Flood Mitigation Measures Cougar Creek

Town of CANMORE

1200 WATERFRONT CENTRE
200 BURRARD STREET
P.O. BOX 48600
VANCOUVER BC V7X 1T2
CANADA

Memorandum

Date:	Meetings on January 6, 2016 in Calgary, AB and January 7, 2016 in Canmore, AB		
	Coordination Conference Call on February 10, 2016		
	Subsequent conversations, conference calls and email correspondence		
Issue:	Options for Bottom Outlet Structure		
	Options for Cut-Off Wall, Structural Details of Seal Wall		
Participants	January 6, 2016	January 7, 2016	February 10, 2016
	Andy Esarte, ToC Felix Camire, ToC Julia Eisl, ToC	Andy Esarte, ToC Felix Camire, ToC Julia Eisl, ToC	Andy Esarte, ToC Felix Camire, ToC Julia Eisl, ToC
	Norbert Morgenstern, Rev. ToC John Sobkowicz, Rev. ToC Mauricio Pinheiro, Thurber Eng.	Emad Iskander, CHT Jürgen Laubbichler, CHT Manfred Scheikl, CHT	Emad Iskander, CHT Jürgen Laubbichler, CHT Manfred Scheikl, CHT
	Emad Iskander, CHT Jürgen Laubbichler, CHT Manfred Scheikl, CHT		
Distribution:	Andy Esarte, ToC Felix Camire, ToC Julia Eisl, ToC	Emad Iskander, CHT Jürgen Laubbichler, CHT Manfred Scheikl, CHT Anton Henle (CHT)	Anna Lettner (CHT) Jürgen Suda (CHT) David Powell (CHT) Florian Falkeis (CHT)

U:\6000_CHT\16494\07_DETAIL_ENG\90_REPORTS_MEMOS\ALTERNATIVES\LTMM CC - M - ALT-BOS-00 - 2016-02-24.docx

1 Introduction

At the above referenced meetings, the 60 percent design submitted in November 2015 (“base design”) was reviewed. In addition to the base design, new options for the bottom outlet structure and construction methods options for the seal wall, as well as the potential benefits, risks and cost impacts were discussed.

These discussions were centered around a) achieving the maximum durability of all construction elements, b) decreasing future maintenance efforts, and c) reducing the structural complexity.

1.1 Base Design and Options for the Bottom Outlet Structure (BOS)

The following options for the Bottom Outlet Structure were discussed:

- The base design, a segmental, box-shaped, reinforced cast-in-place bottom outlet structure aligned straight in the creek centerline, which feeds through the seal wall of the retention structure. It is placed on the alluvium, which is treated with ground improvement measures such as jet grouting to reduce differential settlements.

- Option 1, a segmental, box-shaped, reinforced cast-in-place bottom outlet structure placed on a rock-slope-cut at the orographic left (southeastern) abutment of the retention structure, aligned along the rock face with curves.
- Option 2, a mined diversion tunnel around the southeastern side of the retention structure.

The left abutment was selected for the optional alignment of the bottom outlet structure, because it appears to be more stable.

1.2 Base Design and Options for the Cut-Off in the Alluvium and the Seal Wall in the Embankment Dam.

The following options for the Cut-Off in the Alluvium and the Seal Wall in the Embankment Dam were discussed:

- The base design, a combination of a secant pile wall with reinforced secondary piles intersecting primary piles, socketed into bedrock with a minimum of 1m and a segmented reinforced, cast-in-place concrete wall with expansion joints and flexible water stops between wall segments.
- Option 1, a mixed in place wall using a double cutter hydro-phrase and cement slurries for seepage cut-off in the alluvium and for establishing the seal wall core of the embankment dam. The construction would be performed top-down from the dam crest. This option requires the BOS in a diversion tunnel.
- Option 2, an asphalt concrete core placed on a special formed and pretreated plinth which is structurally connected to the secant pile wall. At the abutments, a steep plinth is tied into a pre-grouted bedrock trench.

The Town of Canmore instructed CHT to evaluate these options in order to provide a decision-making basis for a confirmation or an adjustment of the current design.

2 Scope

This memorandum provides an evaluation of the options discussed, in order to ensure all issues have been thoroughly considered and the benefits as well as the drawbacks of each option are fully understood by all parties.

3 Options for the Bottom Outlet Structure (BOS)

3.1 Base Design - BOS at the Creek Centerline

3.1.1 Description

The base design provides a 100m long reinforced segmented concrete bottom outlet structure constructed directly on the alluvium. The alluvium is improved with ground treatment to reduce vertical displacements. The BOS is located in the middle of the creek as shown on CHT’s 60% design drawings.

The construction of the BOS itself is simple and quantities required for construction are the lowest compared to other options:

- Minimal trench excavation in the alluvium, leveling, preparation of jet grouting heads.
- Simple formwork and pouring for the complete outlet structure at a safe distance from the abutments and from rock-fall exposure.
- Reinforced concrete quantity for the culvert and pipe encasement is 2,880 m3 without intake.
- The overall inclination of the BOS is 4% and its length excluding intake is 100m.
- The proposed scheme is shown in Figure 1, Figure 2 and Figure 3. Additional details are provided by the 60% design package.

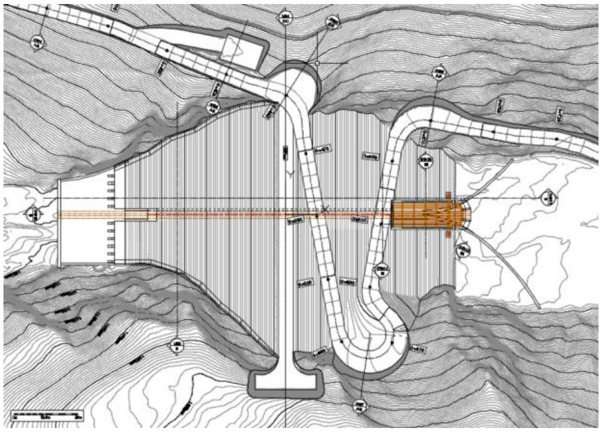

Figure 1: BOS at the creek centerline, in plan

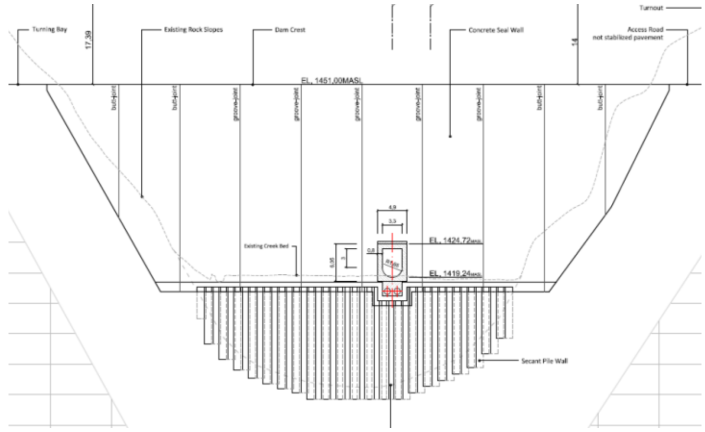

Figure 2: BOS at the creek centerline, in creek cross section

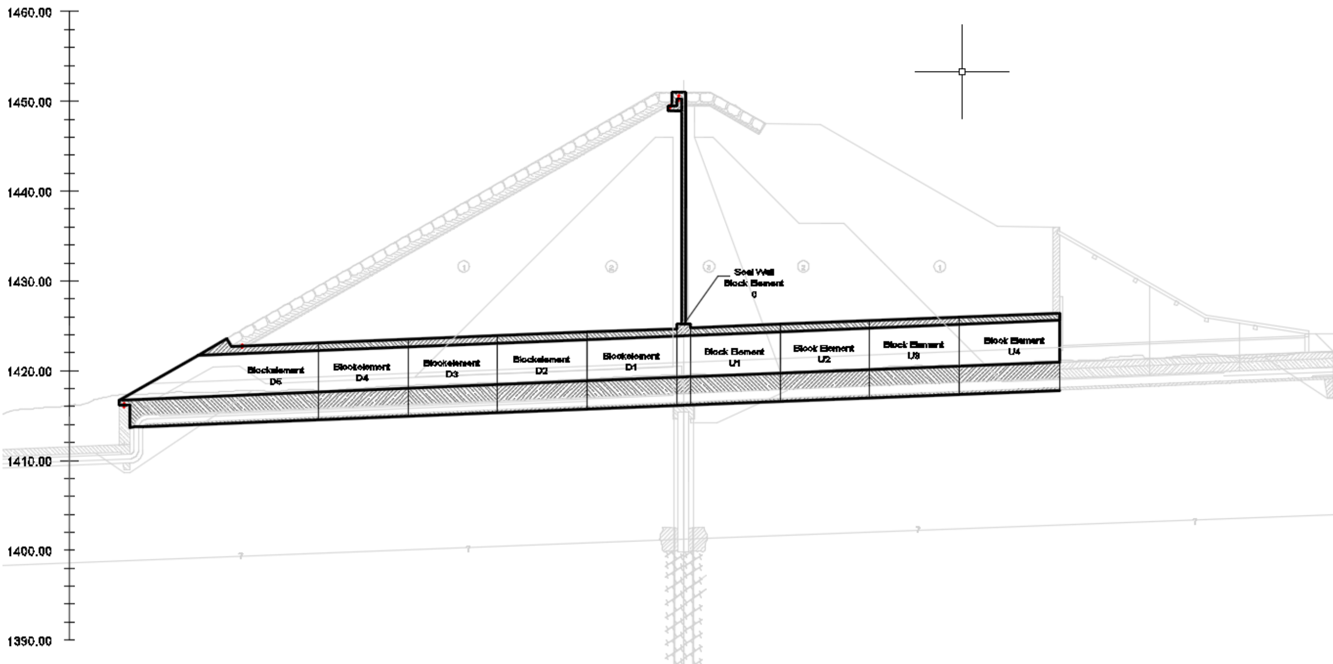

Figure 3: BOS at the Creek Centerline, in dam cross section

3.1.2 Detailed Design Requirements

The detailed design needs to address differential vertical displacements under varying embankment weight and variable loading conditions of (a) the BOS itself and (b) of the cut-off wall in the alluvium.

With respect to item a):

- The BOS will experience differential vertical loads under the variable embankment weight and varying loading conditions.
- To compensate vertical displacements along the BOS, in particular differential vertical displacements, ground improvement will be performed prior to construction. Applicable ground improvement techniques are low-pressure

soil permeation grouting, mixed-in-place columns arranged in a dense triangle pattern, jet grouting columns, rectangular jet grouting bodies or jet grouting walls arranged along the BOS.

- Underneath the joint between the seal wall and the adjacent BOS segments, reinforced bored piles shall be provided instead of jet-grouted columns to minimize differential displacement at this particular and most sensitive joint.
- The structural scheme provides expansion joints arranged at 10m c/c along the BOS to reduce bending moments and to allow for compensation of shrinkage.

With respect to item b)

- Since the secant pile wall for cut-off is founded on rock, it will experience minimal vertical deformations compared to the adjacent BOS segments. This also applies to the seal wall.
- The adjacent BOS segments would experience significant vertical displacement without ground improvement. In addition, the axis of the opening for the BOS in the seal wall rotates under lateral water pressure in the case of full storage a few millimeters, while the adjacent upstream culvert segment will follow the horizontal displacement of the embankment and tends to rotate less.
- To mitigate differential displacements, ground improvement below the BOS is proposed. Drawing LTMM-CC-DFG-401 shows a layout in plan of 4.0 m diameter jet grouting piles located along the culvert. Jet grouted columns are placed underneath each joint and at the middle of each BOS segment. Reinforced bored piles are provided next to the seal wall. Dependent on the Young´s modulus of the jet grouted columns, displacements according to Table 1 are resulting considering bored reinforced pile support of the BOS next to the seal wall. A range of reachable Young´s moduli for jet-grouted sand was demonstrated by Axtel & Stark at the DFI Conference in 2008. Covil & Skinner (1994) are reporting an elastic modulus of 6,000 MPa for the Soil Crete technology applied in sand. The highest potential strength and stiffness can be reached in gravel sand aggregates such as the existing alluvium at Cougar Creek, which is forming strength similar as concrete, dependent on the type of cement used.

Table 1: Vertical displacements at the BOS and the seal wall versus quality of ground improvement at full impoundment

Assessment of displacements at the BOS		**Young´s Modulus of Jet-Grouted Columns**		
		1GPa	**2 GPa**	**4 GPa**
Maximum vertical displacement calculated	Seal wall	3.88	3.80	3.75
	BOS	11.3	8.18	7.29
Differential Displacement at interface BOS/SW		4.79-3.55 = 1.24mm	4.08-3.51 = 0.57mm	4.12-3.39 = 0.73

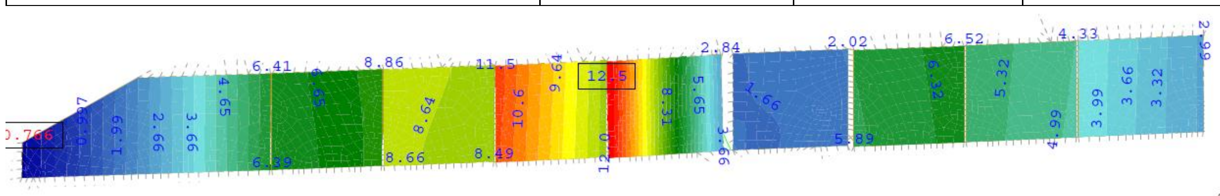

Figure 4: Vertical displacements [mm] of the BOS with jet-grouted columns with an E-Modulus of 1GPa, arranged as in the base design.

- Considering an average depth of the alluvium of 16m, the corresponding jet grouting quantity will be 3,600 m3.
- Considering an average depth of the alluvium of 16m, the additional amount of bored reinforced concrete piles, 1.2m in diameter, is 72m3.
- Sparing four columns in total, one underneath each of the first two upstream BOS segments and one at the first two downstream BOS segments results in maximum differential displacements between BOS segments of approximately 4mm, which is within acceptable limits. Four additional piles support the joint between the seal wall and adjacent BOS segments. The overall volume of ground improvement with jet grouting is than approximately 2,800m3.

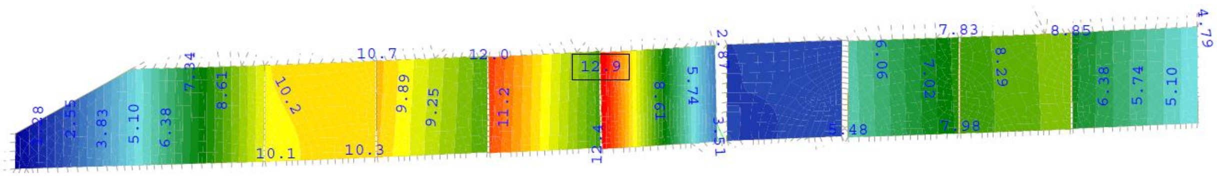

Figure 5: Vertical displacements [mm] of the BOS with jet-grouted columns with E=1GPa, sparing 4 columns at the intake and outlet.

- The remaining differential displacements at joints will be addressed as following:
  - A joint cover at the extrados of the BOS avoiding the inflow of fill material into the BOS under loading conditions and allowing for differential displacement with elements such as:
    - a) Outside stainless steel plate;
    - b) Layers of bitumen based jointing compounds placed in a v-shaped joint,
  - Embedded water-stops in the center of joints;
  - Omega seals at the intrados of the culvert, protected by removable stainless steel plates for maintenance purposes.

The omega seals are accessible, maintainable and removable. By removing the omega seal, the centered water stop is accessible for updates as well. The joint cover at the extrados is flexible and of non-aging components. In addition, a plan for joint maintenance will be established to ensure they remain functional over the lifespan of the project.

A preliminary parametric FE analysis as attached in **Appendix B** showing the investigated arrangement of ground improvement with jet grouted columns as well as resulting displacements of a segmented BOS.

3.2 Option 1 – Bottom Outlet at the left Abutment of the Structure

3.2.1 Description

This option relocates the culvert to the left (southeastern) abutment so that it is built on bedrock, with the following features:

- Rock slope cutting estimated at 9,800m3 and slope protection as well as trench excavation in rock estimated at 2,600m3 is required for the proper arrangement of the groundwater control pipe which is located underneath the BOS.
- Concrete construction and joint details similar to the base design but for lower differential displacements in horizontal and vertical direction.
- No ground improvement required.
- The complete construction site is exposed to rock-fall, as it is located directly at the creek slope and temporary rock-fall protection is required.
- The intake is permanently exposed to rock-fall and permanent rock-fall protection and maintenance of rock-fall protection is required.

Sketches of this option for the construction stage rock cut and excavation are provided in Figure 6 and Figure 7. Figure 8 and Figure 9 provide sketches in plan and in creek cross section; conceptual drawings are provided in **Appendix C** of this memorandum.

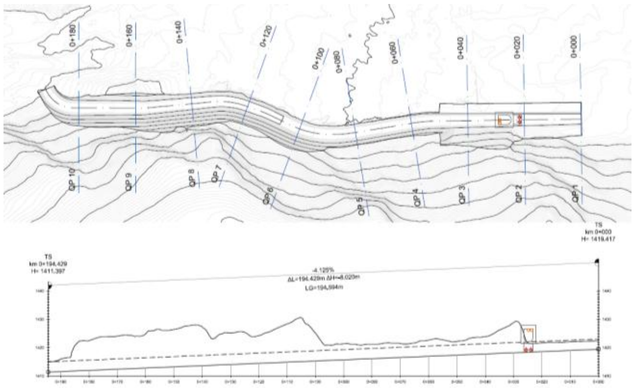

Figure 6: Rock Cut for the BOS at the left Abutment, in plan

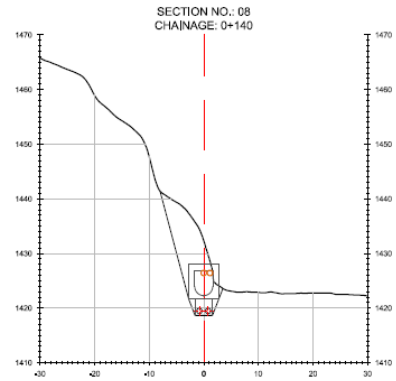

Figure 7: BOS at the left abutment, cross section

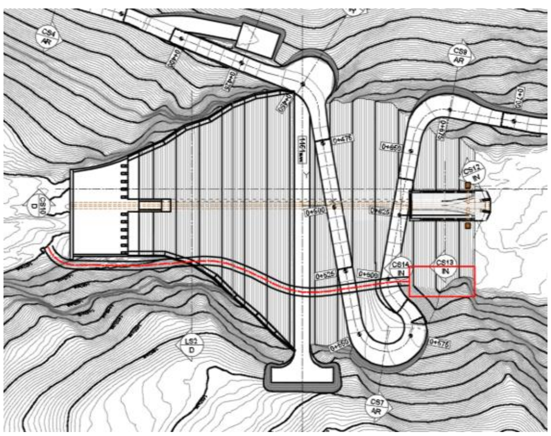

Figure 8: Bottom Outlet at the left abutment, in plan

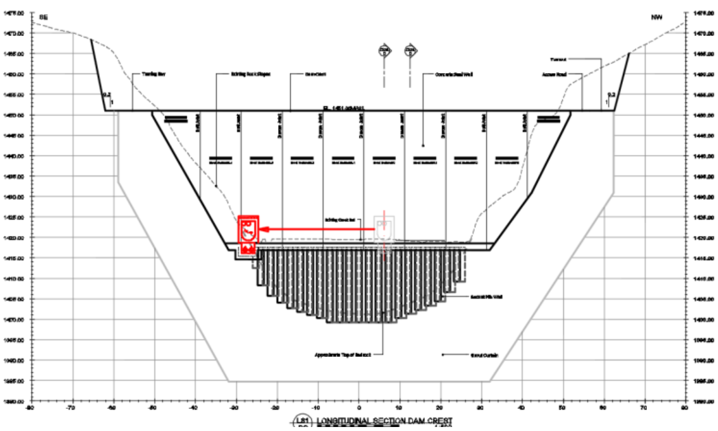

Figure 9: BOS at the left Abutment , creek cross section

3.2.2 Detailed Design Requirements

If selected, the design needs to provide details for following issues aside additional investigations as follows:

- Detailed assessment of geological structures at the left abutment for the design of the rock slope cut and the design and dimensioning of slope support element such as rock bolts and wire meshes, anchors and concrete support beams as required.
- Stability assessment of the slope cut and eventually scheduling of a two or three stage approach for slope cut and subsequent establishment of slope stabilization measures.
- Development of a detailed alignment and a detailed geometry and stabilization measures for the rock cut at the left abutment;
- The preliminary rock cut volume is 9,800m3. Trench excavation is 2,600m3. Scaling as well as slope stabilization measures need to be established for approximately 2,350m2 including trench slopes.
- The preliminary length of the BOS excluding intake is 129m.
- Regarding hydraulic functionality and discharge of bedload during small events considering an overall inclination of approximately 3.1% (dependent on the detailed alignment), no substantial disadvantage compared to the base design with 4% inclination is expected.

- Because of latent rock-fall hazards, which lead to complications during geotechnical investigation in August 2015, temporary rock-fall protection needs to be designed along the complete length of the outlet structure. Permanent rock-fall protection shall avoid potential damages and danger for maintenance personnel at access points at the intake structure and the outlet. Temporary rock-fall protection needs to be provided at a length of approximately 200m and permanent protection at approximately 40m.
- Monitoring of the stability of the rock slope cut and on site determination of rock support measures during construction.

3.2.3 Comparative Analysis

The evaluation matrix in **Appendix A1** provides an overview of the advantages, disadvantages of this option.

Appendix A2 provides comparative cost estimates.

3.3 Option 2 – Diversion Tunnel

3.3.1 Description

This option includes a diversion tunnel which is completely detached from the dam structure. While it was initially considered by the Design Team as well as other tunnel options for access, it was not pursued due to the potentially higher cost for design, procurement and construction. It is now evaluated more closely in response to the Expert Reviewer's comments and suggestions.

The main features are:

- A 150m long diversion tunnel of the same discharge area as the culvert, curved in plan with an overall inclination of 2.6% (dependent on the detailed alignment);
- Excavation by drill & blast and initial support by rock bolts and shotcrete.
- Permanent lining by cast in-place concrete or fiber-reinforced shotcrete.

The tunnel option would de-couple the outlet from the cut-off wall, both in terms of structural behavior as well as in construction schedule. Concerns regarding joint detailing are alleviated.

The general arrangement and the tunnel cross section are shown in Figure 10, Figure 11 and Figure 12, conceptual drawings of this Option are provided in **Appendix C** of this memorandum.

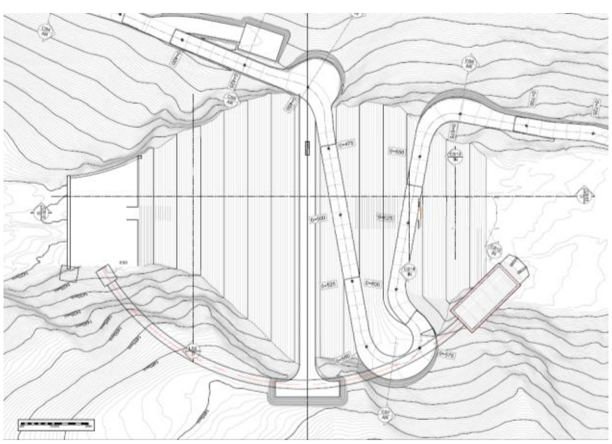

Figure 10: BOS in a diversion tunnel and intake, in plan

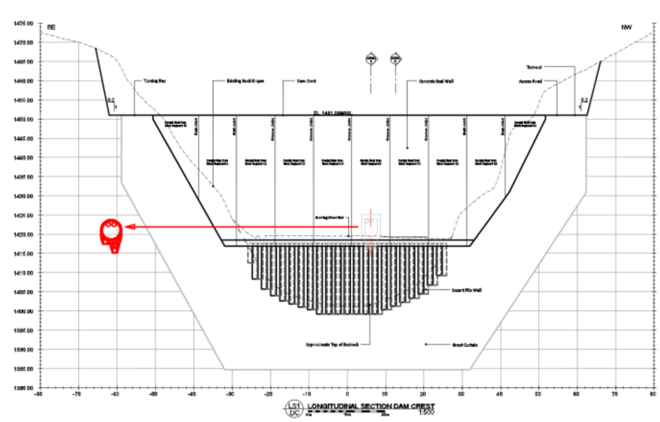

Figure 11: BOS in a diversion tunnel, in creek cross section

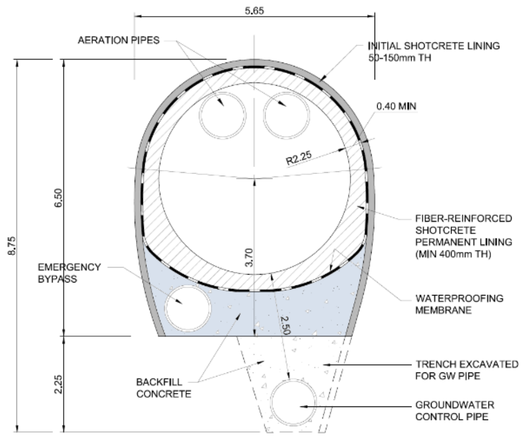

Figure 12: Proposed tunnel cross section

3.3.2 Detailed Design Requirements

If selected, the design will need to address the following issues:

- Re-location of the intake to the left abutment and backfilling to obtain an integration into the embankment dam;
- Integration of the outlet at the left side of the stilling basin;
- Re-configuration of the alignment of the bottom outlet to a curved structure;
- Excavation sequences and temporary support measures, both at the portals as well as within the tunnels;
- Development of a permanent portal structure, including rock-fall protection to avoid damages at the intake structure; permanent rock-fall protection will be approximately 40m of length for access points at the intake and the outlet.
- Detailed requirements for the final structure, including waterproofing and concrete lining requirements. However, to avoid recurring detachments due to action of frost-thaw cycling and water inflow and subsequent damages as well as for comparability with the base design in terms of durability, a final lining according to the concept drawing (see Figure 12) appears to be advisable.
- The channel itself will be equipped with a steel lining for protection against abrasion, identical to the base design.
- Procurement and allocation of the geotechnical risk for the tunnel construction.
- Regarding hydraulic functionality and discharge of bedload during small events considering an overall inclination of approximately 2.6% (dependent on the detailed alignment), a decrease of flow velocity by approximately 10 to 20% needs to be taken into account. However, a channel with an inclination of 2.6% with steel lining has a high capability of clear water and bedload discharge. Most likely, the spacing of beams at the gravel rake will limit component sizes potentially discharged through the BOS rather than its hydraulic capacity. Due to buildability and construction constraints, the invert of the tunnel has a radius of 2.25m, which is bigger compared to channel in the base design with a radius of 1.65m. The quantity of steel armoring will be similar.

3.4 Comparative Analysis of BOS Options

The evaluation matrix in **Appendix A1** provides a comparative analysis of options for the bottom outlet structure. **Appendix A2** provides comparative cost estimates.

3.5 Comparative Cost Estimates for BOS Options

According to cost estimates provided in **Appendix A2** construction costs are distributed as shown in Figure 13.

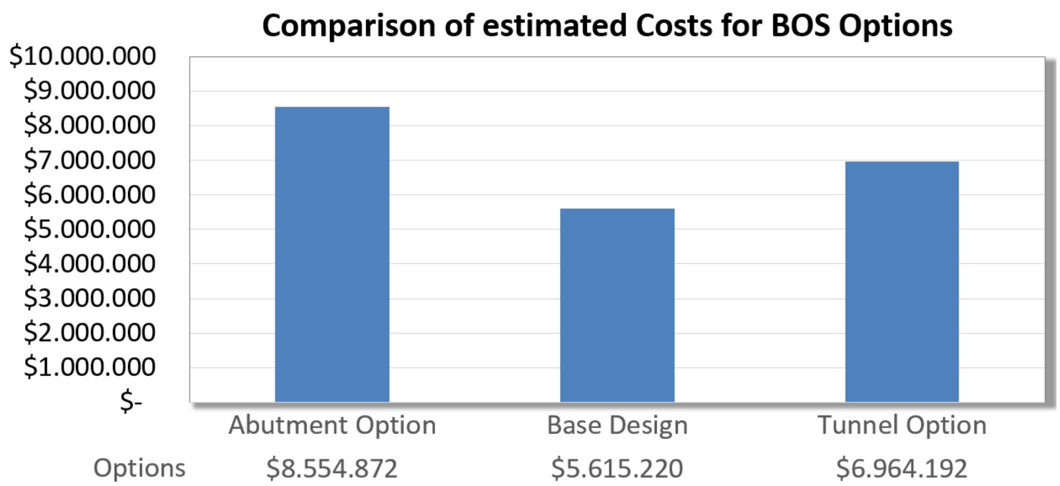

Figure 13: Comparison of construction costs for equally equipped and equal durable options for the BOS

4 Options for Cut-Off and Seal Wall

4.1 Base Design – Secant Pile Wall and Segmented reinforced Concrete Wall with Expansion Joints

4.1.1 Description

The base design provides a 52m long, straight aligned secant pile wall were secondary piles are reinforced and cut into the primary piles which are not reinforced. Piles are socketed into bedrock with a minimum depth of 1m. The selected pile diameter is 1.2m for compensation of axial deviations during drilling (structurally required diameter is 1m). A spacing of 0.9m and an intersecting length of 0.3m results in a nominal wall thickness of 0.8m and a minimum wall thickness of 0.5m considering deviations during drilling. Connection reinforcement for structural connection to a reinforced cast in place seal wall is integrated in the reinforcement cage of the secondary piles. Sleeve pipes are provided for structural grouting at the interface between the seal wall and the secant pile wall. In addition, the interface between pile wall and seal wall is covered with impervious material placed upstream of the core structure.

The seal wall is a segmented cast-in-place concrete wall, with a thickness of 0.5m. The wall segments are 10m long and expansion joints are arranged between wall segments. At the abutments the seal wall is tied into the pre-grouted bedrock by means of an abutment trench. Structural grouting will provide the appropriate sealing of the contact between the bedrock and the seal wall. An impervious layer is placed upstream of the seal wall for additional sealing. Downstream of the seal wall a vertical drainage layer is placed which is connected to a horizontal drainage layer underneath the downstream embankment.

The structural characteristics of a reinforced concrete wall structurally connected to a secant pile wall provides following advantages:

- Structural integrity of the complete cut-off and sealing system from bottom up to crest.
- Structural integrity with the spillway construction.
- Control of crack width limitation by means of reinforcement.
- Highest resistance against erosion compared to all other alternatives.
- Beneficial and controllable behaviour of the seal wall under sudden loading which is typical for flood retention structures. Obligatory observation of long-term behaviour of alternative less robust sealing measures under impoundment conditions is not possible. Intervention in case of leakages, detected by subsequent impoundment, typically for hydropower dams, is not feasible for flood retention structures.

4.1.2 Detailed Design Requirements and Limitations

The structural complexity of the detail were the BOS is feeding through the seal wall requires the following:

- Development of an appropriate joint detail allowing for remaining vertical (1 to 4mm) as well as for horizontal displacements.
- Detailed structural analyses for reliable calculation results for displacements, acting forces and moments.
- Development of a detailed water proving system comprising the following:
    - A joint cover at the extrados of the BOS avoiding the inflow of fill material into the BOS under loading conditions and allowing for differential displacement with elements such as:
        - Outside stainless steel plate
        - Layers of bitumen based jointing compound
    - Embedded water-stops in the center of joints

- Omega seals at the intrados of the culvert, protected by removable stainless steel plates for maintenance purposes.

Structural complexity and maintenance issues regarding joints between seal wall segments require the following:

- Development of joint details similar as required for the BOS.
- Investigation of the functionality of the additional impervious layer placed upstream of the seal wall under loading conditions and differential horizontal displacements between wall segments.

As a **sub option to the base design** a **fully reinforced seal wall** construction **without expansion joints using shrinkage compensating concrete** (K-Type Cement or B-Component) is recommended for further investigation.

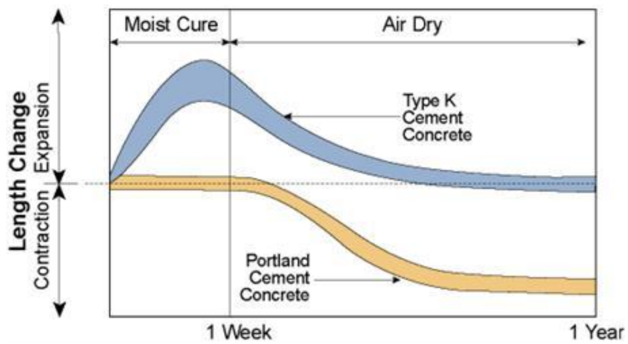

Figure 14: Concrete behavior over time using OPC and K-Type cements

- This sub-option appears to a feasible alternative according to a basic investigation on concrete quality and construction staging.
- Using additives and/or appropriate basic concrete mixes, no substantial shrinkage is to be expected at the same concrete qualities (see Figure 14). Hindering expansion is resulting in higher concrete strengths.
- ACI report 223R gives guidelines on shrinkage-compensating concrete as follows:
  - Requirement of 10% higher W/C ratio for proper hydration and workability.
  - Concrete mix has to provide 4.5"-6.0" slump at the point of discharge for good workability.
  - Very dense, with higher resistance to abrasion and chemical attack.
  - Fresh concrete is cohesive but not sticky resulting in less or almost no bleeding or segregation, suitable for pumping.
  - Pouring at temperatures lower than 20°C is preferable. Otherwise, a higher water content is required to avoid loss of workability.
  - Faster setting than OPC based concrete.
- Appropriate construction staging needs to be developed and applied to minimize required structural grouting at the abutments.
- Curing is recommended to optimize shrinkage compensation.
- According to the outcome of a preliminary investigation, additional costs in a range of approximately 50 to 100 CAD per cubic meter and 75,000 to 150,000 CAD for the complete seal wall need to be considered.

- However, sparing flexible water stops and sparing vulcanizing of water stop crossings at working joints are expected to more than compensate for these costs. For a similar project in Austria additional costs of EURO 80,000 for water stops (approximately the same amount as required for Cougar Creek 311m/280m) and vulcanizing of water stop crossings had to be paid. Transferring these numbers to Canada approximately 160,000 to 200,000 CAD need to be reserved for high performance flexible water stops and preparation of crossings at working joints. In addition working joints would need to be made of the same material as water stops for expansion joints for being connectable/weldable.

4.2 Option 1 – Diaphragm Wall

A diaphragm wall is an almost impervious comparable thin structural element reducing percolation. Diaphragm walls in the original sense are constructed by means of deep trench excavation formed for the placement of cast-in-place reinforced concrete wall segments (panels) or for the placement of pre-cast wall elements. The traditional way of diaphragm wall construction is trench excavation supported by bentonite slurries and subsequent placement of reinforcement cages and tremie concreting. Bentonite slurry, replaced by concrete can be reused. Diaphragm walls are constructed to a depth of 45 to 50m. For trench excavation a grabber or a hydrophraise is used. The technology of the hydrophraise was combined with in situ soil mixing to the CSM wall technology, where cutting and soil mixing is done in one continuous operation.

4.2.1 Traditional Diaphragm Wall Construction with Grabber or Cutter

For constructing the trench, a grabber (clamshell bucket) or a hydrofraise (reverse circulation trench cutter) is used. Bentonite slurry is pumped into the trench in order to balance the soil pressure and prevent the trench from collapsing. The wall panels are constructed in alternating fashion as primary and secondary elements.

Because of the prevalent ground water flow conditions at the Cougar Creek alluvium and the abundance of cobbles and boulders, the application of support slurries appears not to be feasible. The following major risks appear too high for further consideration of this method:

- Dislocated cobbles falling into the trench and hindering the placement of the reinforcement cage.
- Dislocated cobbles falling into the trench and penetrating the wall if no reinforcement is used.
- Substantial washout of support slurry and collapse of the complete panel trench.

In addition, the use of bentonite slurries for the construction of a diaphragm wall top down from the crest is connected to a substantial risk of spoiling drainage layers and drainage pipes in the ready built structure. Because of this and issues listed above, this sub-option was not further investigated.

4.2.2 CSM Wall

4.2.2.1 Description

The CSM-method uses modified, high-performance cutters for cutting of intersected trench segments and soil mixing down to great depths. It is best applicable in very densely packed uniform soils. Predrilling and exchange of soil with aggregates allows the application of the CSM-method in less uniform and loose soils. Keying a CSM wall into rock is possible by using special rock cutters. However, rearrangement needs to be done for each single wall segment. It is a suitable technique in particular for securing deep excavation pits and for updating cut-off and sealing of dams and dykes.

For the Cougar Creek project, the CSM method is discussed as an alternative for sealing the dam structure as well as for constructing the cut-off wall in the alluvium in one lift, cut from top down of the structure. This option came into discussion in conjunction with alternatives discussed for the bottom outlet structure shifted into a diversion tunnel.

The construction of a CSM wall requires specific site facilities, comprising the following:

- Drill rig for predrilling and soil exchange;
- Trench cutter for depths of up to approximately 46m;
- Slurry management including silo for cement, mixer, slurry container and pumps;
- Working platform with a minimum with of approximately 10 to 12m.

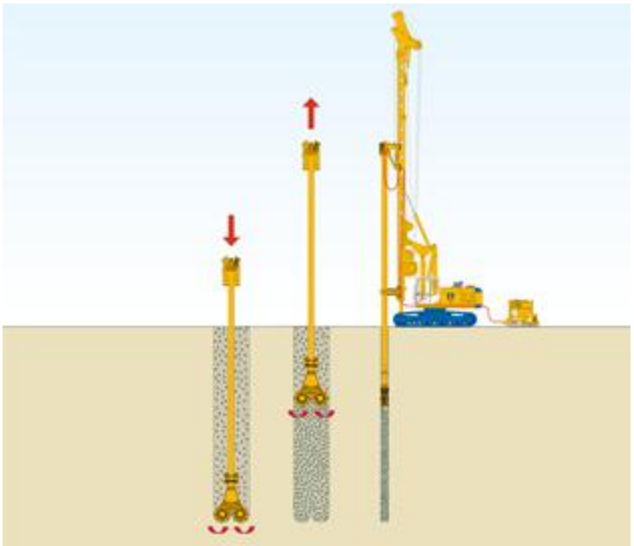

Figure 15: General construction staging scheme for a CSM wall (Bauer Foundations)

Figure 16: CSM Equipment for deep applications (www.bauforum24.biz)

4.2.2.2 Detailed Design Requirements and Limitations for Construction

During discussions with specialized contractors on the applicability of the CSM technology the following issues were emphasized:

- One major benefit a CSM wall provides is that it can be constructed in one piece from top down. This would provide higher flexibility in terms of construction staging. Dam filling could be done without interruption for concreting and short term, if ideal weather conditions occur during cold seasons. In general, temperatures below zero do not allow fill and compaction works.
- The fact that a CSM wall has no joints and spares the development and construction of joint details as well as maintenance long term, is an additional benefit.
- Viewing the specific project setting for the rather small Cougar Creek dam structure it appears that given limited availability of hydrofraises and size of machines as well as the required installation of slurry management result in unrealistically high costs for mobilization.
- Because bedrock surface, investigated with geophysical methods as well as with test drillings, appears to be glacial imprinted and showing a typical u-shape, trench cutting for keying in the CSM wall does not appear to be feasible without predrilling. Predrilling for establishing horizontal segments in the bedrock for later cutting and soil mixing is theoretically possible but problematic. Detachment of rock wedges between levels cannot be controlled or avoided, not detected accordingly as well.
- Potential contractors interviewed during the current stage of procurement noted that the given composition of the alluvium, recorded within past investigation campaigns, requires soil exchange before applying CSM. The risk of

instabilities at the trench walls and boulders or cobbles disturbing the mixing process requires predrilling and soil exchange ahead of applying CSM.

- As consequence of the issues listed above, predrilling and replacement of soil with aggregates needs to be done. Predrilling and soil exchange needs to be done with casing very similar as for a secant pile wall but using aggregates instead of concrete. Aggregates are mixed in place at a later stage of the CSM construction process. Viewing the required pre-treatment of the alluvium, it appears impractical to put this construction step ahead of CSM, instead of constructing a secant pile wall and having the same effort. Construction the secant pile wall provides better quality as well as better control as well as more flexibility for intervention.
- Constructing a CSM wall from top down requires large construction machines carrying the cutter head.
  - For reaching suitable stability during cutting the cuter head needs to be arranged in front of the platform. A minimum width of 10 to 12m was estimated for the setup. Because the dam crest is designed with a width of 6m allowing for access with rock trucks and roller compactors, the design would need to be changed and, amongst others, would result in an additional fill volume of approximately 15,000m3.
  - Alternatively, constructing the upper part of the seal wall differently would result in a structural change, in additional joints to be managed and maintained and in corresponding discussions on proved solutions for an overtoppable structure.
  - Doing a CSM wall in two steps with smaller machines would result in predrilling and soil exchange prior to CSM with the same effort as for the secant pile wall but with less confidence. Cutting into the pre-constructed cut-off from the crest at a sensitive level exposed to high water pressure and acting forces at loading conditions is required for this sub-option. At this level the structure is containing drainage layers and drainage pipes sensitive for spoiling with cement slurries.
- Mobilizing and transport of equipment to the dam crest at the end of dam filling would require adaption of the design of access roads to a larger width. This would consequently lead to higher slope cuts.
- Some of the interviewed potential contractors share our concern of spoiling filters, drainage layers as well as drainage pipes with cement slurry potentially extruded from the designed vertical layer containing the aggregates for later mixing. Intervention and appropriate control during construction is not feasible and not proved for similar structures. The risk of damaging the structure and/or substantial facilities with high relevance in terms of dam safety right at the end of construction is strongly relativizing potential benefits of this option.
- One of the locally represented and highly specialized firms, producing their own hydrophrases for CSM and diaphragm wall applications, would primarily not recommend doing CSM because of given constraints and high construction risks.
- Aside issues listed above, a CSM wall provides a mixed in place produced concrete wall without reinforcement. However, the design requirement is to provide cut-off measures and, in particular, a seal core with limited crack development and crack width limitation under loading conditions. A concrete wall in general requires limited segment widths and expansion joints between segments allowing for controlled shrinkage and allowing for differential deformation. To meet durability criteria these joints need to be engineered to fulfill requirements. However, the fact that a CSM wall does not have joints on purpose does not mean that it is not a concrete wall underlying shrinkage and loading, both resulting in the development of cracks but without any control. This circumstance is relativizing the benefit of sparing joints and reducing structural complexity because of comparing a reinforced structure with a not reinforced one. Because hiding of structural issues in the discussion as explained before should be avoided, the CSM wall would need to be compared with a joint-less cast in place seal wall.

4.3 Option 2 – Asphalt Concrete Core (ACC) placed on a Secant Pile Wall

4.3.1 Asphalt Concrete Core Dams (ACCD´s) and References

Asphalt concrete cores for hydropower embankment dams (ACCD) were built many times mainly in Austria, Germany, China and Norway over the last 60 years. Extensive investigations demonstrated a tolerance of asphalt concrete to ground movement and self-healing of cracks was observed. Because core construction is possible under wet and cold conditions it increases construction schedule flexibility. Due to ductile behavior of asphalt concrete the core can accommodate differential settlements between the core and adjacent fill material. Case studies are reported by Alicescu et. al. (2008); Arnevik et al. (1988). Breth, H. (1964), Chen et al. (2008); Hoeg, K. (1993); among many others. All references are referring to permanently impounded and not overtopable structures. No references for overtopable flood retention dams are available.

Almost all of the ACCD are built on foundation plinths directly founded on bedrock. Some structures were built on compressible alluvium with considerable high differential vertical displacement between the ACC and the adjacent embankment structures. Most of the dams built with an ACC are equipped with drainage and control galleries for additional intervention and drainage. Leakages through the central ACC membrane were reported from some structures.

- Eberlaste dam in Tirol, Austria (Rienössl et. al. 1972, 1973) experienced several meters of subgrade settlement during construction (2.3m). The ACC is placed on a ductile slurry cut-off wall. At Eberlaste dam leakage was detected but classified as not notable.
- At Majiagou ACC Dam (Liu Jiangang et. al. 2005) severe leakage through the asphalt concrete core was detected endangering the overall dam structure.
- At Yele dam (Wang et. Al. 2010) a total seepage of approximately 350l/s was detected, additional grouting was carried out, and new drainage wells were drilled from a drainage gallery. After intervention the total seepage was approximately 280/s.
- Because asphalt concrete cores cannot compensate substantial tension, the ability of asphalt concrete cores was questioned and discussed intensively. Extensive investigation on transverse cracks were carried out and summarized by the Norwegian Geotechnical Institute (NGI) in 1995 on transverse cracking in embankment dams. Results are highlighting that valley shapes comparable to Cougar Creek as well as a combination of rigid slopes and compressible alluvium are sensitive for substantial development of tension cracks at several sections of the seal wall.
- Because cracks have been observed in ACC´s and leakages with decreasing seepage over time, studies and experiments on the self-healing behaviour have been carried out by the NGI (Hóeg 2009, 2012). Results are indicating that cracks developed under loading conditions are sealing by themselves within 10 to 80 hours to a width resulting in acceptable seepage. This observation makes ACC a competitive alternative for hydropower embankment dams. However, a flood retention structure normally is not impounded and seepage phenomena cannot be observed and managed subsequently. Self-healing within some hours is not acceptable. Most relevant loading conditions at flood retention dams are developing very quickly, not allowing for coordinated intervention such as re-grouting or establishing of drainages for leakages in the core.

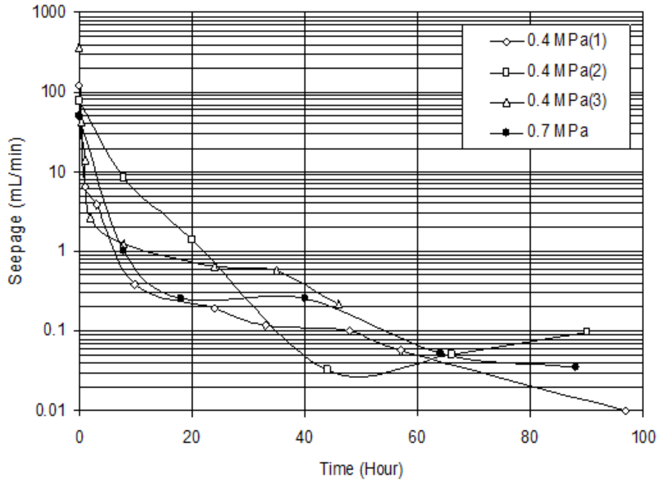

Figure 17: Seepage over time through an asphalt concrete specimen under varying normal stresses (Presentation Hóeg 2009)

4.3.2 Detailed Design Requirements and Limitations

If selected, the design needs to address the following issues:

- Structural investigation of the ACC-Dam for transverse cracking and the selection of an adequate core thickness for expected settlement and horizontal deformation.
- Investigation of an adequate AC-mix;
- Investigation of an adequate plinth construction without joints for both, at the head of the secant pile wall and the bed-rock abutments;
- Development of construction specifications for the following:
  - Initial layers and lateral sections adjacent to the abutments for construction by hand to provide the required space (15-30m)
  - Construction process using a paver which incorporates storage devices for bituminous concrete and transition materials (which are alternately applied), compaction and levelling, as well as for warming and drying devices,
  - Adequate compaction criteria for the core and transition zones.
  - Quality control and monitoring of air inclusions and cracks during construction.

Investigations and literature research on the application of asphaltic concrete cores for flood retention structures with an overtopable crest were not successful. No references could be found for this specific application. Local constraints and design requirements are not appearing suitable for an ACC. Therefore, no evidence of selecting an ACC as a proved and practiced design concept for an overtopable flood retention structure can be presented by now.

- Aside arguments listed above an asphalt concrete core is providing a much lower resistance against dam erosion;
- Selecting an ACC, structural complexity, maintenance issues and associated discussions on details would be transferred to the interface between the plinth and the ACC and to the interface between the ACC and the crest construction, both extremely sensitive to seepage or erosion.

Figure 18: Paver for ACC (Hóeg 2009)

Figure 19: Cracks due to compaction of adjacent fill material (Hóeg 2009)

5 Conclusion

With respect to the cut-off wall, having evaluated all technical aspects and compared the costs, we recommend to select the base design including an adjustment of the structural layout such as eliminating the expansion joints and using shrinkage compensating concrete.

With respect to the bottom outlet structure, the evaluation of alternatives shows that feasible alternatives, both the tunnel option and the bottom outlet structure at the abutment, would increase the project cost.

Therefore, we recommend the base design, which needs to be refined by developing and optimizing joint details as well as ground improvement as shown in the corresponding sections of this memorandum and supported by calculations in Appendix B.

However, we believe there may be a benefit for the Town of Canmore to take advantage of the current slowdown in mining and thus receive competitive pricing for the tunnel option as well. Therefore, we would recommend including the tunnel option in the bid documents, and letting Contractors bid on both options.

Prepared:	E. Iskander, J. Laubbichler, M. Scheikl, January 25
1st Review:	J. Laubbichler, M. Scheikl, February 26, 2016
Revised and Extended:	M. Scheikl, February 24, 2016
2nd Review:	J. Laubbichler, February 29, 2016
	ToC, February March 1, 2016
Final Revision:	M. Scheikl, March 14, 2016
Final Review:	J. Laubbichler, March 15, 2016