Cross Section 01	1:200 279x864mm
16494-OPT.C-013	Diverting Structure, Station KM 2+360	Length Section Cross Section	1:500, 1:250 279x1095mm
16494-OPT.C-020	Ground Sills	Regular Drawing Cross Section Length Section	1:200 279x1016mm
16494-OPT.C-030	Gravel Retention Structure, Station KM 0+720	Site Map	1:500 432x864mm
16494-OPT.C-031	Gravel Retention Structure, Station KM 2+450	Length Section	1:500 279x1016mm
16494-OPT.C-032	Gravel Retention Structure, Station KM 2+450	Cross Sections	1:150 432x559mm

01 EXECUTIVE SUMMARY

01.01 Background, Objectives and Basic Data

The Town of Canmore, as well as the Highway 1, the Bow Valley Trail and the CP Railway Line, were hit several times by debris floods and floods discharged from the watershed of the Cougar Creek. The size of the watershed is approximately 43 km². The latest flood event took place in 2013, devastating infrastructure and houses.

To mitigate against this hazard, a long term mitigation project was initiated. As a first step within the actual design work, an option analysis for long-term mitigation measures was conducted, worked out by **alp**infra consulting + engineering gmbh. Input data for this study was provided mainly by BGC Engineering Ltd. in form of a forensic report, a hazard assessment and a hydro-climatic analysis of the event of June 2013.

alpinfra performed supplementary hydrological and hydraulic investigations for the derivation of design hydrographs to serve as a basis for the conceptual design of protection structures. Preliminary, site specific, geological and geotechnical information could be collected during **alp**infra's own on-site investigations. First results from a geotechnical investigation program were provided by Thurber Engineering Ltd. in form of a memorandum.

01.02 Development of Options

alpinfra developed two main strategies for long term mitigation measures as a first draft. These strategies were accomplished through several potential placements of structures. To refine those options a coordination workshop was held on July 25, 2014. Revisions for a second draft were made in close coordination with stakeholders, based upon different mitigation strategies and drawings prepared by **alp**infra. The following options were selected and renamed for further design work.

<u>Option A:</u>

Debris flood retention at station KM 2+900, leading to (a) a highly reduced remaining peak discharge and (b) highly reduced debris mobilization in the channel. The conceptual structure height is 34m, calculated from the existing channel bed level to the crest of the structure. Additional structures were developed and designed conceptually to ensure that the remaining and reduced discharge flows into the existing and reinforced channel.

<u>Option B:</u>

Debris flood retention at station KM 2+500, leading to (a) a highly reduced remaining peak discharge and (b) highly reduced debris mobilization in the channel. The conceptual structure height is 24m, calculated from the existing channel bed level to the crest of the structure. Additional structures were designed to ensure that the remaining and reduced discharge flows into the existing and reinforced channel.

<u>Option C:</u>

Debris retention at station KM 2+450, without flood retention. The conceptual structure height is 11 m, calculated from the existing channel bed level to the crest of the structure. Additional structures were designed to ensure the following: (a) that the un-retained flood-discharge will flow into the existing and reinforced channel, (b) that the channel banks and the channel bed is protected against erosion by gravel under-saturated water flood discharge, and (c) that the gravel, accumulated by the flood discharge at the channel, is retained at a second retention structure at station KM 0+720.

01.03 Further Investigations

For detailed design work for a selected option, additional investigation will be required:

a) Geotechnical investigation program once the preferred option is selected:
   a. Core-drillings at potential abutments and footprints
   b. Geo-mechanical and geo-hydraulic in-situ-testing during drilling
   c. Plate pressure tests at footprints of structures
   d. Geo-mechanical and geo-hydraulic characterization of dam-filling material
   e. Compaction tests of potential dam-fill material
   f. Geological and geotechnical surface mapping, predominantly at the abutments of structures, and stability mapping at the storage slopes

b) Detailed hydrological analysis
   a. Calculation of additional event scenarios
   b. New back-calculation of the flood event of June 2013 considering the hydraulic back-analysis at the culvert of Elk-Run-Boulevard

c) Detailed investigation of relevant potential mass-movements leading to blockage and flood wave impact
   a. Geo-mechanical stability calculations of potential slope failures and description of frame-conditions for failure scenarios
   b. Dynamic calculation of rock- and/or soil avalanches potentially reaching the creek
   c. Estimation of disintegration and crushing of the source material, for derivation of material parameters in terms of estimating stability, erosion-rates as well as seepage of potential creek-blockages
   d. Recalculation of relevant dam breaching and flood-wave impact scenarios

02 LIMITATIONS

alpinfra consulting + engineering gmbh prepared this report for the Town of Canmore. It focuses on the development of options for long-term hazard mitigation measures at Cougar Creek. The option analysis is based mainly on data presented within (a) the hydro-climatic analysis (BGC 2014a), (b) the debris flood-hazard assessment (BGC 2014b), (c) preliminary geotechnical investigations (Thurber Engineering ltd. 2014) as well as supplementary hydrological analysis performed by **alp**infra. The results in this report are based on basic data and general information available to **alp**infra at the time of report preparation. Any use a third party makes of this report or any reliance on decisions based on it, is done within the responsibility of such a third party. **alp**infra takes no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. In particular, **alp**infra accepts no responsibility for changes in real estate values that may occur as a consequence of this report. In terms of protection to our client, the public, and **alp**infra, this report is submitted for further use by the Town of Canmore. Authorization outside of this use needs our approval. The report and the design worked out within this step of option analysis are to be understood as preliminary. Design drawings are not suitable as basis for permitting or construction but for estimating costs and making decisions during option selection. Divergences at assumed geotechnical frame conditions are possible, resulting in the requirement of re-designing current options or designing new options.

03 INTRODUCTION

03.01 Project Background and Scope of Work

03.01.01 Project Background

Canmore, Alberta, has been significantly hit by flooding on a number of steep mountain creeks in recent decades. The floods of June 2013 resulted in severe devastation of houses and infrastructure. The Town is now seeking options for long term mitigation strategies. Therefore, the Town of Canmore invited international experts in the field of mountain hazard engineering to work out proposals addressing the development of protection structures.

03.01.02 Scope of Work

alpinfra consulting + engineering gmbh, an Austrian engineering company highly specialized in mountain hazard protection and correlated geotechnical engineering, was retained to undertake the project . The scope of work for **alp**infra was based on the following key deliverables:

a) Mitigation strategies

b) Option analysis for mitigation measures

c) Full preliminary design for one selected option

d) Detailed design

e) Advices within preparation of tender documents and during construction-stage

This work is to be carried out in close coordination with the Town of Canmore and its third party consultants.

This report relates to the option analysis for long-term mitigation measures at Cougar Creek.

04 INPUT DATA

This chapter is listing basic data, obtained from reports mainly provided by BGC, as well as other basic data available. All data discussed herein, is relevant and required for further actions within the option analysis. In this chapter we describe main issues and point out needs for further, more detailed basic data analysis.

04.01 Debris Flood Hazard Assessment

04.01.01 General Information

The Debris Flood Hazard Assessment prepared by BCG Engineering highlights the torrential hazards caused by Cougar Creek, leading to impacts on the developed area of the fan, deposited by the creek. The report gives a very good overview and allows us to develop a detailed understanding of the relevant hazardous processes for Cougar Creek, in a high standard. The hazard assessment points out a severe potential of returning floods and debris floods leading to potential impacts on the developed areas on the fan. Erosion within the catchment area is leading to inundation and deposition of gravel-debris at the developed area on the fan of Cougar Creek. Floods and debris floods are mainly induced by heavy rainfall. BGC identified another potential debris flood source resulting from blockage of Cougar Creek from landslides or debris-flows from tributary creeks, potentially resulting in dam breaching. According to BGC's hazard assessment report, sediments bonded on the flanks of Cougar Creek are indications of those phenomena. Within the hazard assessment, BGC Engineering investigated flood and debris flood frequencies as well as corresponding magnitudes.

04.01.02 Quantitative Estimations

Derived flood and debris flood discharges of former events were classified by BGC as "reasonable approximations for the respective return period class" (BGC 2014b).

The hazard report provides data for the 2013 event and for annual gravel volumes, summarized below:

- By comparison of terrain models from 2009 and 2013, the sediment volume, accumulated downstream of the bedrock canyon during the event of 2013, was estimated to be 227,500m³. This includes the amount of gravel eroded at this section.
- The potential of dam breach scenarios was identified within the hazard assessment by BGC. Estimations of peak-discharges, due to dam breaching, lead to values of up to 1000m³/s. Because of this, there is the potential for high flow and high impact forces that need to be taken into account in the design of mitigation structures. There is a mutual agreement between involved parties that a more detailed investigation of dam breaching scenarios is needed. This investigative program will be coordinated between BGC and **alp**infra for the next steps.
- The annual gravel accumulating at the fan, between Bow Valley Trail and the CPR, which has to be taken into account, is estimated to be approximately 5,000m³ to 8,000m³.
- The event frequency, according to the hazard assessment (fig. 4-3), is showing 15 likely debris flood events within the past ~150 years (see Figure 1).
- The estimated peak-discharge for return periods between 100 and 300 years is approximately 60m³/s. Flood-discharge and debris volumes are shown in Table 1 and Table 2. A hydrological analysis, based on records of precipitation data, was not done within the hazard assessment. A supplementary hydrological analysis is necessary before any further design work takes place.

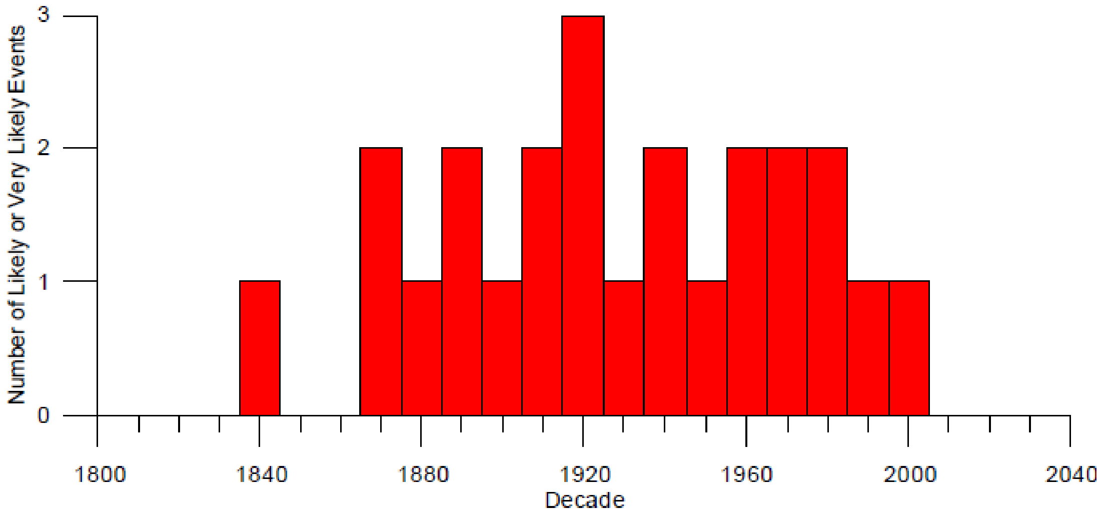

Figure 4-3. **Number of very likely to likely events per decade as interpreted from the dendrochronology analysis of 70 tree samples.**

Figure 1: Figure 4-3 from the hazard assessment done by BGC Engineering Ltd.

Table 1: Table ES-1-1. Debris flood frequency – magnitude relation for Cougar Creek (Hazard Assessment BGC 2014b)

Return Period (T) (yrs)	Annual Probability (1/T)	Volume Best Estimate (m^3)	Peak Discharge (m^3)	Dominant Hydro-Geomorphological Process
1-10	1-0.1	< 6,000	-	flooding
10-30	01-0.03	30,000	30	flooding/debris floods
30-100	0.03-0.01	40,000	50	flooding/debris floods
100-300	0.01-0.003	60,000	60	debris floods
300-1000	0.003-0.001	160,000	700	landslide dam outbreak floods
1000-3000	0.001-0.0003	260,000	1000	landslide dam outbreak floods

Table 2: Scenarios supplied by BGC Engineering

Table 7-2. Simulated scenarios and input parameters.

Return Period (yrs)	Volume Estimate (m^3)	Sediment concentration (%)	Peak* Flow (m^3/s)	Hydro-Geomorphic Processes	ID	Model Runs and Assumptions
< 10	< 6,000	0	-	Flooding		No run
10 to 30	30,000	10	30	Flooding/ Debris flood	1	ERBC performs to capacity
30-100	40,000	20	50	Debris flood	2	ERBC performs to capacity
100 to 300	60,000	20	60	Debris flood/LDOF	3a	ERBC performs to capacity
					3b	ERBC is blocked
300 to 1000	160,000	30	700	LDOF	4	ERBC is blocked
1000 to 3000	260,000	30	1000	LDOF	5	ERBC is blocked
No mitigation[1]	90,000	20	80	Debris flood	6	ERBC performs as it is kept open artificially

LDOF = landslide dam outbreak flood, ERBC = Elk Run Boulevard culvert, [1] represents June 2013 event, * Peak flow as reported here is the total discharge including the sediment in transport.

04.02 Hydro-Climatic Analysis of the June 2013 Storm

In BGC's Hydro-Climatic Analysis (BGC 2014b), all available rainfall and precipitation data was collected and processed extensively. This collected data provides the input data that is the basis for hydrological calculations needed for further design of mitigation measures.

04.03 Photographs and Videos of the 2013 flood at Cougar Creek

Photos and videos available on the internet provide us with valuable indications on the evolution of the hydraulic regime at the culvert of the Elk Run Blvd. during the flood event of 2013 and allows back-modelling.

04.04 Complementary Hydrological Analysis

Based upon the hydro-climatic analysis of the June 2013 storm (BGC 2014a), a supplementary hydrological analysis was performed by **alp**infra to assess the magnitude of floods for different rainfall event durations and return periods. The results of this analysis are a set of flood hydrographs assigned to different return periods. They serve as a basis for the design of mitigation options and protection structures. Particularly the event of June 2013 shows that in the case of a flood event, the volume of water which has to be taken into account for Cougar Creek is far larger than assumed in previous estimates, where general and more regional approaches were used. Our experience is that steep mountain creeks react very differently to storm events, in terms of flood discharges. Within the current hydrological analysis, comparable conditions at case studies from Austrian Alps, helped us to find an appropriate approach for establishing parameters.

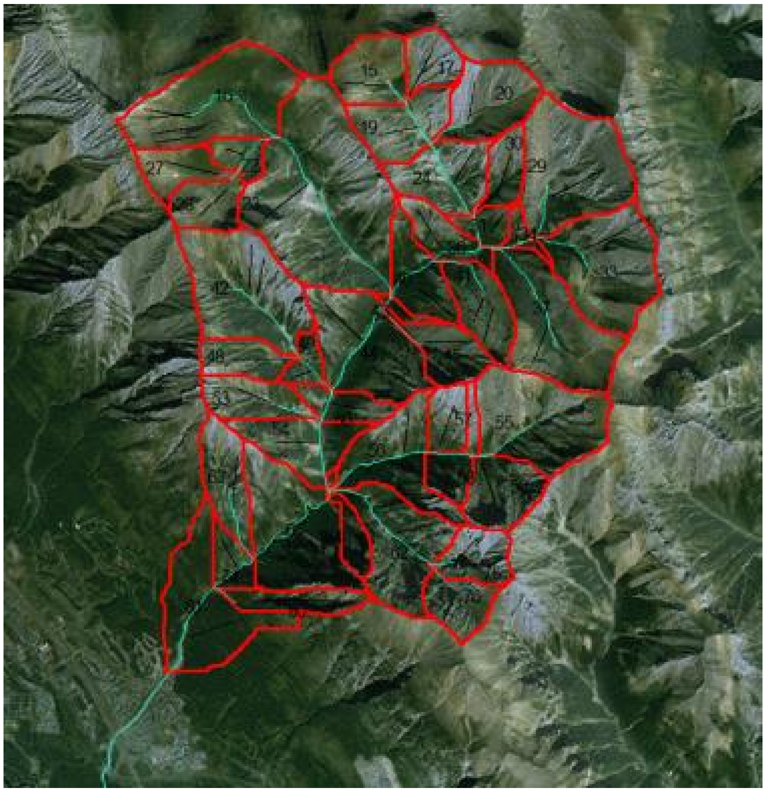

Figure 2: Location of the study area (unspecified scale) and sub-catchment areas

04.04.01 Methodology

04.04.01.01 General Approach

Precipitation-runoff models are used to model the runoff regime of a watershed for particular precipitation situations. The model used for Cougar Creek mathematically derives the hydrographs of a watershed, considering topography, precipitation intensity and specific local runoff characteristics.

For modelling of the precipitation-runoff, the HEC-HMS model (Hydrologic Modeling System by the Hydrologic Engineering Center of the US Army Corps of Engineers) was used. A detailed description of the model and the input parameters can be downloaded at the following homepage (http://www.hec.usace.army.mil/).

Prior to calculation, we characterized the runoff regime based on similar, well-calibrated case studies available to us. For the parameterization we used the software module HEC-GeoHMS, which is a geospatial hydrology toolkit developed for generating input parameters for the actual model.

Using this toolkit, the watershed area was divided into sub-catchments. The resulting sub catchments were connected according to flow direction. For each sub-catchment area, the processes of runoff formation, runoff concentration, and retention in the channel bed were modeled. The duration of flow was determined by topographic conditions, land use, assumed soil texture, forest cover and topography. The flow resistance in the channel bed was taken into account as a function of the stream geometry, stream inclination and assigned roughness.

04.04.01.02 Model Setup

The precipitation-runoff model developed, comprises 42 sub-catchment areas with an area of approximately 1 km² each. For each sub-catchment stream sections were defined. The pre-processing toolkit HEC-GeoHMS was applied in advance. HEC-GeoHMS extracts topographic, topologic and hydrologic information from digital spatial data, e.g. a digital terrain model (DTM). This data gathered at the complete watershed and the precipitation component is then processed by means of HEC-HMS to automatically create a schematic network of sub-catchment areas and streams. Figure 3 shows the model setup with the delineated sub-catchments and channel networks.

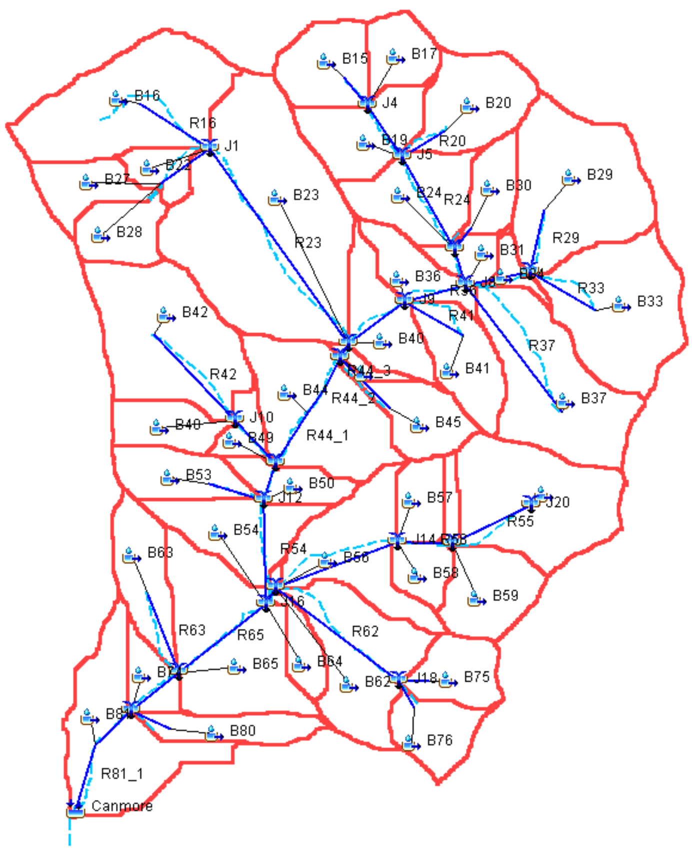

Figure 3: Setup of the precipitation-runoff model of COUGAR CREEK

04.04.01.03 Parameters

<u>Curve Number and Initial Abstraction</u>

The Runoff Curve Number (CN-value) is an empirical parameter developed by the Soil Conservation Service (SCS) at the US Department of Agriculture (USDA) and determines the amount of rainfall contributing to direct runoff. The Curve Number depends on the soil type, land use and hydrologic conditions and is the basis for further calculations within the modelling. The higher the Curve Number, the higher the amount of direct runoff or the lower the amount of water infiltrating into the ground.

The initial CN-values for each sub catchment of Cougar Creek were determined based on the geologic situation and the vegetation cover.

Initial abstraction describes the amount of water before runoff, theoretically being absorbed by the watershed (infiltrated or stored by interception and evaporation), without increasing the discharge. Initial loss is calculated using the formula developed by Kleeberg and Øverland (1989):

$$initial\ abstraction\ [mm] = 0.1 \cdot \left(\left(\frac{25{,}400}{CN} \right) - 254 \right)$$

Table 3: Initial loss values depending on the CN-value

SCS Curve Number	Initial abstraction [mm]
40	38.1
50	25.4
60	16.9
70	10.9
80	6.4

Base-flow

The base flow describes the amount of discharge added to the stream by groundwater inflow. During short term, or rapidly rising floods, the base flow reaching the stream is temporarily delayed, and thereby reduces the peak discharge. The “recession base flow method” was applied within the current analyses.

Time of concentration

The time of concentration was individually determined for each sub-catchment area according to the approach developed by Izzard (1946). Within Izzard’s approach, the time of concentration is modifiable with the precipitation intensity. With this approach it is possible to consider the fact that flow velocity increases with increased discharge, which results in the decrease of the time of concentration at each junction between the sub-catchments. The correct determination of the concentration time is essential, to be able to compute realistic results for the whole watershed, as well as in the sub-catchment areas (which can be exposed to largely differing precipitation conditions). Convective, local, as well as widespread rainfall can therewith be processed in the same precipitation-runoff model.

Dynamic factor of loss

By calibrating and back-calculating a number of local, as well as regional precipitation-runoff analysis sources, **alp**infra could employ an approach called **loss**rate. Applying **loss**rate, we assign a CN-value to each sub-catchment area, based upon preceded rainfall and the characteristics of the sub-catchment areas. This factor can be compared with the CN-I and CN-III value of the SCS method, which adapts the originally assigned CN-value by including antecedent moisture conditions. Because of the lack of reliable data in the Cougar Creek watershed that would lead us to a decent estimate of peak discharges, run-off parameters need to be estimated. The estimated parameters are based on comparable case studies calibrated by means of water level records, as well as records of rainfall gauges.

Furthermore, an increase rate of the runoff coefficient related to precipitation was determined. The increase rate is based on similar, calibrated case studies. It takes into account that direct runoff increases with precipitation. The longer the precipitation lasts, the more saturated is the soil and the less infiltration occurs. The increase rate depends primarily on the rainfall duration and, to a lesser extent, on the rainfall intensity.

The currently assumed run-off parameters are shown in Table 4.

Table 4: Parameters for runoff coefficients (Return period =100 years, Duration = 5h)

subbasin	area km²	% forest	start CN	start run-off coefficient	end run-off coefficient
			73	**0.31**	**0.44**
15	0.82	0%	80	0.43	0.52
16	2.18	55%	71	0.27	0.42
17	0.54	0%	80	0.43	0.52
19	0.96	0%	80	0.43	0.52
20	1.08	0%	80	0.43	0.52
22	0.35	100%	64	0.17	0.33
23	4.12	52%	72	0.28	0.43
24	1.04	22%	77	0.36	0.48
27	0.58	46%	73	0.30	0.44
28	0.60	38%	74	0.32	0.46
29	2.08	11%	78	0.40	0.50
30	0.49	1%	80	0.43	0.52
31	0.32	61%	70	0.26	0.41
33	1.90	11%	78	0.40	0.50
34	0.18	57%	71	0.27	0.42
36	0.58	28%	76	0.35	0.47
37	1.97	45%	73	0.30	0.44
40	0.85	74%	68	0.23	0.38
41	0.84	24%	76	0.36	0.48
42	2.08	45%	73	0.30	0.44
43	0.12	73%	68	0.23	0.39

subbasin	area km²	% forest	start CN	start run-off coefficient	end run-off coefficient
44	1.77	47%	73	0.29	0.44
45	0.70	6%	79	0.41	0.51
48	0.57	64%	70	0.25	0.40
49	0.30	86%	66	0.20	0.36
50	0.46	74%	68	0.23	0.38
53	0.70	63%	70	0.25	0.41
54	1.01	78%	68	0.22	0.38
55	1.93	1%	80	0.43	0.52
56	1.34	55%	71	0.27	0.42
57	0.45	18%	77	0.37	0.49
58	0.54	59%	71	0.26	0.42
59	0.86	36%	74	0.32	0.46
62	1.54	74%	68	0.23	0.39
63	0.82	66%	69	0.24	0.40
64	0.37	99%	64	0.17	0.33
65	2.05	71%	69	0.23	0.39
74	0.40	100%	64	0.17	0.33
75	0.63	4%	79	0.42	0.51
76	0.63	42%	73	0.31	0.45
80	0.55	52%	72	0.28	0.43
81	1.74	99%	64	0.17	0.33

Flood Routing

We applied the Muskingum-Cunge method to calculate stream and overland flow. This method is based upon topographic data and stream profiles.

04.04.02 Precipitation Data

04.04.02.01

The characteristic design precipitation was extracted from the hydro-climatic analysis of the June 2013 storm (BGC 2014a). The intensity-duration-frequency curve (IDF-curve) for the nearby Kananaskis station is shown in Figure 4. This figure shows the rainfall intensities for different durations and return periods. In general, the intensities decrease with increasing rainfall duration.

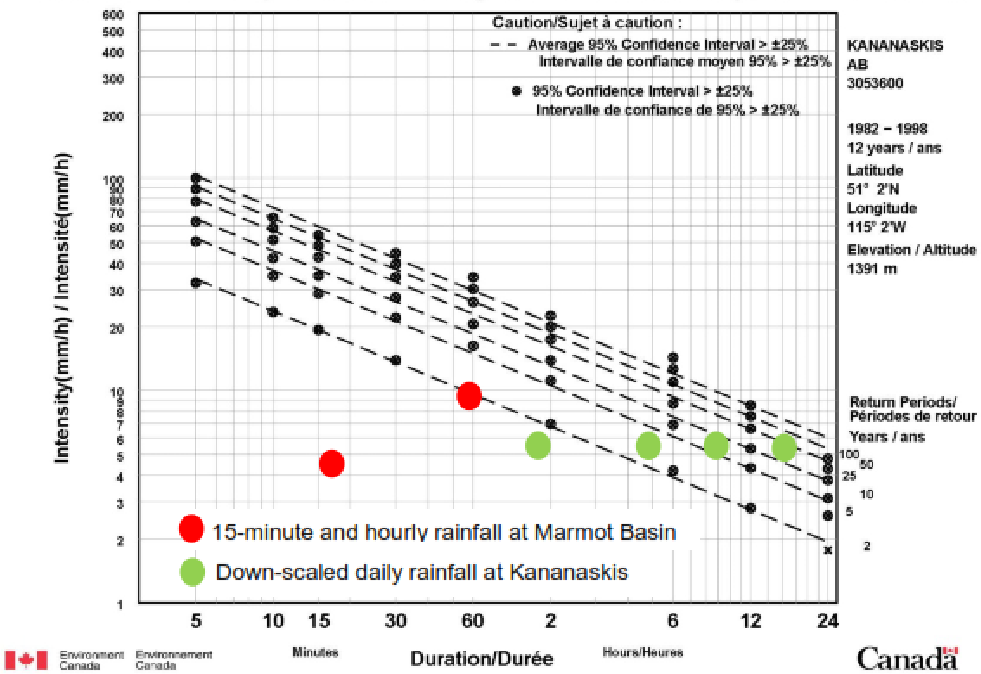

Figure 4: Intensity-Duration-Frequency curve for Kananaskis climate station with June 2013 event rainfall intensities superimposed (BGC 2014a)

The values displayed in the IDF-curve are the maximum values for a certain scenario and are technically only valid for a single point. As the rainfall intensity changes over the area, a reduction factor has to be taken to account for bigger watersheds. In this case, the reduction of the rainfall intensities in reference to the catchment area, was neglected because the Cougar Creek watershed can be seen as a relatively small area, where widespread, as well as very high rainfall intensities, have to be reckoned with.

04.04.02.02 Deriving effective rainfall

The effective rainfall is the amount of the total rainfall that contributes to the discharge. It is computed by the total rainfall minus the losses described above (initial abstraction and watershed storage expressed in the runoff coefficient). The result is the amount of water over a certain time period. This time period is determined by the rainfall duration.

The resulting average discharge coefficient of the watershed, at the beginning of the rainfall, is between 0.25 and 0.38. The discharge parameters assigned within for calculating design hydrographs are based upon assuming high pre-precipitation and fairly correspond to the SCS CN-III method.

04.04.02.03 Design precipitation

For modelling the 100-year event, rainfall durations of 1 to 9 hours were applied. The analyses of the hydrographs for the events with return periods of 300 years and 1,000 years were both done for a rainfall event with a duration of 2 hours as the 100-year event showed that those durations generates the highest peak flow. The appropriate rainfall intensities were extrapolated from Figure 4. Further hydrological analyses are planned for the detailed design phase, at which point more scenarios will be considered. The current analysis shall provide an initial, but sound approximation of flood volumes resulting from heavy rainfall events. Preliminary Design Hydrographs

The hydrographs were calculated taking rainfall with different durations into account. Basic data used are described in chapter 04.04.02. The modeling approach was described within chapter 04.04.01. Figure 5 shows preliminary hydrographs for different precipitation durations and a return period of 100 years that can be expected at the fan apex of Cougar Creek. Figure 6 shows preliminary design hydrographs for return periods of 300 and 1,000 years, for precipitation duration of 2 hours.

Table 5 summarizes peak discharges derived from preliminary and supplementary hydrological analysis. Values referring to a return period of 10 years and 1,000 years are both derived by extrapolation of precipitation data using the Gumbel distribution. Minimal peak discharges shown in Table 5 represent more advantageous run-off conditions, the maximum values are rather conservative.

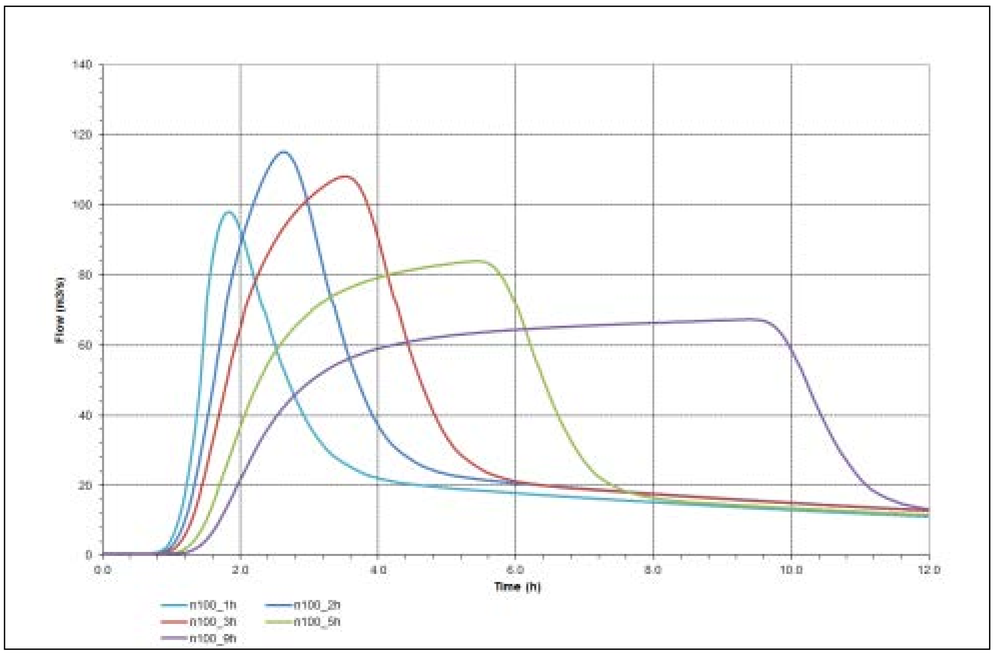

Figure 5: Design hydrographs at the fan apex, return period = 100 years, different durations

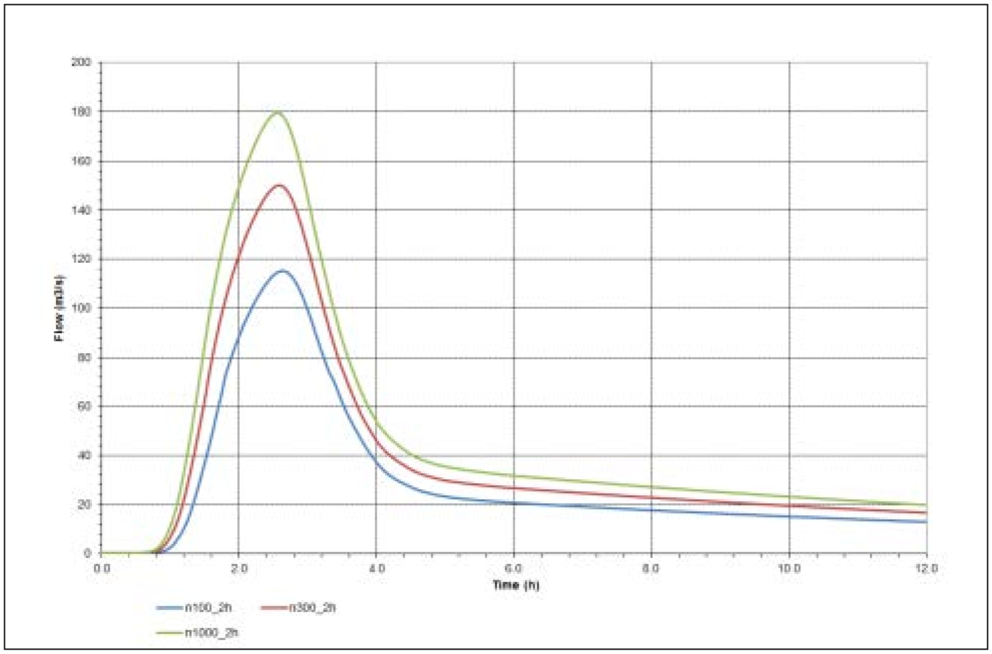

Figure 6: Design hydrographs at the fan apex, return periods = 100, 300 and 1,000 years, duration = 2 hours

Table 5: Hydrological data derived from the precipitation – discharge calculations

Return Period [years]	Peak flow (min) [m3/s]	Peak flow (max) [m3/s]
10	50	60
100	100	115
300	120	145
1,000	140	180
10,000	180	230

04.05 Estimation of Channel Capacity

04.05.01 Hydraulic Considerations

For calculating the hydraulic capacity of the channel flowing through the developed area, the code HEC-RAS 4.1 was used. HEC-RAS is a well-established, one-dimensional, hydraulic calculation code applicable to open channel structures. It was also developed by the U.S. Army Corps of Engineering, Hydrologic Engineer Center. Additional information can be found on the homepage (http://www.hec.usace.army.mil/).

Channel geometry was taken into account according to drawings provided by ISL Engineering (2013). Results are shown in Table 6 and Figure 7.

The hydraulic channel capacity is sufficient to discharge water up to 150m³/s which corresponds to an event-return period of approximately 1,000 years. The calculation is limited to (a) a pure water discharge and (b) to a stable channel bed without sedimentation and without erosion.

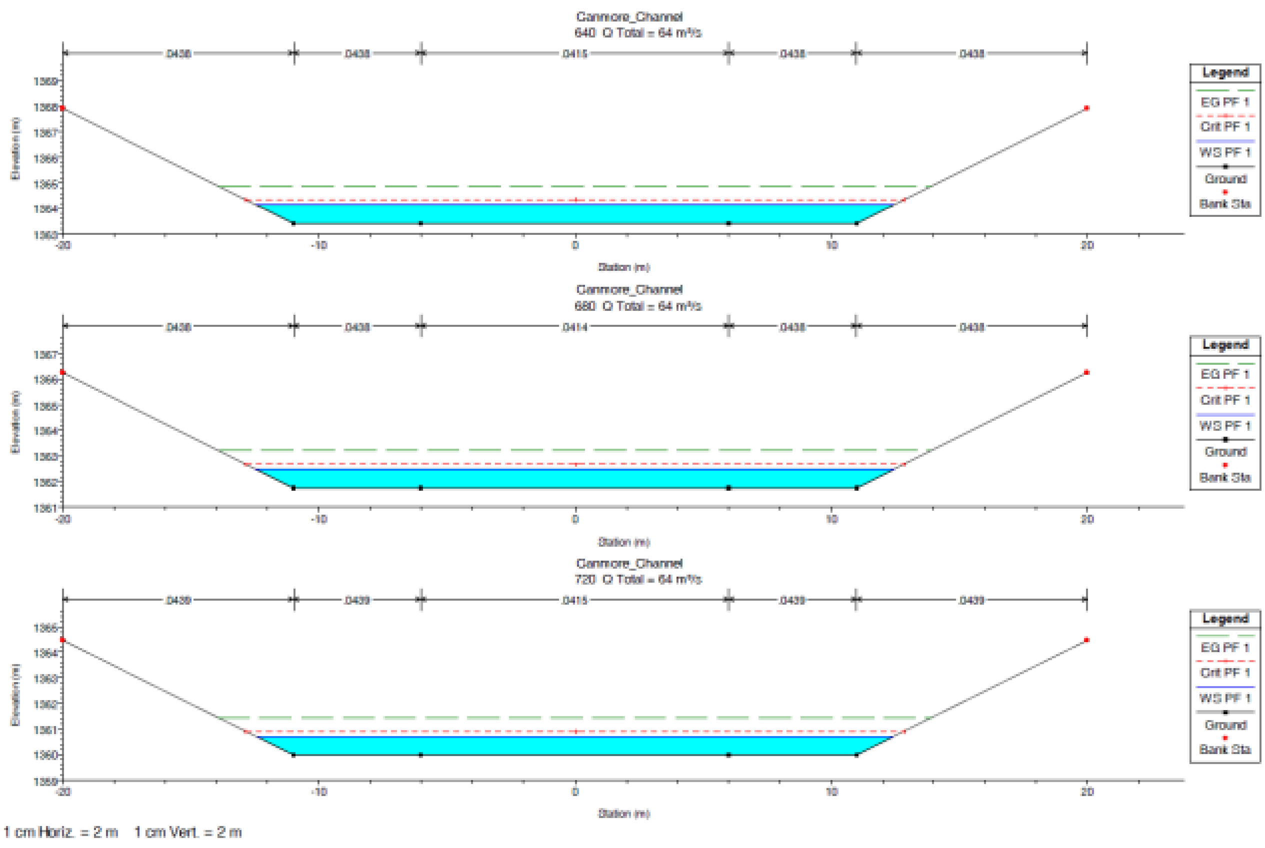

Figure 7: Channel Profiles according to the drawings of ISL Engineering Drawing Nr.: 1344 S 01

Table 6: Hydraulic Capacity of the Channel of Cougar Creek at the Town of Canmore

River Station	Q Total (m3/s)	Min. Channel Elevation (m)	Flow Depth (m)	Velocity Total (m/s)	Manning roughness coefficient	Energy Grade Elevation (m)	Energy grade Slope (m/m)	Flow Area (m2)	Froude
640	**64**	1363,42	0,73	3,75	0,043	1364,86	0,043057	17,08	1,44
	80	1363,42	0,83	4,09	0,042	1365,1	0,043063	19,56	1,49
	100	1363,42	0,94	4,46	0,042	1365,37	0,043037	22,4	1,53
	120	1363,42	1,04	4,8	0,041	1365,63	0,043051	25,02	1,57
	140	1363,42	1,13	5,09	0,041	1365,88	0,043001	27,48	1,6
	160	1363,42	1,22	5,37	0,041	1366,11	0,04302	29,79	1,63
	180	1363,42	1,3	5,63	0,04	1366,33	0,043027	31,99	1,66
680	**64**	1361,75	0,74	3,68	0,043	1363,19	0,040596	17,39	1,41
	80	1361,75	0,84	4,02	0,042	1363,42	0,040692	19,89	1,45
	100	1361,75	0,95	4,39	0,042	1363,69	0,040742	22,77	1,49
	120	1361,75	1,05	4,72	0,041	1363,94	0,040811	25,43	1,53
	140	1361,75	1,15	5,02	0,041	1364,19	0,04089	27,9	1,56
	160	1361,75	1,24	5,29	0,041	1364,42	0,040986	30,23	1,59
	180	1361,75	1,32	5,55	0,04	1364,64	0,041054	32,45	1,62
720	**64**	1359,97	0,71	3,85	0,043	1361,43	0,047033	16,63	1,5
	80	1359,97	0,81	4,19	0,042	1361,67	0,046597	19,09	1,54
	100	1359,97	0,92	4,56	0,042	1361,95	0,046184	21,92	1,58
	120	1359,97	1,02	4,89	0,042	1362,21	0,045831	24,55	1,61
	140	1359,97	1,11	5,18	0,041	1362,45	0,045538	27	1,64
	160	1359,97	1,2	5,45	0,041	1362,69	0,045266	29,34	1,66
	180	1359,97	1,28	5,7	0,041	1362,91	0,045047	31,55	1,69

04.05.02 General Design Considerations

Given the current situation of the channel, the hydraulic capacity is not the limiting factor, but the resistance of the channel bed to erosion is, as well as the limited stability of the channel slopes. The slopes are reinforced by means of cable-concrete mats. Steel-cables are connecting single concrete-bodies, forming a mat. The technical documents enclosed with the drawings show a design flow depth of 0.7 m for a discharge of 64m³/s, taking into account the capacity of the culverts at Highway 1 and Bow Valley Trail. The maximum design water level at Elk Run Boulevard is indicated at 3.4m, due to back water effects, resulting from the high level of the culvert bottom. This is according to the design drawings provided by ISL Engineering Ltd. Low stability of the channel bed, as well as high likelihood of flow concentration on the outside bends, which could result in an overload of the cable-concrete mats, can limit the capacity and overall stability of the channel.

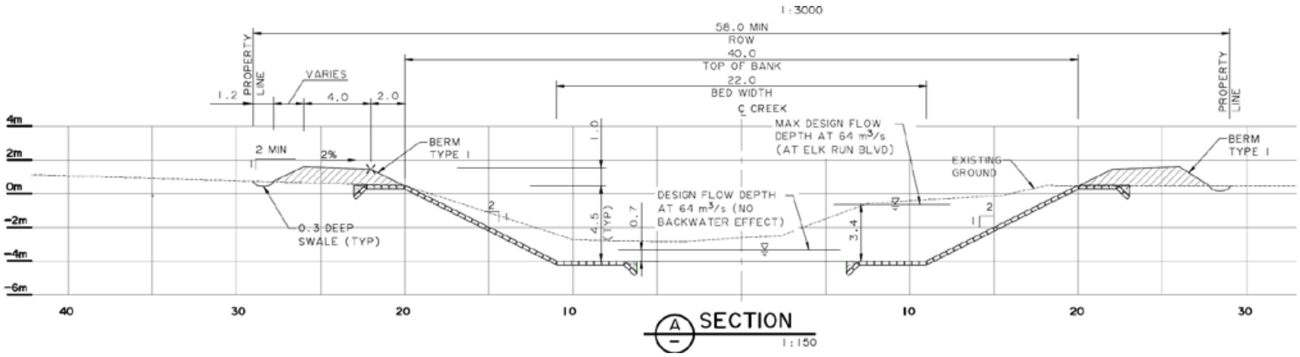

Figure 8: Design of the channel reinforcement (ISL Engineering, 2013)

04.06 Hydraulic Considerations and Limitations at Culverts

04.06.01 Collection of existing Hydraulic Calculations

As investigated by BGC Engineering Ltd., the following capacities at existing culverts are to be considered:

Table 7: Culverts listed by BGC Engineering

Table 2-3. Cougar Creek Road Crossings

Crossing	Description
Elk Run Boulevard (Photo 2-1)	• elliptical, multi-plated steel culvert • design drawings by Engineering Associates Canada (1982) show the culvert is approximately 6.45 m high, 9.5 m wide and 29 m long with a gradient of 5% • capacity = 160 m^3/s (CH2M HILL, 1993a) • concrete wingwalls at 45° are located at both the inlet and outlet, and extend out about 5 m • concrete cutoff wall extends about 1.4 m below the culvert invert. • The culvert was rehabilitated in 2012. Design drawings by Associated Engineering (2012) show the installation of an invert protection plate, repairs to the cutoff wall and placement of a concrete liner at the base of the culvert.
Highway 1 (Photo 2-2)	• three concrete box culverts: 2.44 m wide x 2.75 m high x 64 m long • constructed in 1967 • capacity = 64 m^3/s (CH2M HILL, 1993a) • culverts installed with a concrete apron and wingwalls
Highway 1A (Photo 2-3)	• three concrete box culverts: 2.44 m wide x 2.75 m high x 20 m long • constructed in 1967 • capacity = 64 m^3/s (CH2M HILL, 1993a) • culverts installed with a concrete apron and wingwalls
CP Railway	• three concrete box culverts: 3 m wide x 1.55 m high x 5.8 m long (Hydroconsult, 1999) • 2 x 900 mm CSP culverts[1]

[1] The culverts were installed following the May 1990 flood event, which was the result of the failure of the upstream rockfill dam (see Section 3.3).

04.06.02 Back-Calculation, Culvert at Elk Run Boulevard, Flood of June 2013

Using the code HEC-RAS, we performed hydraulic calculations at Elk Run Boulevard to obtain a better understanding of the hydraulic situation that occurred at the culvert of Elk Run Boulevard during the flood event of June 2013. Videos and photographs of the extreme situation were analyzed to help the back calculations.

Within the calculations, gravel accumulation at the inflow and outflow was taken into account. For the culvert, the Manning coefficient was set to a value assuming that gravel transport through the culvert increases turbulence. Because of high flow velocities during the peak discharge, no gravel will deposit in the culvert. The gravel which was found in the culvert after the June 2013 flood, was very likely deposited during a late stage of the event, as the flow was strongly subsiding.

The photo taken on the 20th of June 2013 (see Figure 9), is showing a situation that corresponds closely with the hydraulic calculation shown in Figure 11. A plausible discharge of 120m³/s can then be assigned to this specific situation. This comparison helps to get a rough calibration of the maximum flood-discharge of the event of June 2013. This analysis can be used to calibrate and check the discharge values derived within the supplementary hydrological analyses.

Figure 9: Extreme Inflow Situation on the 20th of June 2013

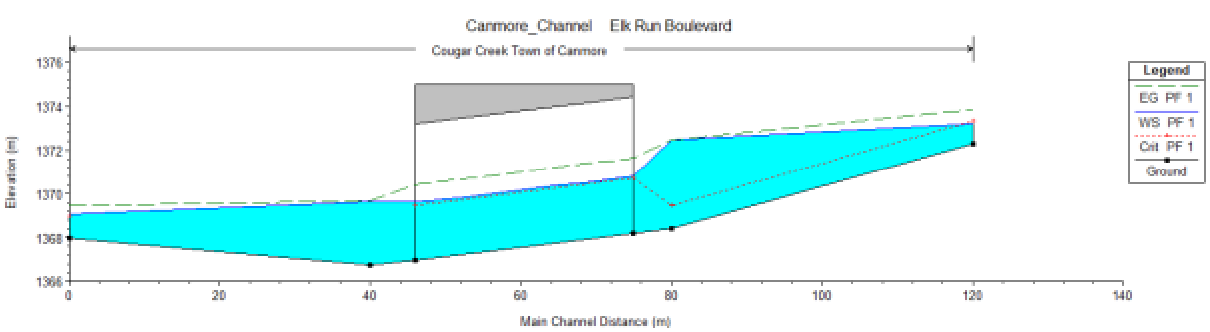

Figure 10: Situation at the Elk Run Blvd. with a flow rate of 75 m³/s, taking gravel accumulation into account

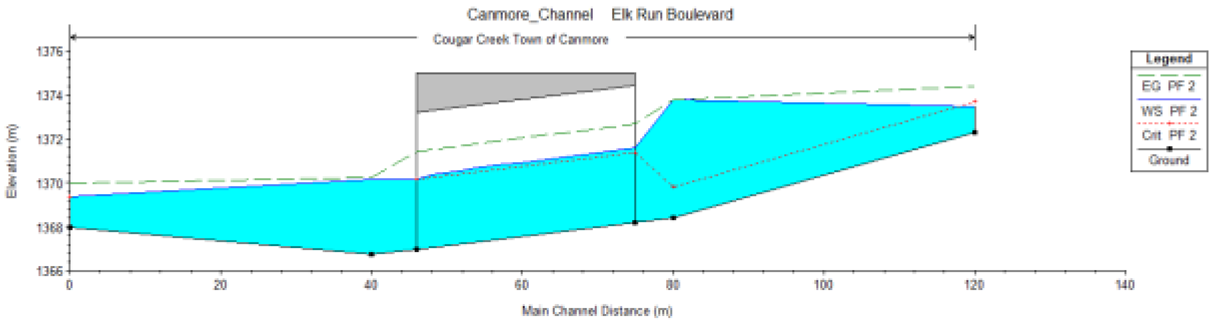

Figure 11: Situation at the Elk Run Blvd. with a flow rate of 120m³/s, taking gravel accumulation into account

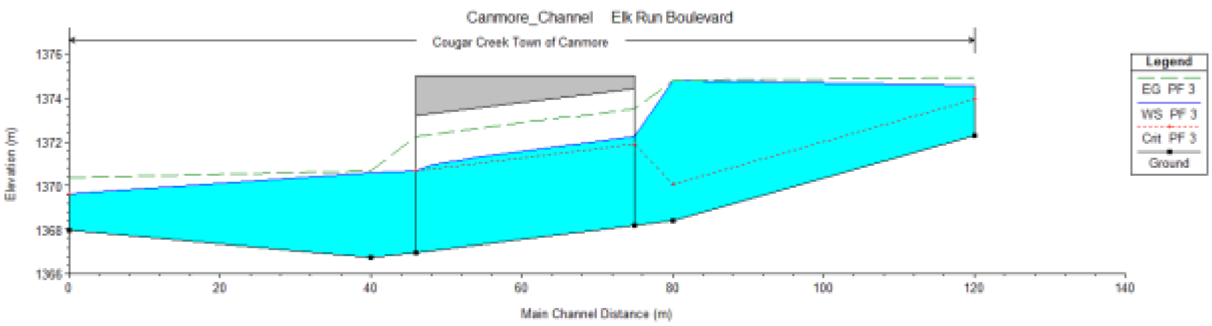

Figure 12: Situation at the Elk Run Blvd. with a flow rate of 160m³/s, taking gravel accumulation into account

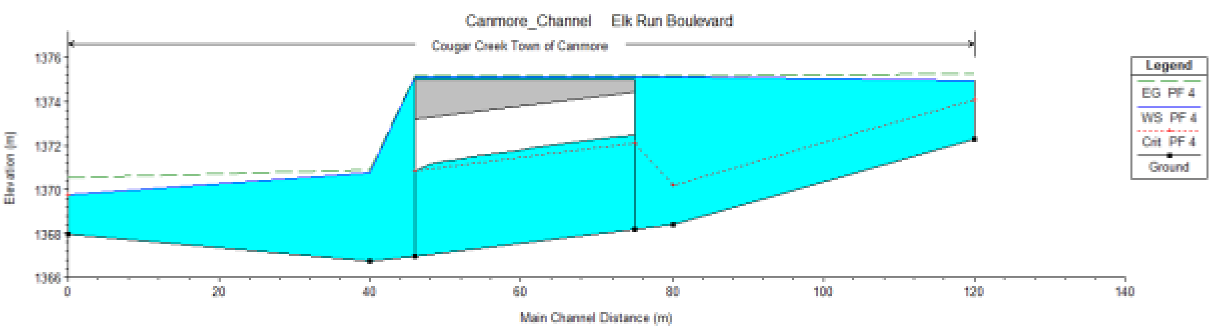

Figure 13: Situation at the Elk Run Blvd. blocked due to gravel accumulation

04.07 Design Debris Volumes

Estimating the gravel-debris being transported within floods is probably the most challenging issue during preparation of basic data for design work. The amount of gravel being transported is being affected by small, but very relevant, changes of material parameters, as well as changes in the exact flood characteristics. We recommend that more detailed transport-calculations should be done during the detailed design phase. For the current option analysis, a design debris volume of 175,000 m³, passing Station Km 2+500, was taken into account for the conceptual design of the gravel retention structure of Option C. This volume is corresponding to the numbers of BGC Engineering for a 300-1,000 years return period flood event due to a landslide dam outbreak.

05 OPTIONS

05.01 Overview

alpinfra developed two main strategies for long term mitigation measures as a first draft, sediment retention and debris flood retention. These strategies were accomplished through several potential placements of structures. To refine those options, a coordination workshop was held on the 25th of July 2014 with key stakeholders. Revisions for a second draft were made in close coordination with those stakeholders, based upon different mitigation strategies and drawings prepared by **alp**infra. The following options were selected and renamed for further design work.

Option A:

Debris-flood retention at station KM 2+900, leading to (a) highly reduced, remaining peak discharges downstream of the structure and (b) highly reduced debris mobilization in the channel. The conceptual structure height is 34m, calculated from the existing channel bed level to the crest of the structure. Additional structures were developed and preliminary designed to ensure that the remaining and highly reduced discharge flows into the existing and reinforced channel.

Option B:

Debris-flood retention at station KM 2+500, leading to (a) highly reduced, remaining peak discharges downstream of the structure and (b) highly reduced debris mobilization in the channel. The conceptual structure height is 24m, calculated from the existing channel bed level to the crest of the structure. Additional structures were designed to ensure that the remaining and reduced discharge will flow into the existing and reinforced channel.

Option C:

Gravel-debris retention at station KM 2+450, without flood retention. The conceptual structure height is 11m calculated from the existing channel bed level to the crest of the structure. Additional structures were designed to ensure the following: (a) that the un-retained flood-discharge will flow into the existing and reinforced cannel, (b) that the channel banks and the channel bed is protected against erosion by clear water flood discharge, and (c) that the gravel, accumulated by the flood discharge at the channel, is getting retained at a second retention structure at station KM 0+720.

Input Hydrographs

The input hydrographs are shown and explained in chapter 04.04.

05.02 Option A – Debris Flood Retention Embankment Dam at Station 2+900

Related preliminary drawings are attached as follows:

Drawing Nr.:	Content	Type	Scale / Size
16494-OPT.A-001	Overview Map	Site Map	1:1,000 279x1064mm
16494-OPT.A-010	Flood Retention Dam, Station KM 2+900	Site Map	1:500 279x864mm
16494-OPT.A-011	Flood Retention Dam, Station KM 2+900	Cross Section 01	1:250 432x1295mm
16494-OPT.A-012	Flood Retention Dam, Station KM 2+900	Cross Section 02	1:250 432x1095mm
16494-OPT.A-013	Diverting Structure – Station KM 2+360	Site Map Length Section	1:500, 1:250 279x1095mm

05.02.01 Description

The main structure of Option A, a debris flood retention embankment dam, is placed at the apex of the fan, respectively the mouth of the creek. The location corresponds with the site where the temporarily installed debris net is placed.

The flood retention dam is designed as rock/earth-fill embankment dam. The “support bodies” of the dam are designed with inclinations of 2h:1v at the downstream slopes and 1.75h:1v at the stepped upstream slopes. The rock-fill core is planned to be constructed by compacting processed crushed grain (range of diameters 0.1 to 300mm) with layer thicknesses of around 60cm. Compaction to 100% proctor density shall be done by using vibratory roller compactors with a mass of at least 15 tons. The grain size distribution-bands need to be designed according to fulfill all relevant filter criteria. The upstream embankment is planned with one or two berms to decrease the general slope for stability reasons, most relevant during rapid draw down. The structure is planned to be equipped with a central sealing wall, made out of reinforced concrete. The outflow-structure consists of an open inflow-structure with side walls, an open outflow structure with side walls, both connected by a closed box shaped outlet structure (tunnel box profile). The inflow structure is equipped with a debris rake which filters out and retains wooden debris and gravel. The height of the rake is set to a level that allows the retention of the design gravel volume. The opening is planned to be equipped with a rigid throttle made out of a simple, but stiff steel plate.

Because seepage has to be considered, a grout curtain needs to be established at the bottom of the structure, connected with the footing of the seal wall and reaching into the abutments at the creek flanks. The grout curtain will reduce the flow-rates due to seepage, to avoid damage due to suffusion, inner erosion and piping, all potentially resulting in slope failure of the downstream embankment. It is very likely that grouting of OPC-based slurries through boreholes or a multiple sleeve pipe system (MPSP) is feasible. Alternatively intersecting concrete piles with additional rock-grouting can be taken into account, as well as intersecting jet-grouting piles jetted “fresh in fresh”. The grout procedure needs to be designed in detail on the basis of more extensive geotechnical investigations and by experienced grouting experts. The contractual regulations for grouting need to be kept flexible to be able to react on unforeseen but technically manageable conditions.

05.02.02 Hydrologic and Hydraulic Function

In case of a flood event, the reservoir will fill up. The throttle placed at the upstream side of the opening is set to a specific width and height such that the remaining peak discharge is reduced to a maximum of 50m³/s to 60m³/s. The resulting storage volume needed for retaining a flood with a 1,000 year return period induced by a

2h rainfall is approximately 650,000m³, at a storage level of 30m. The throttle needs to be ventilated to avoid cavitation. The following figures display the storage curve of the basin for different event scenarios over time. The upper graph shows the storage volume and the storage elevation, the graph below shows the inflow and outflow hydrographs. Figure 18 displays the change in storage volume in relation to structure height.

If the storage level of 30m is getting overtopped, the spillway is designed to discharge all possible additional flood discharge. At this stage, no further retention takes place. The spillway discharges into a stilling-basin, dissipating energy of the high energetic spillway discharge flow. The lining of the spillway is made out of rocks set into a reinforced concrete bed or alternatively by means of reinforced concrete.

In case of the blockage of the throttle, an emergency bypass is necessary. This is planned to be realized using a steel pipe protected by the debris rake and an additional rake in front of the pipe inlet. The emergency by-pass is opened by a hand or motor driven shut-off device, installed at the middle or the downstream-end of the pipe.

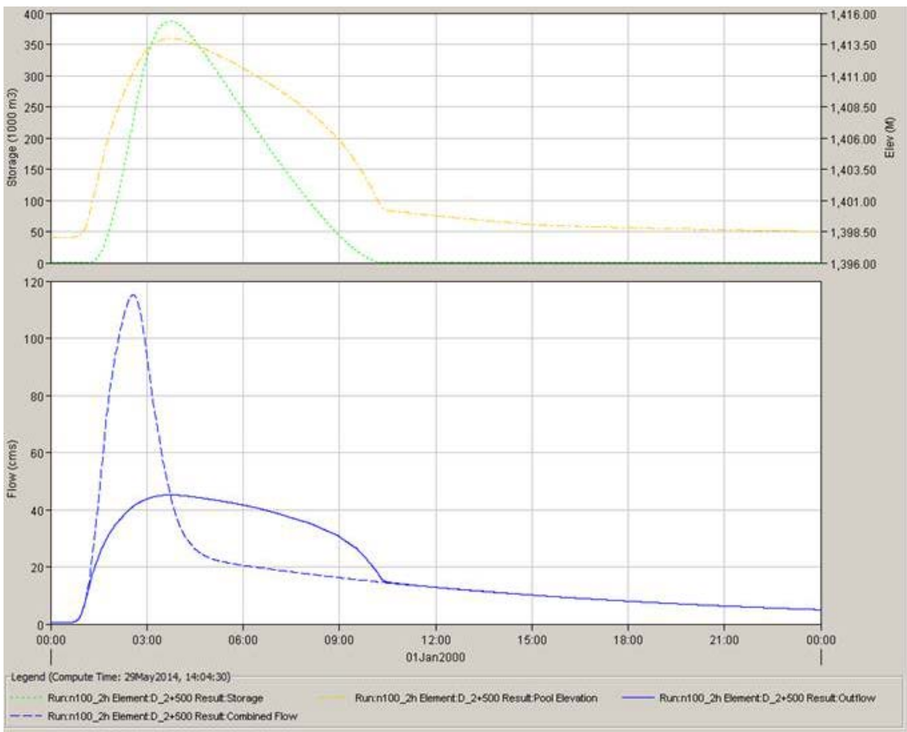

Figure 14: Inflow and outflow hydrographs 100 year flood, 2h rainfall for a flood retention structure between stat. 2+500 and 2+900

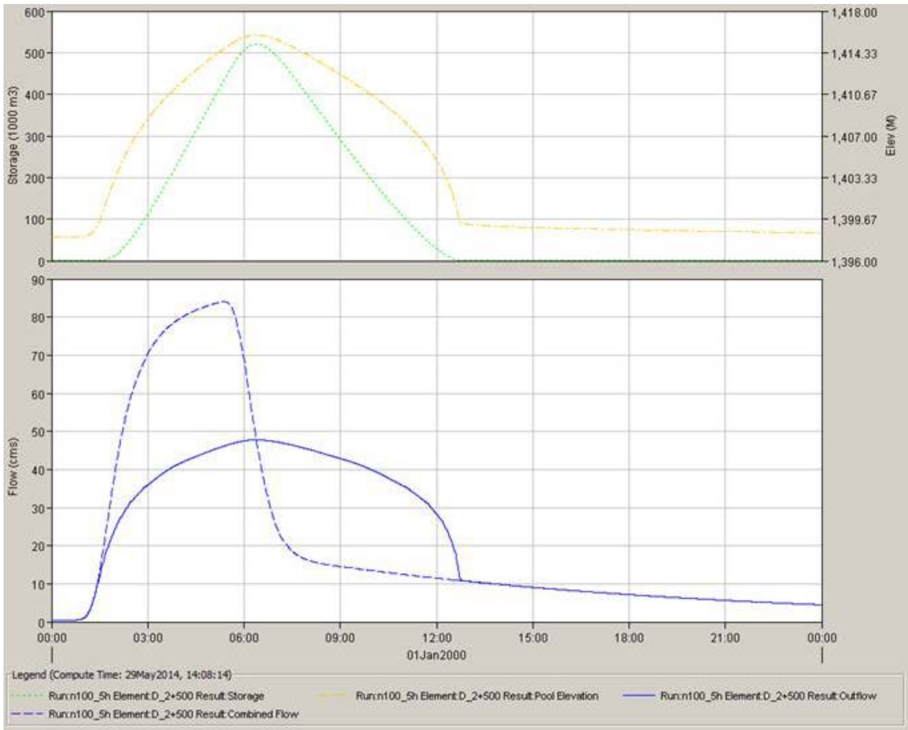

Figure 15: Inflow and outflow hydrographs 100a-flood 5h rainfall for a flood retention structure between stat. 2+500 and 2+900

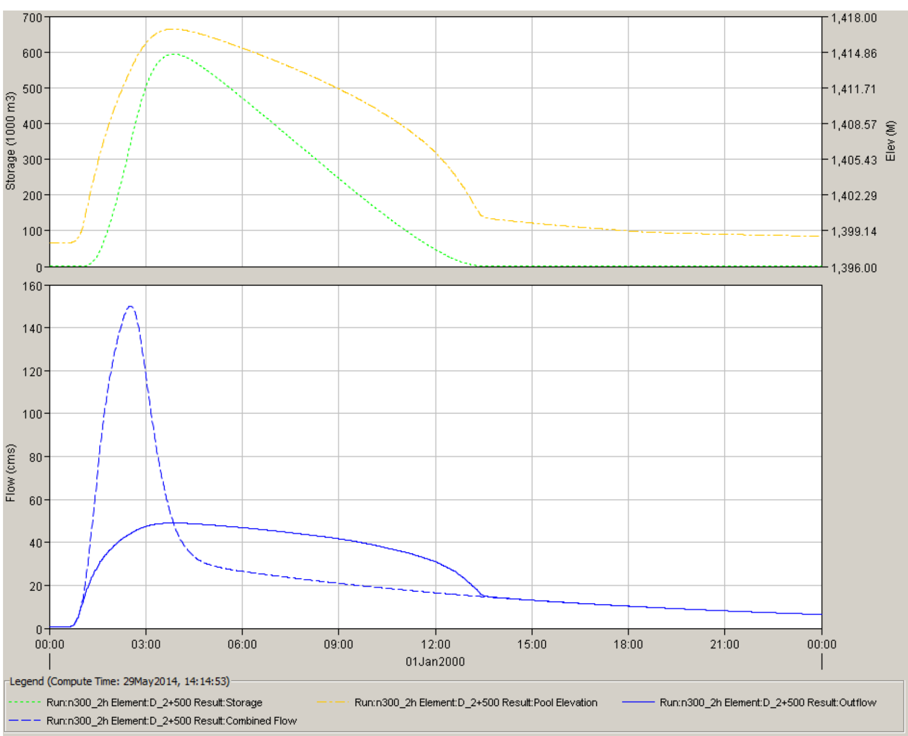

Figure 16: Inflow and outflow hydrographs 300a-flood 2h rainfall for a flood retention structure between stat. 2+500 and 2+900

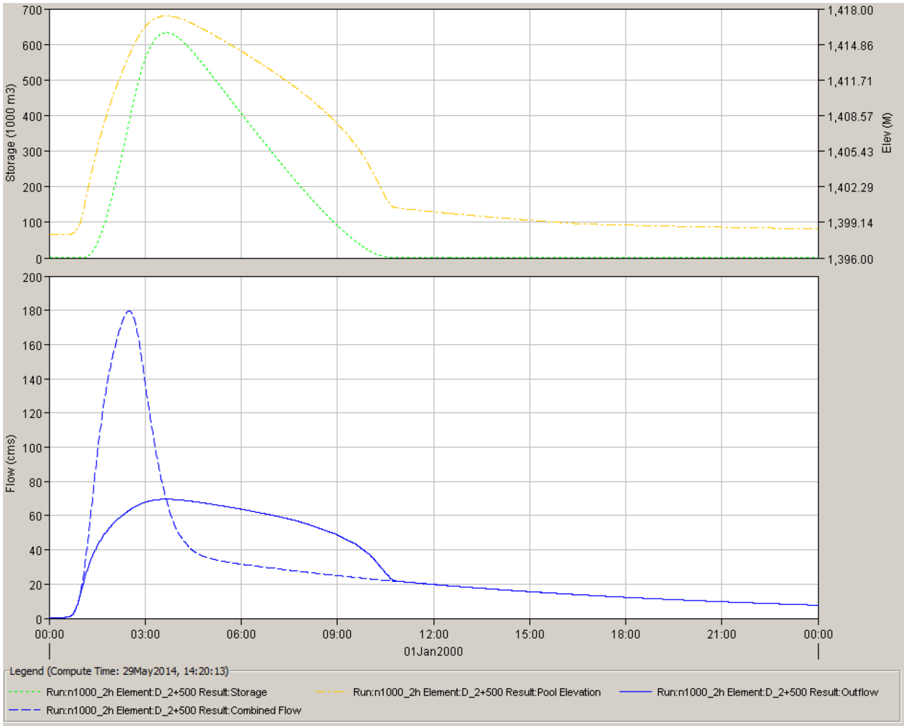

Figure 17: Inflow and outflow hydrographs 1,000a-flood 2h rainfall for a flood retention structure between stat. 2+500 and 2+900

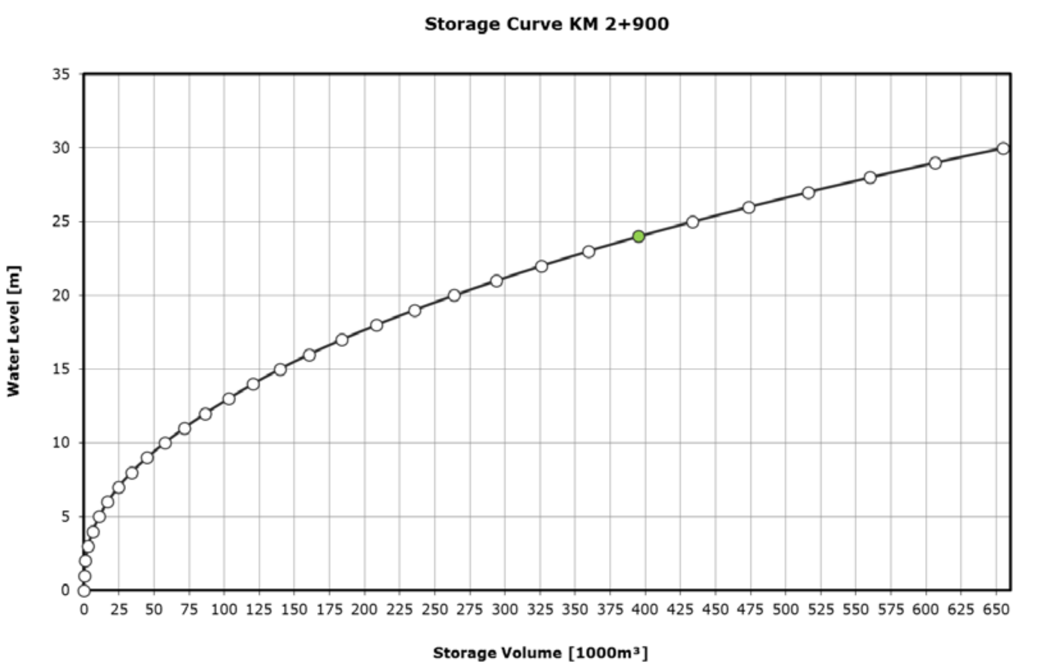

Figure 18: Storage curve for retention structures at station KM 2+900

05.02.03 Complementary Mitigation Measures

A deflection wall with two wings on each bank of the creek is planned to divert the remaining flood discharge into the channel. This structure is placed at Station KM 2+366. The creek section between the diverting structure and the existing channel is planned to be excavated to the profile of the already existing channel. Ground sills are planned to be constructed as grade control structures at the section where no concrete mats are installed. If remaining peak discharges of more than 60m³/s are planned, the channel needs to be upgraded with small grade control structures.

Within detail design, more investigations on the gravel rake andthe shape of openings are required.

05.02.04 Mass Estimation

Table 8: Quantity Estimation, Flood Retention Structure Option A

Flood Retention Structure 2+900 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
00	**Open Removal and Dyke Excavation**		
00 01	Open Removal Footprint	18,000	m³
00 02	Rock Dykes by means of Cutting for Seal Wall Abutments	2,300	m³
00 03	Rock/Conglomerate removal for Dam Abutments	14,000	m³
00 04	Excavation Footings Plunge Pool	660	m³
00 05	Excavation Footings Inlet	130	m³
00 06	Excavation Footing Seal Wall	1,200	m³
		0	m³
01	Rock/Earth Fill Core		
01 01	X-Section minus Outlet and Wall	145,000	m³
01 02	Overheight for adequate compaction of outer lining	7,300	m³
02	**Filter**	**35,000**	**m³**
03	**Seal Wall with Footing**		
03 01	Wall	2,900	m³
03 02	Footing	230	m³
03 03	Reinforcement	310	T
04	**Lining - Upstream Embankment**	**0**	**m³**
04 01	Stones	0	m³
04 02	Concrete	0	m³
04 03	Reinforcement AQ100	0	T
05	**Lining - Blast Drain // Spillway**		
05 01	Stones + Plungepool	3,570	m³

Flood Retention Structure 2+900 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
05 02	Concrete	1,000	m³
05 03	Reinforcement	31	t
06	**Backfilling of Concrete Elements**		
06 01	Seal Wall Abutments	1,500	m³
06 02	Footings Plunge Pool	560	m³
06 03	Footings Inlet	110	m³
06 04	Footing Seal Wall	880	m³
07	**Outlet incl. Plunge Pool**		
07 01	**Concrete sum**	**4,720**	**m³**
07 01 01	Inflow Sidewalls	360	m³
07 01 02	Outflow Sidewalls	520	m³
07 01 03	Inflow Frontwall	180	m³
07 01 04	Outflow Frontwall	210	m³
07 01 05	Inflow small Walls	30	m³
07 01 06	Baseplate incl. Plungepool	2900	m³
07 01 07	Frame Walls Plunge Pool	400	m³
07 01 08	Footings Plunge Pool	60	m³
07 01 09	Footings Inlet	30	m³
07 01 10	Support Beams	30	m³
07 02	Reinforcement 0,1 t/m³	472	t
07 03	Protective Stone Lining Bottom	1,200	m³
07 04	Protective Stone Lining Sides	600	m³
08	**Debris Rake - Steel**		
08 01	Rake-Beams (IPE 300)	30	t
08 02	Support Beams (HEB300)	10	t
09	**Emergency Bypass**	**0**	**m³**
09 01	Steel Pipe 610mm/7,1mm	9,5	t
09 02	Shut-off Device	1	Pc
10	**Grout Curtain**		
10 01	OPC-Based Grouting ~30kgOPC/m² Final Spacing of P, S + T Bore Holes 0,75m	9,000	m²
10 02	Grouting of Contact Seal Wall/Grout Curtain	1,200	m²

Grade Control Channel - Option A - Quantity Estimation			
01	**Diverting Structure incl. Ground Sill**		
01 01	Excavation	1,940	m³
01 01 01	Dyke for Diverting Wall	1,000	m³
01 01 02	Shoring for Dykes	940	m²
01 02	Wall and Ground Sill		
01 01 01	Concrete	760	m³
01 01 02	Reinforcement	80	t
01 01 03	Protective Stone Cover Overflow-Section	52	m²
02	**Excavation of Channel until Stat. 2+230**		
02 01	Excavation		
02 01 01	Excavation Channel-Section	17,000	m³
03	**Grade Control Channel until Stat. 2+230**		
03 01	Excavation and Backfilling		
03 01 01	Open Excavation 1:1 - Ground Sills	3,000	m³
03 01 02	Backfilling	2,000	m³
03 01 03	Removal of Mats at Ground Sills	1,000	m²
03 02	Ground Sills		
03 02 01	Concrete	400	m³
03 02 02	Reinforcement	40	t
03 02 03	Protective Stone Overflow-Section	200	m²

05.03 Option B – Flood Retention Embankment Dam at Station 2+500

Related preliminary drawings are attached as follows:

Drawing Nr.:	Content	Type	Scale / Size
16494-OPT.B-001	Overview Map	Site Map	1:1,000 432x864mm
16494-OPT.B-010	Flood Retention Dam, Station KM 2+900	Site Map	1:500 559x 1064mm
16494-OPT.B-011	Flood Retention Dam, Station KM 2+900	Cross Section 01	1:250 432x 1295mm
16494-OPT.B-013	Diverting Structure, Station KM 2+360	Site Map Length Section	1:500, 1:250 279x1095mm

05.03.01 Description

Option B is functionally and structurally similar to Option A. The structures differ in height, width and placement. Option B is placed further downstream in the area referred to as “No-Man´s Land”. The area is characterized by a wide torrent-bed, extending between the “Kame-Terrace” on the orographic right bank and a terrace followed by a bedrock flank on the left bank (see Drawing Nr. 16494-OPT.B-010). Taking into account the same retention-function as Option A, the preliminary resulting height of the structure is 24m from the existing torrent bed to the crest. It is around 10m lower than Option A, but wider. The outlet structure is the same as for Option A.

The storage curve for Option B is shown in Figure 19.

05.03.01.01 Hydrologic and Hydraulic Function

The hydrologic and hydraulic functions, as well as the hydraulic structures, are exactly the same as those described for Option A. However, the spillway and the stilling basin of Option B are wider, and the spillway is 2m deep instead of 3m. The box-shaped outlet structure between the inflow and outflow-structure is shorter.

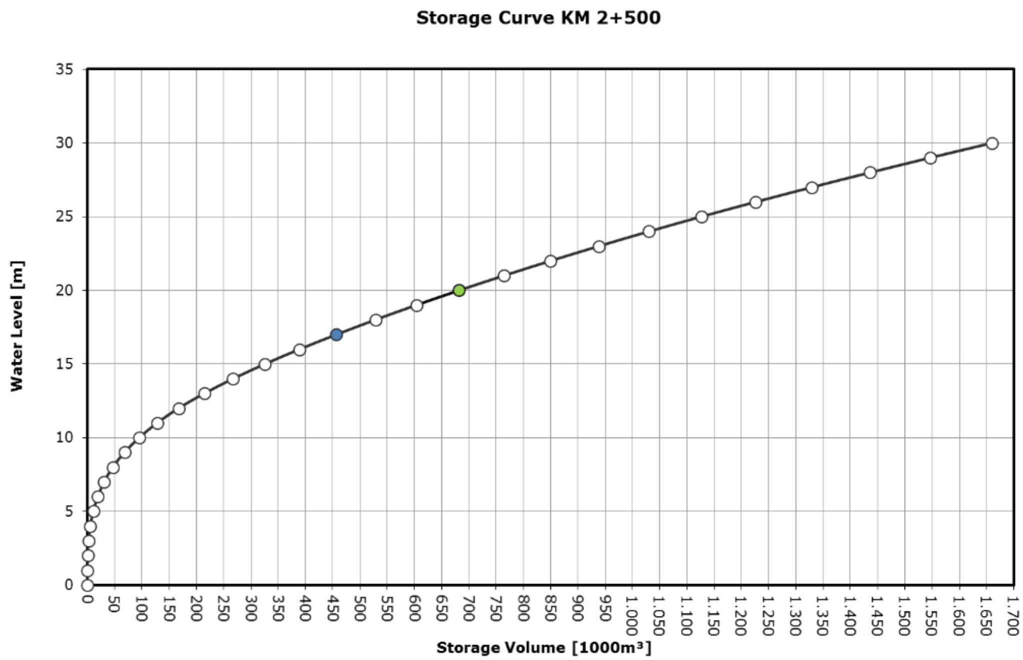

Figure 19: Storage curve for retention structures at station KM 2+500. Level-point green 680,000m³, Level-point blue 460,000m³ = 16m

05.03.02 Additional Measures

Like Option A, a deflection wall with two wings on each bank of the creek is planned to divert the remaining flood discharge into the channel. This structure is planned to be placed at Station KM 2+366. The section between the diverting structure and the existing channel is planned to be excavated and equipped with ground sills as grade control.

During detailed design, more in-depth investigation of the gravel rake and the shape of openings is recommended.

05.03.03 Mass Estimation

Table 9: Quantity Estimation, Flood Retention Option B

Flood Retention Structure 2+500 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
00	**Open Removal and Dyke Excavation**		
00 01	Open Removal Footprint	35,900	m³
00 02	Rock/Conglomerate Dykes by means of Cutting for Seal Wall Abutments	10,600	m³
00 03	Rock/Conglomerate for Dam Abutments	7,900	m³
00 04	Excavation Footings Plunge Pool	1,700	m³
00 05	Excavation Footings Inlet	130	m³
00 06	Excavation Footing Seal Wall	8,500	m³

Flood Retention Structure 2+500 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
01	**Rock/Earth Fill Core**		
01 01	X-Section minus Outlet and Wall	215,000	m³
01 02	Overheight for adequate compaction of outer lining	18,700	m³
02	**Filter**	25,000	m³
03	**Seal Wall with Footing**		
03 01	Wall	6,900	m³
03 02	Footing	1,400	m³
03 03	Reinforcement	830	t
04	**Lining - Upstream Embankment**		
04 01	Stones	0	m³
04 02	Concrete	0	m³
04 03	Reinforcement AQ100	0	t
05	**Lining - Blast Drain // Spillway**		
05 01	Stones + Plungepool	7,000	m³
05 02	Concrete	2,000	m³
05 03	Reinforcement	62	t
06	**Backfilling of Concrete Elements**		
06 01	Seal Wall Abutments	6,900	m³
06 02	Footings Plunge Pool	1,397	m³
06 03	Footings Inlet	110	m³
06 04	Footing Seal Wall	6,178	m³
07	**Outlet incl. Plunge Pool**		
07 01	**Concrete sum**	**5,460**	**m³**
07 01 01	Inflow Sidewalls	370	m³
07 01 02	Outflow Sidewalls	410	m³
07 01 03	Inflow Frontwall	180	m³
07 01 04	Outflow Frontwall	180	m³

Flood Retention Structure 2+500 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
07 01 05	Inflow small Walls	30	m³
07 01 06	Baseplate incl. Plungepool	3580	m³
07 01 07	Frame Walls Plunge Pool	440	m³
07 01 08	Footings Plunge Pool	140	m³
07 01 09	Footings Inlet	100	m³
07 01 10	Support Beams	30	m³
07 02	Reinforcement 0,1 t/m³	546	t
07 03	Protective Stone Lining Bottom	850	m²
07 04	Protective Stone Lining Sides	400	m²
08	**Debris Rake - Steel**		
0801	Rake-Beams (IPE 300)	30	t
08 02	Support Beams (HEB300)	10	t
09	**Emergency Bypass**		
09 01	Steel Pipe 610mm/7,1mm	5	t
09 02	Shut-off Device	1	Pc
10	**Grout Curtain**		
10 01	OPC-Based Grouting ~30kgOPC/m² Final Spacing of P, S + T Bore Holes 0,75m	13,800	m²
10 02	Grouting of Contact Seal Wall/Grout Curtain	2,500	m²

Grade Control Channel - Option B - Quantity Estimation			
01	**Diverting Structure incl. Ground Sill**		
01 01	Excavation	1,940	m³
01 01 01	Dyke for Diverting Wall	1,000	m³
01 01 02	Shoring for Dykes	940	m²
01 02	Wall and Ground Sill		
01 01 01	Concrete	760	m³

Grade Control Channel - Option B - Quantity Estimation			
01 01 02	Reinforcement	80	t
01 01 03	Protective Stone Overflow-Section	52	m²
02	**Excavation of Channel until Stat. 2+230**		
02 01	Excavation		
02 01 01	Excavation Channel-Section	17,000	m³
03	**Grade Control Channel until Stat. 2+230**		
03 01	Excavation and Backfilling		
03 01 01	Open Excavation 1:1 - Ground Sills	3,000	m³
03 01 02	Backfilling	2,000	m³
03 01 03	Removal of Mats at Ground Sills	1,000	m²
03 02	Ground Sills		
03 02 01	Concrete	400	m³
03 02 02	Reinforcement	40	t
03 02 03	Protective Stone Cover - Overflow-Sections	200	m²

05.04 Option C - Gravel Retention at Station KM 2+450

Related preliminary drawings are attached as follows:

Drawing Nr.:	Content	Type	Scale / Size
16494-OPT.C-001	Overview Map	Site Map	1:1,250 559x2592mm
16494-OPT.C-010	Gravel Retention Structure, Station KM 2+450 Diverting Structure, Station KM 2+360	Site Map	1:1,000 279x736mm
16494-OPT.C-011	Gravel Retention Structure, Station KM 2+450	Length Section	1:500 279x918mm
16494-OPT.C-012	Gravel Retention Structure, Station KM 2+450	Cross Section 01 Cross Section 01	1:200 279x864mm
16494-OPT.C-013	Diverting Structure, Station KM 2+360	Length Section Cross Section	1:500, 1:250 279x1095mm
16494-OPT.C-020	Ground Sills	Regular Drawing Cross Section Length Section	1:200 279x1016mm
16494-OPT.C-030	Gravel Retention Structure, Station KM 0+720	Site Map	1:500 432x864mm
16494-OPT.C-031	Gravel Retention Structure, Station KM 2+450	Length Section	1:500 279x1016mm
16494-OPT.C-032	Gravel Retention Structure, Station KM 2+450	Cross Sections	1:150 432x559mm

05.04.01 General Description

A debris retention structure is designed to retain gravel and woody debris that has been mobilized and transported by floodwater. The gravel-debris is filtered out at the retaining structure but water will be discharged, not retained. The peak flood-discharges remain nearly unchanged.

Gravel retention protects structures downstream, like culverts, from blocking. It also prevents overtopping of the channel due to gravel aggradation and decreasing available flow-capacity.

On the other hand, the remaining clear water flood discharge will mobilize any gravel it can entrain, until saturation is reached (according to the flow-regime at a certain section). Therefore, complementary grade control structures are needed downstream of the gravel retention structure. Otherwise more erosion would take place at the channel banks and the channel bed, resulting again in problems at culverts and along the channel.

Gravel retention Structure at Station 2+450

The gravel retention structure at station KM 2+450 is designed as an embankment dam with a rock/earth-fill core. The “support bodies” are designed with a slope inclination of 2h:1v at the downstream embankment and 1h:5v at the upstream embankment slope. The upstream embankment is to be seen rather as a wall and is to be designed as a wave-breaker. The rock-fill core is planned to be constructed by compacting processed crushed gravel ranging in diameter between 0,1mm and 300mm, built in with layer thicknesses of around 60cm, using vibratory roller compactors with a mass of at least 15 tons. The grain size distribution-bands need to be designed accordingly to fulfill relevant filter criteria. The upstream embankment is planned as a stone pitching-wall set in concrete, reinforced by means of steel-mats. The head of the upstream embankment is designed vertically as a wave-breaker, needed to reduce impacts on the downstream embankment in case of a dam outbreak flood, potentially overtopping the structure.

The stone lining (stone pitching) at the upstream embankment slope reduces damage during the clearance of the basin by excavators, provides the needed static stability, and protects the slope from erosion due to flood wave impact.

Additional structural measures are required to avoid damage at the structure by a flood wave and by overtopping, in case of a completely filled basin and blockage of the outlet.

a) The seal-wall and the crest of the structure at the right abutment are increased to deflect the overtopping flood discharge away from the abutment. Side erosion and exposure of the abutment would lead to severe damage of the structure.
b) The complete downstream embankment was designed to be constructed with a stone lining made out of a stone-cover, set in a reinforced concrete bedding.
c) The stilling-basin extends for the full width of the downstream embankment to avoid erosion and collapse of the embankment in case of overtopping.

The structure is planned to have a central sealing wall, reaching 5m to 7m underneath the footprint of the structure to reduce seepage to an acceptable level. The seal wall is planned to be constructed of reinforced concrete.

The gravel filter is designed as an open structure containing 4 box-shaped openings, with a width of 5m and a height of 8m each. The openings are designed to discharge all possible floods, even if partial blockage of openings occurs. The filter structure consists of a head beam along the axis of the seal wall, bearing the upstream arranged gravel rake bars. Additional support and side walls are planned to be constructed for static reasons.

The support walls are arranged between the openings. All concrete surfaces at the filter structure, which are exposed to gravel abrasion, are planned to be covered with hard-stone lining (granite, diorite, ignimbrite or similar), or alternatively with steel plates. The spillway is placed above the filter structure, which is designed to discharge a design flood of 230m³/s. The orographic left wing of the seal wall is additionally planned to reach up to the lower terrace and it is foreseen to increase the height of the wall by 1m at the shoulder of the left creek bank. This shall avoid overtopping and erosion of fossil, buried flow channels. This, and the inclinations of all crests of the seal wall at both sides of the spillway are designed to ensure that, the remaining flow is diverted back to the spillway as well as to the filter-structure in any situation and filling stage.

Because seepage due to a complete blockage of the openings has to be considered, the seal wall was designed to reach 5m to 7m into the ground. This can best be established by an open excavation and refilling of the structure. No additional grout curtain is anticipated to be required.

05.04.02 Hydrologic and Hydraulic Function

During a flood event, the reservoir fills as gravel and wooden debris is filtered by the gravel-rake. The openings permit the free flow of the remaining flood water until the rake is completely blocked. At this stage, the structure is overtopped and gravel and water will be discharged over the spillway. The lining of the downstream embankment is designed to carry this debris laden flow and the stilling-basin at the toe of the downstream slope is designed to prevent erosion and dissipate energy.

Gravel-transport through the filter-structure will be possible during small floods and regular annual discharge. To optimize this continuity the gravel rake can be opened step by step at the interface of the inflow structure, starting with a 30cm wide gap at every second rake-beam. The initial gaps between the rake beams are set to 50cm. The capacity of the retention structure, assuming 2% aggradation, is developing with storage height as follows:

6m: 86,000m³
7m: 127,000m³
8m: 175,000m³
9m: 246,000m³
10m: 296,000m²

During detailed design, more in-depth investigations of the gravel rake and the shape of openings will be completed.

<u>Diverting Structure at Station 2+366</u>

The flood discharge needs to be diverted into the channel by means of a deflection wall consisting of two wings on each bank of the creek. This structure is planned to be placed at Station KM 2+366. The section between the diverting structure and the existing channel is planned to be excavated to the shape of the existing channel and one ground sill is to be placed and structurally connected with the wing walls.

<u>Grade Control in the channel</u>

From station Km 2+850 to the Trans-Canada Highway, the channel was upgraded by means of cable-concrete mats in combination with a debris net. The capacity of the channel was checked again within this project (see chapter 04.05). Beside other factors, the resistance of the mats against abrasion is identified to be the limiting point. Investigations on the abrasion of concrete structures at gravel-debris loaded mountain creeks were performed at the ETH Zürich, indicating limitations for the concrete mats. As a result, adequate flow and grade control structures must be established in conjunction with upstream debris-retention. Therefore ground sills are proposed to be arranged in series beginning at the diverting structure and ending upstream of Bow Valley Trail. The series of structures is interrupted at the culverts of Elk Run Blvd., Highway 1A, Bow Valley Trail, further at the Canadian Pacific Railway, and inside the second gravel retention basin at Station 0+720. The sills are shaped according to the channel shape and are fully buried. The top of the sills, the overflow section, is designed with a hard stone lining to avoid abrasion. The distance between the sills is ruled by the depth of the sills. The top of the downstream sills must not be lower than the footing of the upstream sill plus 0.5m for scour protection. 38 sills are needed to protect the channel as described. Stone pitching set in reinforced concrete is planned to protect the outer bends against undercutting and collapse of the concrete mats.

Small Gravel Retention Basin at Station 0+720

To retain the annual gravel load due to small floods and annual flow, predominantly during the snow melt season, an additional gravel retention basin is planned. The design principle is similar to the main gravel retention structure at Station 2+450, except the stilling-water basin, which in this case covers the width of the spillway only. The embankment dam is connected to the filter structure and the western embankment-slopes of Highway 1A. The preliminary dam height is roughly 6,5m at the interface to the filter structure (outlet) and is almost zero at the interface to the highway slopes.

The complete structure is planned to be equipped with a seal wall, 1.5-2m deep, built into the ground to reduce seepage to an acceptable limit. The area surrounded by the retention structure is planned to be excavated to the level of the foot of the embankment dam. Preliminary results show a retention volume of 30,000m³ to 35,000m³. The spillway and the openings are designed to cover a peak discharge of 230m³/s. A slight inclination at the crest of the structure enables the backflow to the spillway and the filter structure, if the structure would be fully filled with gravel debris and overtopped.

Additional ground sills are planned to be placed between the stilling-water basin and the western embankment-slope of the Bow Valley Trail. Another ground sill is planned to be placed at the western embankment-slope of the CPR.

05.04.03 Mass Estimation

Table 10: Quantity Estimation, Gravel retention and Grade Control, Option C

Gravel Retention Structure 2+450 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
00	**Open Removal and Dyke Excavation**	16,000	m³
01	**Rock/Earth Fill Core**	18,000	m³
02	**Filter**	3,000	m³
03	**Seal Wall with Footing**		
03 01	Wall	1,600	m³
03 02	Footing	200	m³
03 03	Reinforcement	180	t

Gravel Retention Structure 2+450 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
04	**Lining - Upstream Embankment**		
04 01	Stones	2,000	m³
04 02	Concrete	1,000	m³
04 03	Reinforcement AQ100	31	t
05	**Lining - Downstream Embankment**		
05 01	Stones + Plungepool	2,400	m³
05 02	Concrete	1,200	m³
05 03	Reinforcement	36	t
06	**Backfilling of Concrete Elements**		
06 01	Backfilling Footings	6,500	m³
07	**Outlet incl. Plunge Pool**		
07 01	**Concrete sum**	**3,840**	**m³**
07 01 01	Beams	20	m³
07 01 02	Inlet Walls side	20	m³
07 01 03	Inlet Wall upstream	30	m³
07 01 04	Support Walls	290	m³
07 01 05	Side Walls Outlet	360	m³
07 01 06			
07 01 07	Outletwall Plunge Pool	60	m³
07 02 01	Baseplate incl. Plunge Pool	2400	m³
07 02 02	Concrete Padding	410	m³
07 02 03	Footings Plunge Pool and Outlet	250	m³
07 02 04	Footings Inlet	20	m³
07 03	Reinforcement 0,1 t/m³	384	t
07 04	Protective Stone Lining Bottom	880	m²
07 05	Protective Stone Lining Sides	590	m²
08	**Debris Rake - Steel**		
08 01	Rake-Beams (IPE 300)	28	t

Gravel Retention Structure 2+450 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
08 02	Support Beams (HEB300)	12	t

Grade Control Channel - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
01	**Diverting Structure incl. Ground Sill**		
01 01	Excavation		
01 01 01	Dyke for Diverting Wall	1,000	m³
01 01 02	Shoring for Dykes	940	m²
01 02	Wall and Ground Sill		
01 01 01	Concrete	760	m³
01 01 02	Reinforcement	80	t
01 01 03	Protective Stone Cover - Overflow-Sections	52	m²
02	**Excavation of Channel until Stat. 2+230**		
02 01	Excavation		
02 01 01	Excavation Channel-Section	17,000	m³
03	**Grade Control Channel**		
03 01	Excavation and Backfilling		
03 01 01	Open Excavation 1:1 - Ground Sills	26,000	m³
03 01 02	Backfilling	21,000	m³
03 01 03	Removal of Mats at Ground Sills	7,000	m²
03 01 04	Removal of Mats at Stone Pitching	43,000	m²
03 02	Ground Sills		
03 02 01	Concrete	4,900	m³
03 02 02	Reinforcement	490	t
03 02 03	Protective Stone Cover - Overflow-Sections	2,000	m²
03 03	Stone Pitching at Curves		
03 03 01	Stones	9,000	m³
03 03 02	Concrete	4,900	m³

Grade Control Channel - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
03 03 03	Reinforcement AQ100	150	t
	Sections for Stone Pitching at Cut Bank — Length of Section [m] KM 0+445 to 0+520 — 90,00 KM 0+655 to 0+695 — 45,00 KM 0+920 to 1+015 — 95,00 KM 1+286 to 1+375 — 90,00 KM 1+915 to 2+005 — 90,00 KM 2+185 to 2+320 — 150,00		

Gravel Retention Structure 0+720 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
00	**Open Removal and Dyke Excavation**		
00 01	Dam Footprint	7,000	m³
00 02	Clearing Basin	19,600	m³
01	**Rock/Earth Fill Core**	12,000	m³
02	**Filter**	2,000	m³
03	**Seal Wall with Footing**		
03 01	Concrete - Wall	1,400	m³
03 02	Concrete - Footing	600	m³
03 03	Reinforcement	200	t
04	**Lining - Upstream Embankment**		
04 01	Stones	2,300	m³
04 02	Concrete	500	m³
04 03	Reinforcement	18	t
05	**Lining - Downstream Embankment**		
05 01	Stones	3,100	m³
05 02	Concrete	2,000	m³
05 03	Reinforcement	60	t
06	**Backfilling of Concrete Elements**		

Gravel Retention Structure 0+720 - Quantity Estimation			
Pos. Nr.	**Substructure**	**Rough Mass Calculation**	**Units**
06 01	Upstream	1,400	m³
06 02	Downstream	620	m³
07	**Outlet incl. Plunge Pool**		
07 01	**Concrete Sum**	**2,220**	**m³**
07 01 01	Beams	20	m³
07 01 02	Inlet Walls side	20	m³
07 01 03	Inlet Wall upstream	30	m³
07 01 04	Support Walls	120	m³
07 01 05	Side Walls Outlet	130	m³
07 01 06	Side Walls Plunge Pool	70	m³
07 01 07	Outletwall Plunge Pool	40	m³
07 01 08	Outletwall Outlet	10	m³
07 01 09	Baseplate	1100	m³
07 01 10	Concrete Padding	320	m³
07 01 11	Footings Baseplate	360	m³
07 02	Reinforcement 0,1 t/m³	222	t
07 03	Protective Stone Lining Bottom	760	m²
07 04	Protective Stone Lining Sides	250	m²
08	**Debris Rake - Steel**		
08 01	Rake-Beams (IPE 300)	19	t
08 02	Support Beams (HEB300)	9	t

05.05 Monitoring and Emergency Management Measures

In addition to structural mitigation measures, we emphasize that constant monitoring of relevant flow-system parameters at the apex of the fan and at the openings of retention structures are recommended. The following parameters need to be monitored and constantly sent to an online-system. Warnings can be sent to the members of an emergency staff and the fire brigade via SMS and MMS. Monitoring could include:

a) Flow height
b) Flow velocity
c) Gravel content
d) Height of the creeks bed
e) Height of the bed of the retention structures
f) IR-Video

06 REFERENCES

BGC Engineering Ltd. (2014a). Hydroclimatic analysis of the June 2013 storm. Final report prepared for the Town of Canmore. August 2014.

BGC Engineering Ltd. (2014b). Cougar Creek debris flood hazard assessment. Final report prepared for the Town of Canmore. March 2014.

ISL Engineering and Land Services (2013). Cougar Creek at Town of Canmore. Channel bank protections. Drawings issued for construction. October 2013

Thurber Engineering Ltd. (2014). Cougar Creek long-term debris flood mitigation. Initial geotechnical field investigation. Memorandum prepared for the Town of Canmore. August 2014

Kleeberg, H.-B., and H. Øverland (1989). Zur Berechnung des effektiven oder abflusswirksamen Niederschlags. Mitt. Inst. Wasserwesen 32, Univ. der Bundeswehr Munich, Germany.

Izzard, C.F. (1946). Hydraulics of Runoff from Developed Surfaces. Proc. Highway Research Board , vol. 26, p. 129-146. 1946.