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Abbreviations

AAR	alkali-aggregate reaction
AEP	Alberta Environment and Parks
AER	Alberta Energy Regulator
AWWID	Alberta Water Well Information Database
CDA	Canadian Dam Association
CEAR	Canadian Environmental Assessment Registry
CFEM	Canadian Foundation Engineering Manual
CH	high plastic glacio-lacustrine clay
CPT	cone penetration test
CWMS	Civil Works Master Specification
D50	median particle size
ECO Plan	Environmental Construction Operations Plan
EDGM	earthquake design ground motion
El	elevation
FEM	finite element method
FOS	factors of safety
FSL	full service level
GIS	geographic information system
GL	glacial lacustrine
GT	glacial till
IDF	inflow design flood
LAA	local assessment area
LLOW	low-level outlet works
MDE	maximum design earthquake
NRCB	Natural Resources Conservation Board
OHI	Obermeyer Hydro, Inc.
PDA	Project development area

PLC	programable logic controller
PMF	probable maximum flood
PMP	probable maximum precipitation
Project	Springbank Off-stream Reservoir
PSHA	probabilistic seismic hazard assessment
RAA	regional assessment area
RCC	roller compacted concrete
SDI	Slake durability index
SPT	standard penetration test
TDR	technical data report
TOR	terms of reference
USACE	United States Army Corps of Engineers
USBR	United States Department of Interior, Bureau of Reclamation
VC	valued component

8 DAM SAFETY

Question 449

Volume 1, Section 1.3, Figures 1-6
Volume 1, Section 1.3.2.1, Figure 1-7
Volume 1, Section 1.3.2.2, Figure 1-8

The Project Development Area (PDA) is shown as a contour level in the reservoir area but is not described as such.

a. What is the contour level shown for the PDA in the reservoir? Does it correspond with the PMF? If it does not correspond with the PMF explain the discrepancy.

Response 449

a. The contour level shown for the PDA in the reservoir is the top of dam (elevation [El.] 1,213.5 m), as indicated in Volume 1, Section 1.3. This contour level does not correspond with the probable maximum flood (PMF; El. 1,212.0 m). Effects during flood operations, including wind setup and runup at the peak of the PMF impoundment levels, may extend up to but not exceed the top of dam; therefore, there is no discrepancy.

Question 450

Volume 1, Section 3.1, Page 3.1

Alberta Transportation states that *The Project will provide 77,771,000 m³ of active flood storage.*

a. For the active flood storage of 77,771,000 m³, clarify the storage retained within the reservoir and if a portion is contained within the downstream section of diversion channel.

Response 450

a. The calculated volume for temporarily retained water in the reservoir (77,771,000 m³) does not include the additional volume within the excavated diversion channel. Spillway elevations and the dam crest are predicated on the volume retention in the reservoir for a design flood and the flood routing calculations. As a conservative measure, the volume of the channel was not included for these considerations because the channel design was in process and subject to change throughout the design advancement. Inclusion of the channel volume will not change the recommended design.

Question 451

Volume 1, Section 3.1, Pages 3.1 to 3.2
Preliminary Design Report (draft) Appendix B.4-1PMF Analysis Report, Section 3.5, Page 27

Alberta Transportation states that some of the project components are designed for the PMF, which was estimated using a PMP analysis and hydrologic modelling.

In the draft PMF report included as Appendix B.4-1, Section 3.5, Page 27 of the draft Preliminary Design Report, the initial calibration of the hydrologic model to the 2013 and one of the 2005 floods was inconsistent with the results of the flood frequency analysis when simulating a 1:100 year rainfall. The model was therefore adjusted to replicate the 1:100 year peak from the flood frequency results. Alberta Transportation also states in the Preliminary Design Report that *floods on the Elbow River are from a mixed population of snowmelt, spring rain on snow, and summer rainfall only floods*.

a. Was any snowmelt contribution considered when adjusting the model to match the 1:100 year peak discharge from the frequency analysis? If so, what was that contribution? If not, what is the rationale for adjusting the model to match the results of the frequency analysis of a mixed population when simulating a purely rainfall event?

b. Provide verification of the adjusted model by showing the observed and simulated hydrographs of the 2005 and 2013 events.

c. Why were the other two flood peaks that occurred in 2005 not used for calibration or verification?

Response 451

a. Snowmelt contribution was not included in the adjustments of the model to match the 1:100 year peak discharge from the frequency analysis. The rainfall-runoff model was developed to characterize rainfall-dominant flood events similar to previous large historic floods on Elbow River, such as June 2005 and June 2013. The rainfall-runoff model was calibrated to the 2005 and 2013 events without consideration of snowmelt. In both cases, the volume of snowmelt is estimated at less than 10% of total flood volume. These estimates were made based on mapping of the spatial extent of snow cover prior to the 2005 flood (NOAA 2015) and remote sensing data of snow pillow extent and depth prior to and following the June 2013 flood. Therefore, snow melt was determined to not have a major effect on the peak flow and total flood volume associated with the events and not required for inclusion in the calibration.

The frequency analysis considered the effects of a mixed rainfall and snow melt on the peak flow estimate. Based on a review of various curve fitting approaches, two separate functions were selected to estimate peak flow and volume probability. One function was for flows more frequent than 1:10 years that best fits historic mixed rainfall and snowmelt events and the other function was for flows less frequent than 1:10 years that best represents historic rainfall dominated floods.

Therefore, considering the historic storm calibration to rainfall only events and the expected dominance of rainfall runoff for the predicted peak flow, the rainfall-runoff parameters are adjusted to match a precipitation-only event. This approach provides appropriate results for the Project design.

b. The model was not run for the 2005 and 2013 events following adjustment and, therefore, the requested results are not available. The initial model calibration resulted in a model that represented the conditions, at the time of the 2005 and 2013 events, with rainfall patterns specific to that event. Evaluation of the model using modelled floods with more uniform spatially applied rainfall produced results outside expected values and, thus, adjustments were made to model for use with the modelled antecedent and probable maximum precipitation (PMP) rainfall distributions.

   The probable maximum flood (PMF) estimates provided by the model are consistent with past studies and are within the expected range for locations within Alberta. Figure IR520-1 (see the response to IR520) presents the proposed PMF peak on the Creager diagram demonstrating this relationship.

c. Insufficient calibrated radar rainfall time-series data was available to calibrate or validate to the other two flood peaks in 2005.

REFERENCES

NOAA (National Oceanic and Atmospheric Administration). 2015. Regional Snow Analyses: Norther Rockies. National Operational Hydrologic Remote Sensing Center. http://www.nohrsc.noaa.gov/nsa/index.html?region=Northern_Rockies&year=2013&month=5&day=19&units=e#text [accessed March 2015].

Question 452

Volume 1, Section 3.1, Page 3.2
Volume 1, Section 3.2.5.1, Page 3.15
Volume 3D, Section 1.5.1, Page 1.18

Alberta Transportation states that *The off-stream dam is classified as an "extreme" consequence dam….* Similarly, in Volume 1, Section 3.2.5.1, Page 3.15 Alberta Transportation states that *The dam and its appurtenances are designed as an "Extreme" hazard facility in accordance with CDA Guidelines and Alberta Dam and Canal Safety Guidelines*. In other places within the EIA (i.e. Volume 3D, Section 1.5.1), Alberta Transportation states that the dam has been designed to standards consistent with a hazard classification of *very high* per the AEP guidelines and *extreme* per the CDA guidelines.

a. The Alberta Dam and Canal Safety Guidelines (March 1999) by AEP are no longer used for determination of consequence classification. Clarify the consequence classification.

Response 452

a. The off-stream dam is classified as an "Extreme" consequence facility in accordance with CDA (2007).

The statement in Volume 1, Section 3.2.5.1, Page 3.15 should be replaced with "The dam and its appurtenances are designed as an "Extreme" consequence facility in accordance with CDA Guidelines ~~and Alberta Dam and Canal Safety Guidelines.~~"

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 453

Volume 1, Section 3.1, Page 3.2
Volume 1, Section 3.2.1.4, Page 3.8

Alberta Transportation states that ….*the floodplain berm is classified as a "very high" consequence dam. As such, the system elements are designed to safely pass the required dam safety design flows; the probable maximum flood (PMF) for the dam and 1/3 between the 1:1000 year and PMF for the floodplain berm.* In Volume 1, Section 3.2.1.4, Page 3.8, Alberta Transportation states that *The height and southerly extent of its alignment, as determined by dam safety requirements for "Very High" consequence dams, prevents a circumvention by flood events, up to 1/3 between the 1:1000 year and the PMF*. The CDA states that the IDF for a *Very*

***High* consequence dam is 2/3 between the 1:1000 year and PMF. Alberta Transportation states in other sections of the EIA that the consequence classification for the floodplain berm is *High*.**

a. Clarify throughout the EIA which consequence classification and IDF is applicable for the floodplain berm. Update the sections as required so they are consistent.

Response 453

a. The hazard classification for the floodplain berm is "High" in accordance with the CDA (2007). There are two corrections to Volume 1 regarding the consequence classification for the floodplain berm, as follows:

- Section 3.1, page 3.2: "The off-stream dam is classified as an "extreme" consequence dam and the floodplain berm is classified as a "~~very high~~ High" consequence dam.
- Section 3.2.1.4, page 3.8 to page 3.9: "The height and southerly extent of its [ed. floodplain berm] alignment, as determined by dam safety requirements for "~~very high~~ High" consequence dams, prevents a circumvention by floods, up to 1/3 between the 1:1,000 year and the PMF.

The inflow design flood (IDF) discharge for the floodplain berm is 2,210 m^3/s which is 1/3 between the 1:1,000 year flood of 1,930 m^3/s and the probable maximum flood (PMF) of 2,770 m^3/s.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 454

<u>Volume 1, Section 3.2.1, Page 3.5</u>

Alberta Transportation states that *The hydraulic performance and debris management features of the diversion inlet and service spillway were refined using 2-dimensional hydraulic modelling and a 1:16 scale physical model...*

a. Provide the physical model study report by the National Research Council Canada to provide details on the modelling work

b. It appears that the situation with the diversion gates closed and river discharges approaching 160 m^3/s was not modelled with sediment loading. What is the risk that sediment will deposit immediately upstream of the diversion structure during the early part of the flood while the diversion gates are closed, resulting in reduced diversion capacity?

c. Discuss the level of confidence that the diversion will perform as intended under flood, debris, and sediment loading, addressing the uncertainty introduced by:

   i. The necessary adjustments that had to be made for modelling purposes, such as the truncation of the grain size distribution curve;

   ii. The fact that the combination of simultaneous sediment movement and floating debris was not simulated.

Response 454

a. The *Physical Model Study of the Springbank Off-stream Storage Project Diversion Structure on the Elbow River Report* by the National Research Council Canada is provided as Appendix IR454-1.

b. The risk of sediment deposition upstream of the diversion structure during the early part of a flood, while the diversion gates are closed, is low. Prior to operation of the diversion gates, the service spillway gates will be lowered to the fully open position with the bottom hinged gates lowered flush to the river bed, which will provide a smooth unobstructed flow from upstream to downstream.

   The invert elevation of the service spillway structure is El. 1,210 m and the invert elevation of the diversion structure inlet is El. 1,211.5 m, which makes the diversion structure inlet 1.5 metres higher. This elevation difference will flush silt depositions away from the diversion structure inlet and carry them downstream through the service spillway.

   Large sediment deposition on the gates in the open position is not expected because the service spillway gate width is sized to hydraulically match the river at bankfull, thereby maintaining the sediment transport capacity of the river at preflood stages. If sedimentation over the gate were to occur, the sediment could be “lifted” by the pneumatic bladder during operations or would be washed out during flows on the rising limb of the hydrograph. Sediment accumulation is not expected to impact the ability to meet the target flood diversion rate.

c. The level of confidence that the diversion will perform as intended under flood, debris, and sediment loading is high. The diversion structure (including service spillway and diversion inlet gates) is designed to allow for flexible operation and includes additional capacity to address uncertainty associated with the highly complex system of Elbow River. The maximum diversion capacity of 600 m^3/s is 25% greater than the diversion discharge required to mitigate for the design flood (2013 flood event).

   i. The physical model was not intended to represent the full operating hydrograph for the flood and the effect of sediment on the diversion capacity. Rather, its purpose was to gain insight into deposition patterns within the channel and the usefulness of a sluice gate in the service spillway. Sedimentation observed in the physical model was consistent with results from the numerical model; therefore, it was deemed no further consideration of sedimentation using the physical model was needed.

ii. Sediment transport and debris have been considered, as appropriate, using both physical and numerical modeling. Results of that modeling were used to inform decisions about the maximum capacity of the diversion inlet and the flexibility of operations for the service spillway. A diversion capacity 25% over the modelled capacity needed for the design flood (2013 event) is appropriate.

Question 455

Volume 1, Section 3.2.1, Figure 3-1, Page 3.3

Borrow sources #1 and #2 appear to be along the toe of the valley walls.

a. **Will the borrow excavations impact the stability of the perimeter slopes after final grading and reclamation? Explain why or why not.**

b. **What testing was carried out for the potential borrow materials? Describe the suitability of the materials based on the test results.**

Response 455

a. Yes, excavation of the perimeter slopes will change the slope stability from current conditions. However, the final slopes will be designed to the meet minimum factor of safety criteria for short-term and long-term stability. Conceptual layouts for the borrow source excavation used maximum slopes of five horizontal to one vertical. These are anticipated to remain stable after final grading and reclamation. Final configuration of the borrow source will be selected by the construction contractor. Construction specifications will dictate the required factors of safety for these slopes and these will be designed by a professional engineer.

b. Five geotechnical borings were performed in Borrow Source 1. Most of the soils encountered were identified as glacial lacustrine and glacial till that are suitable for use in the embankment as Zones 1A and 2A. Further exploration and testing are anticipated as the design progresses.

Question 456

<u>Volume 1, Section 3.2.1.1, Page 3.6</u>

For the diversion inlet:

a. Provide the design diversion capacity and water level, main structure components, and principal dimensions and elevations.

b. Describe measures to prevent seepage/piping and drainage measures.

c. Describe erosion protection measures (e.g. at entrance and downstream of the stilling basin).

d. Describe construction requirements including temporary works (e.g. site clearing and grubbing, dewatering systems, foundation preparation, etc.).

Response 456

a. The diversion inlet is a concrete gate structure located on the left bank of Elbow River at the entrance to the diversion channel. The primary elements of the diversion inlet are:

- left abutment, retaining walls, and embankment transitions
- two 20 m wide gate-bays with a center pier-divider and fixed crest El. 1,211.5 m
- two 20 m wide by 4 m tall vertical lift-gates with wire-drum hoist and hoist support spanning the 20 m bay width
- access bridge—comprising bridge deck, breastwall, and headwall—that provides access to gate equipment, vehicle access across the diversion channel entrance, and it is a debris and overtopping barrier during extreme flood events
- stilling basin with chute blocks and end sills to provide energy dissipation and minimize channel erosion
- right abutment, retaining walls, and embankment transitions

A cross section through the centerline of the diversion inlet is presented in Figure IR456-1. The design maximum discharge capacity (600 m^3/s) is calculated to occur at headwater El. 1,215.8 m.

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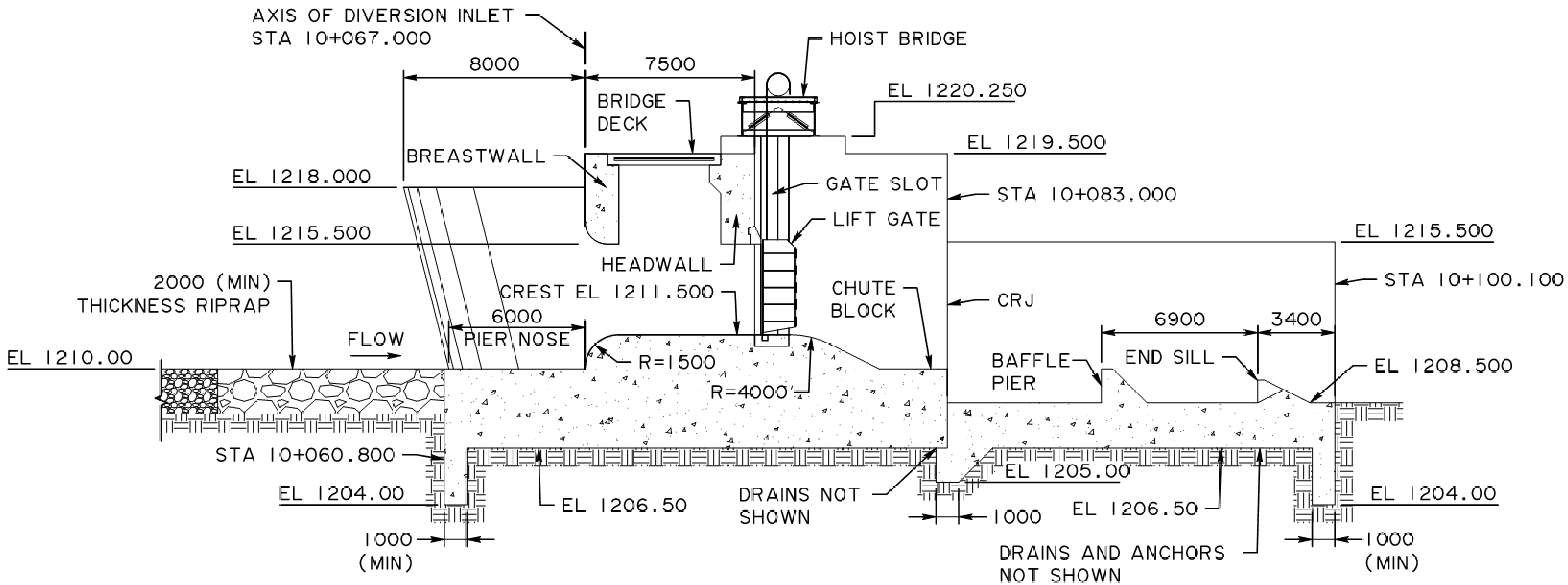

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Sources: Base Data- Government of Alberta, Government of Canada. Thematic Data - Stantec Ltd.

Disclaimer: This map is for illustrative purposes to support this Stantec project; questions can be directed to the issuing agency.

Stantec

ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESEVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Diversion Inlet Design

Figure IR456-1

b. Final design of the diversion inlet structure is currently in progress; seepage will likely be mitigated through construction of a "grout curtain" in the underlying rock deposits. Drainage material and conduits will be installed beneath the structure.

c. Riprap is located for up to 10 m upstream and for up to 46 m downstream of the diversion inlet. The riprap is designed to prevent erosion by withstanding the anticipated velocities and shear stresses for the range of operating conditions.

d. The following are construction considerations for the diversion inlet:

- Clearing and grubbing will be performed in conjunction with the excavation of the diversion channel.
- The excavation will be isolated from Elbow River using cofferdams and groundwater seepage will be pumped from the excavation.
- Foundation preparation will require cleaning and preparation of concrete/rock interface. Preparation will also include installation of foundation underdrain to control uplift pressures.
- Anchors will be required to maintain adequate factors of safety against floatation in the stilling basin. The anchors will be drilled and grouted in a grid pattern prior to placement of the stilling basin concrete.
- The diversion inlet structure and gates will be functional before a tie-in with the diversion channel is made. The gates will be load tested to verify functionality of the diversion inlet gates and gate structure.

Question 457

Volume 1, Section 3.2.1.2, Page 3.7

For the service spillway:

a. Provide the design discharge capacity (e.g. gates down) and design water levels (e.g. gates down and gates in operation).

b. Describe the main structure components and provide the principal dimensions and elevations.

c. Describe measures to prevent seepage/piping and drainage measures.

d. Explain the reasons for backfilling the stilling basin with native substrate. Does this have to be maintained?

e. **Clarify if erosion and destabilization of the left downstream bank between the river and the diversion channel is a concern. If it is, describe the measures that are being provided to address it. If it is not a concern explain how this conclusion was reached.**

f. **Clarify if redundancies have been included to ensure that the gates can be operated (raised to permit diversion) during a flood. Explain the implications of not being able to raise one or both gates to divert flows under various flood events.**

g. **Describe construction requirements including temporary works (e.g. site clearing and grubbing, dewatering systems, foundation preparation, etc.).**

Response 457

a. A series of rating curves was developed for the service spillway for six gate positions, based on results of the physical model and numerical models. Figure IR457-1 provides the relationship between headwater elevation and discharge for the six corresponding gate positions for a single 24 m bay.

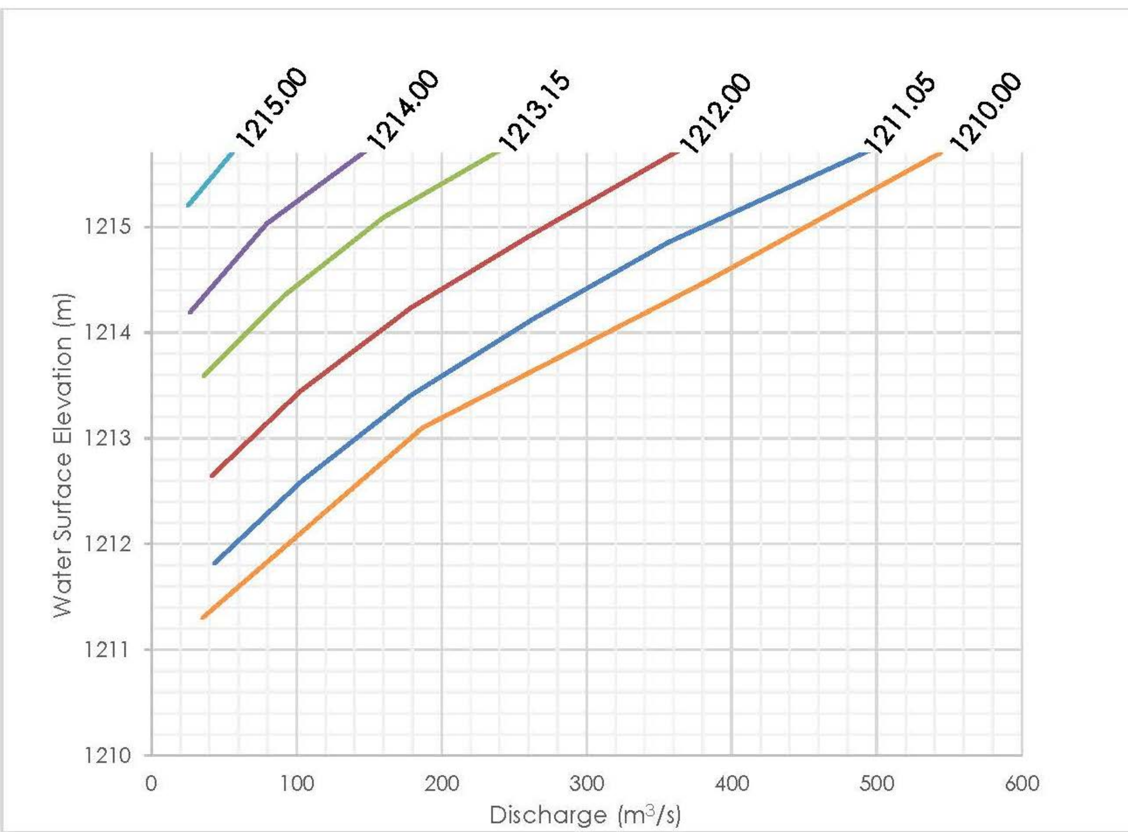

Figure IR457-1 Service Spillway Gate Rating Curve (24 m Wide Bay)

b. The service spillway is a concrete gate structure, located on the main channel of Elbow River, regulates river and diversion flow. The primary elements of the service spillway are:

- left abutment, retaining walls, and embankment transitions
- two 24 m wide gate-bays with a center pier divider and fixed crest at El. 1,210.0 m
- two 24 m wide by 5 m tall bottom hinged crest gates with pneumatic bladder control
- stilling basin with end sills to provide energy dissipation and address channel erosion during gate operations
- right abutment, retaining walls, and transition to roller compacted concrete auxiliary spillway.

A cross section through the centerline of the service spillway is presented in Figure IR457-2.

c. Design of the service spillway structure is currently in progress; seepage will likely be mitigated through construction of a grout curtain in the underlying rock formation. In addition, drainage material and conduits will be installed beneath the structure as appropriate.

d. The stilling basin will be backfilled with native substrate to facilitate fish passage. The material is expected to remain stable during typical annual flows. During high flow events, the material may be washed out, but is expected to be replaced by normal river processes.

e. Risk of erosion and destabilization of the left river bank is mitigated by the extension of the retaining wall 43 m downstream of the stilling basin. Hydraulic model results indicate a dissipation in velocity and shear stress at this location as the flow spreads out from the service spillway. Further, erosion of the bedrock downstream of the service spillway indicate that scour depth will be limited to less than 3 m in the bedrock and is not expected to effect bank stability.

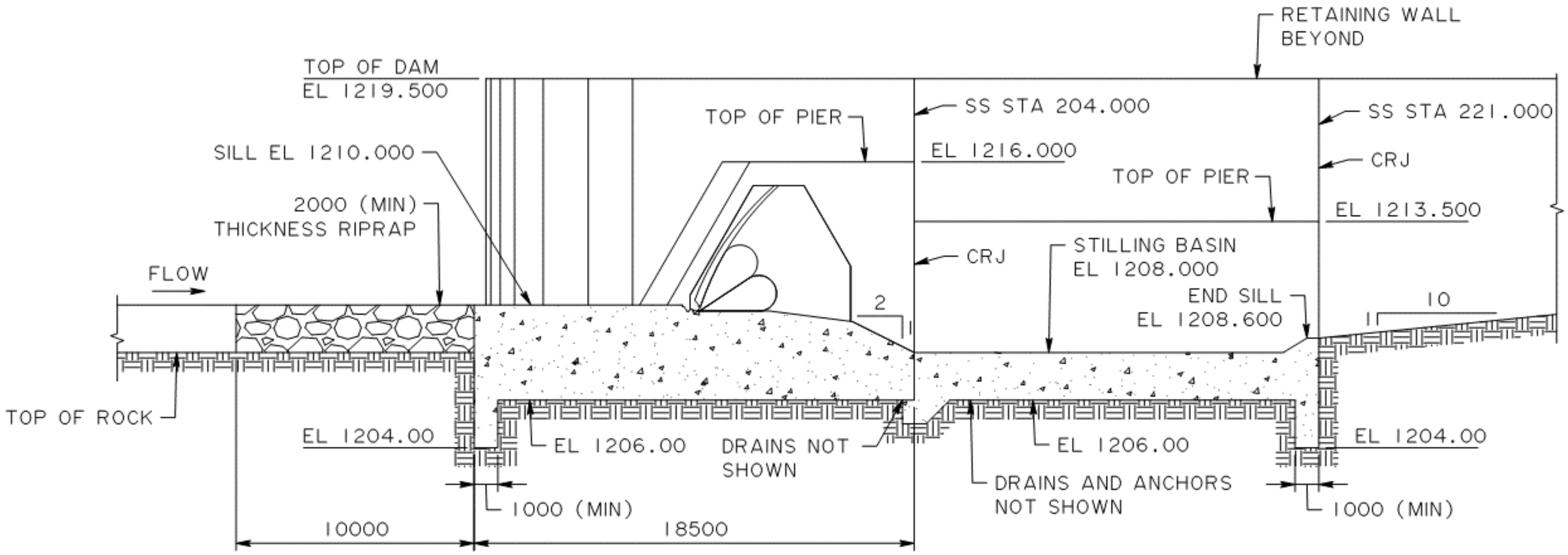

Figure IR457-2 **Cross Section of Service Spillway**

f. Redundant systems for the service spillway flood operations include:

- backup power generation
- backup air supply for the bladder system
- independent gate bay operation
- segmented panels in each bay with independently supplied air bladders

See Table IR4547-1 for operational implications for four scenarios of service spillway gate failure.

Table IR457-1 Spillway Service Gates Failure Scenarios

Failure Scenario for the Service Spillway Gates	Peak River Discharge (m^3/s)	Peak Diversion Rate (m^3/s)	Total Diversion Volume (dam^3)
2013 Event, Both Gates Non-Functional	1,240	393	30,800
2013 Event, One Gate Non-Functional	1,240	600	51,500
1:100 Year Event, Both Gates Non-Functional	765	224	12,000
1:100 Year Event, One Gate Non-Functional	765	360	20,700

For the 1:100 year flood, the Project goal (maintaining flows downstream of Glenmore Dam below 160 m^3/s) will be met with only one service spillway gate functional. If the flood exceeds the 1:100 flood or both service spillway gates fail to operate, flows downstream will be substantially less than without the Project, but may not meet the defined flood mitigation goal, which may result in some flooding downstream of Glenmore Dam.

g. The following items have been identified as construction considerations for the service spillway:

- There will be restricted periods for disturbance within the Elbow River including May 1 to July 15 and September 16 to April 14. This means that construction of a river diversion and the coffer-works for the service spillway will take place between April 15 and April 31 or July 16 and September 15.
- Portions of the right abutment that tie into roller compact concrete placement of the auxiliary spillway may serve as a component of the water control plan during construction and may need to be functional prior to completion of the diversion structure. Many of these details are at the discretion of the construction contractor but will need to be coordinated with the engineer to ensure appropriate loading conditions have been considered in the design.

- Shear key at the upstream edge of the gate bay is required to maintain adequate sliding stability. Care will be taken during excavation of shear key trench to identify unsuitable rock conditions or weak beading planes that could affect capacity of the shear key.
- Foundation preparation will require special care in cleaning and preparation of concrete–rock interface. Preparation will also include installation of foundation underdrain to relieve uplift pressures during spillway operation. Foundation preparation details will be developed during final design.
- Anchors will be required to maintain adequate factors of safety against floatation in the stilling basin. These could be static anchors drilled and grouted in a grid pattern prior to placement of the stilling basin concrete. Anchor details to be provided during final design.
- Hinge anchors, airlines, control lines, restraining strap pockets, and wall plates for service spillway crest gates will be considered during concrete preparation and placement. Placement tolerance for some of these items are more stringent than for typical heavy construction projects due to close fitting of structural components and operating clearance requirements.

Question 458

Volume 1, Section 3.2.1.3, Page 3.8

For the control building:

a. Describe the primary and backup power supply and communications provisions for the gates and controls.

Response 458

a. The control building will be situated adjacent to the retaining wall between the diversion inlet and the service spillway. The building will provide a climate-controlled environment for the electrical and pneumatic devices used to control and monitor the gate systems.

PRIMARY AND BACKUP POWER SUPPLY

Primary power supply will be provided by Fortis Alberta. Fortis has indicated that there is three-phase power currently running along Highway 22 in the PDA. The routing of power lines will be high above ground level and out of flood areas, with overhead lines following the access roads to the control building.

Electrical loads served by the single-phase distribution panel will include the Obermeyer gate air compressor, the crest gate programable logic controller (PLC) control panel, main PLC control panel, radio communications equipment, exhaust fan, space heater, lighting, and auxiliary power receptacles.

Three-phase electrical loads will include the vertical lift gate hoist motors and abutment heaters.

A permanent backup generator will be housed inside the control building. A wall will separate the generator room from the main control room. The primary fuel source for the backup generator will be natural gas and an external propane tank will provide a secondary source of fuel if natural gas is not available. Natural gas and propane are preferred choices because of the lower maintenance requirements when compared to diesel fuel and the need to fuel cycle.

A generator receptacle mounted to the exterior of the control building will be available to plug in a portable generator if utility power is unavailable, and the backup generator fails to run. A manual transfer switch will be designed into the system for operators to control the source of generator power.

COMMUNICATIONS

Primary monitoring and operation of the gate systems will utilize a PLC control system panel within the control building.

Communication between the gate position sensors, operators and the PLC will be transmitted through ethernet cabling to the PLC panel. Water level staff-gauges will be utilized, as visual reference guides, to enable manual local operations of the gates should communications to the panel be interrupted.

Communication between staff will use cellular phones.

Primary communication from the monitoring stations to the control building will use wireless cellular data networks.

Primary communication from the control building to AEP Operations will use the 9,000 MHz radio system that provides communications for the western headworks system. Backup communications will use cellular data networks.

Question 459

<u>Volume 1, Section 3.2.1.4, Pages 3.8 and 3.9</u>

Alberta Transportation states that *The height and southerly extent of its alignment* (i.e. floodplain berm), *as determined by dam safety requirements for "Very High" consequence dams, prevents a circumvention by flood events, up to 1/3 between the 1:1000 year and the PMF.* On Page 3.9, Alberta Transportation also states that *The crest is set at 1 m above the calculated 1:1,000 year flood elevation and would pass the probable maximum flood without overtopping.*

a. Clarify the consequence classification and design flow for the floodplain berm and the corresponding design freeboard that accompanies this design flow. Update the sections as required so they are consistent.

b. Clarify the statement that the floodplain berm is capable of passing the PMF.

Response 459

a. The classification for the Floodplain Berm is "High" consequence. Following the CDA (2007), the diversion structure is designed to safely pass the Inflow Design Flood (IDF) for a "High" consequence facility, with sufficient freeboard from overtopping. A minimum freeboard of 1 m is provided for the IDF for the length of the embankment that exceeds 1 metre in height. This includes the first 700 m of embankment length from the auxiliary spillway to the southeast. For the remaining sections of the floodplain berm, the embankment height is less than 1 m and freeboard is reduced to 0.5 m.

b. Although not identified as the IDF, the probable maximum flood (PMF) was analyzed in the hydraulic modeling. The service spillway and auxiliary spillway hydraulic geometry provide sufficient capacity to pass the PMF with the water surface elevations below the floodplain berm crest height for the full length of the structure; this indicates that river flows associated with a PMF would not overtop the floodplain berm.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 460

<u>Volume 1, Section 3.2.1.4, Figure 3-5, Page 3.9</u>
<u>Volume 1, Section 3.3.1.2, Page 3.23</u>

Figure 3-5 shows a typical cross section of the floodplain berm with a deep cutoff trench penetrating the alluvial soils and weathered bedrock. The cutoff trench is backfilled with impervious fill Zone 1A material and a filter zone is provided on the downstream side. There is no mention that groundwater dewatering will be required in the alluvial soils. The seepage cutoff is intended to protect the embankment against piping during flood operation. Also, the upstream slope of the floodplain berm is protected with riprap, however, no bedding gravel layer(s) is shown.

a. What means of dewatering in the alluvial soils are being considered to facilitate placement of the filter layer on the excavation slope as well as placement of the impervious fill?

b. Due to the depth of cutoff trench and potential groundwater problems, were other cutoff means considered? If so, what was the basis for selecting an impervious fill cutoff? If other cutoff means were not considered explain why.

c. Does the riprap protection include bedding gravel layer(s)? Explain.

Response 460

a. Design advancement has eliminated the cutoff trench beneath the floodplain berm. Therefore, dewatering of the alluvial soils will not be required. An updated cross section is presented in Figure IR460-1. The change in design will not change the conclusions of the environmental effects. The change in design will lower the cost of construction minimally but will not have a material effect on the Project economics and/or decisions based upon the economics.

b. As a diversion structure not intended for long-term detention, seepage beneath the structure is permissible, so long as stability of the structure is maintained. The berm has been designed to meet appropriate slope stability and seepage factors of safety in accordance with CDA (2007).

c. The riprap protection on the upstream face does not include a bedding gravel layer. In lieu of a bedding gravel layer, a non-woven geotextile filter material will be used. The geotextile fabric provides a cost-effective method for separation between the larger riprap and finer underlying soils.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 461

Volume 1, Section 3.2.1.5, Page 3.10
Volume 1, Section 3.2.1.4 , Page 3.9

Alberta Transportation states that *The auxiliary spillway is designed to withstand overtopping for flood events up to 1/3 between the 1:1000 and the PMF with an overtopping depth of 1.5 m*. It was previously stated in Section 3.2.1.4 that the floodplain berm is capable of passing the PMF.

a. Clarify the flood criteria for the design of the auxiliary spillway.

b. Are all of the facilities on the Elbow River designed on the same basis (e.g. same consequence classification and same IDF)?

c. Can the auxiliary spillway pass the PMF? If so, under what conditions related to the diversion and service spillway flows during the PMF? If not, explain how the PMF is managed.

Response 461

a-b. The service spillway, auxiliary spillway and floodplain berm are considered a contiguous structure on Elbow River with a dam safety hazard classification of “High” consequence and an inflow design flood (IDF) that is 1/3 between the 1:1,000 year flood and the probable maximum flood (PMF). The auxiliary spillway, as part of the floodplain berm, is also classified as a “High” consequence structure; therefore, it is designed to the same standard for the IDF.

c. The auxiliary spillway is not designed to pass the PMF. This means that the structure is not designed to meet the selected factors of safety for stability and hydraulics under a PMF operating condition. For a PMF, the service spillway gates would be fully open, and flows will pass through the service spillway and over the auxiliary spillway. During this event, the auxiliary spillway will have limited effect on water surface elevations and river flows up and downstream of the structure.

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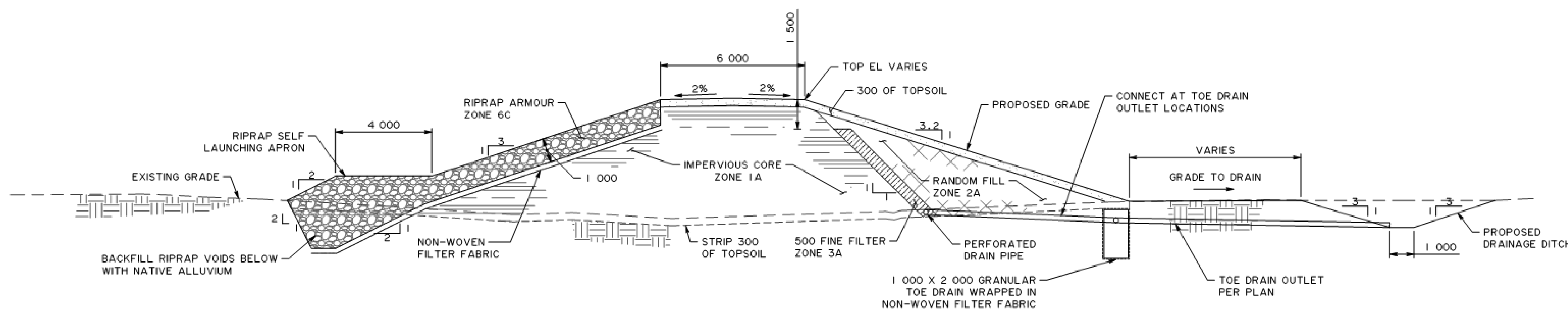

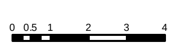

ST-CAL-110773396-290 REVA

NAD 1983 3TM 114

Sources: Base Data- Government of Alberta, Government of Canada. Thematic Data - Stantec Ltd.

Disclaimer: This map is for illustrative purposes to support this Stantec project; questions can be directed to the issuing agency.

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ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESEVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Floodplain Berm Cross Section - October 2018

Figure IR460-1

Question 462

Volume 1, Section 3.2.1.5, Page 3.10

Alberta Transportation states *The auxiliary spillway may also activate for smaller flood events if the conveyance capacity is reduced by debris and sediment at the diversion inlet and service spillway and operations of the gates are not adjusted*.

a. How is the influence of debris and sediment taken into account for the floodplain berm design for the crest elevation?

b. Explain the implications if the auxiliary spillway is activated.

c. If the auxiliary spillway activates at smaller flood events then depending on the maximum discharge capacity of the activated auxiliary spillway, will it still be possible to divert flows to utilize the full storage capacity of the Springbank Offstream Reservoir? Explain why or why not.

Response 462

a. A design freeboard of 1 m is provided over the calculated water surface elevations for the inflow design flood (IDF). Major wind-generated wave setup and runup is not anticipated upstream of the floodplain berm due to the presence of the existing tree cover. Therefore, the freeboard is provided to address the potential effects of turbulence, debris and sediment on upstream water surface elevations.

   Results of the sediment transport analyses using the 2D numerical model indicate that over an extended period of seven days, at a flow rate in the river of 765 m^3/s, deposition of sediment will result in a 0.2 m increase of the water surface elevations upstream of the floodplain berm and auxiliary spillway. At higher flow rates in the river, when the auxiliary spillway is activated, the impacts of sediment and debris are expected to be less, because each incremental increase in water surface elevation results in a substantial gain in discharge capacity in the auxiliary spillway, due to its long length (235 m).

b. If the auxiliary spillway is activated, the cover material may erode along with some of the downstream floodplain. The crest elevation of the concrete weir is set at El. 1,215.8 m to maintain sufficient diversion capacity following activation of the auxiliary spillway. Flows over the auxiliary spillway will be conveyed downstream along the floodplain and re-enter the main Elbow River channel downstream.

c. If the auxiliary spillway activates at smaller flood events, it will be possible to divert flows to utilize the full capacity of the off-stream reservoir. Figure IR462-1 displays the rating curve for the diversion inlet gates. As indicated on the figure, water level at the fixed concrete crest of the auxiliary spillway (El. 1,215.8 m) would result in diversion flows up to the maximum diversion channel capacity of 600 m^3/s. This exceeds the required diversion rate of 480 m^3/s to mitigate flooding for the design flood event; therefore, the system will be able to divert enough volume to utilize the full capacity of the off-stream reservoir for smaller events.

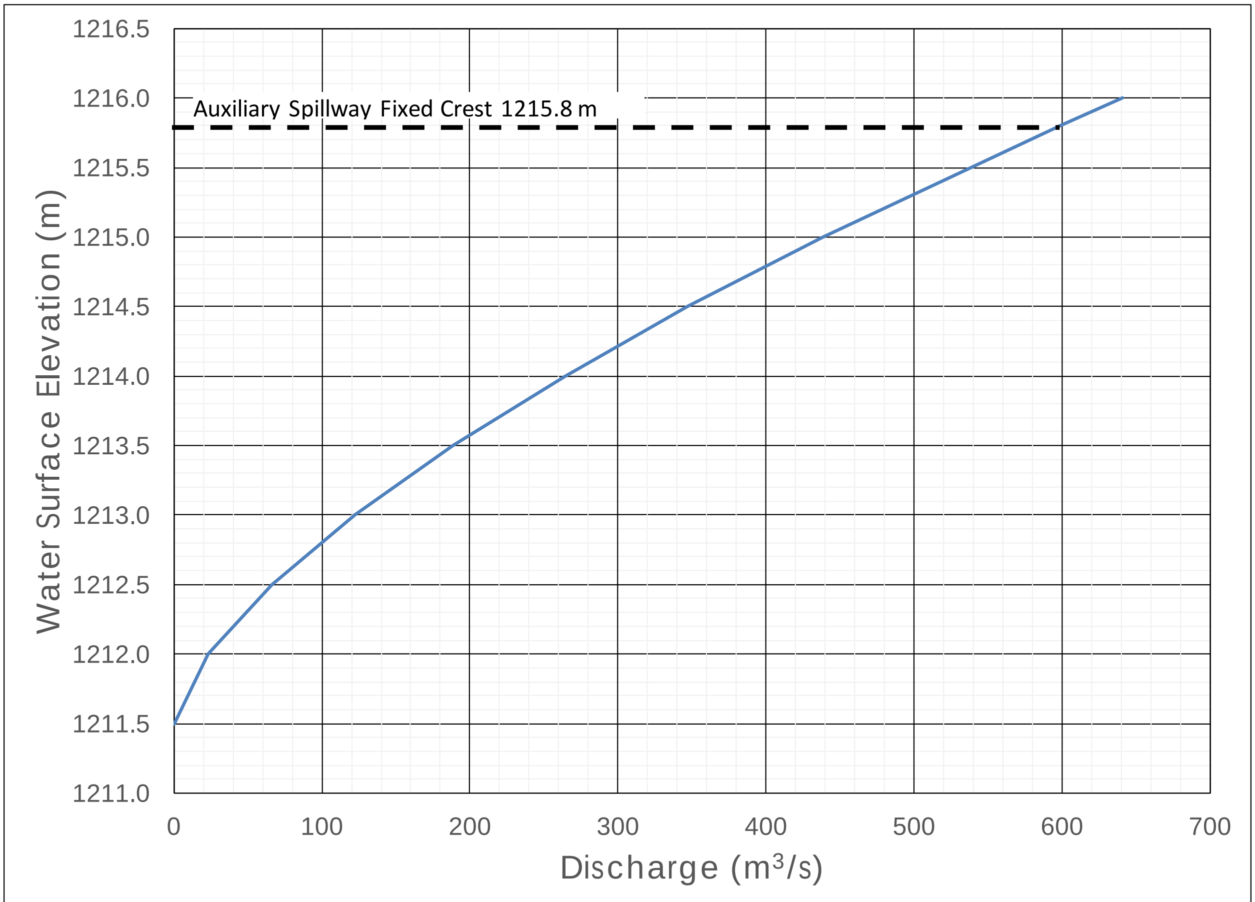


Figure IR462-1 Diversion Inlet Rating Curve

Question 463

Volume 1, Section 3.2.2, Page 3.11

For the diversion channel:

a. Describe the soil conditions where bedrock is not present and its erodibility.

b. Where does the channel cross existing large creeks? Explain how these channel crossings are being addressed.

c. Provide elevations (top of bank, invert and FSL) at the upstream and downstream ends of the canal and at the emergency spillway.

d. Describe the channel outlet and grade control structure, their function and design, and provide the main dimensions and elevations.

e. Provide typical design cross sections in cut and fill.

f. Describe the excavation and stockpiling requirements.

g. For the two bridges across the diversion channel, explain if large debris that finds its way past the diversion inlet can collect at the bridge piers and block the channel.

h. Describe construction requirements including temporary works (e.g. site clearing and grubbing, dewatering systems, foundation preparation, etc.)

Response 463

a. Areas of the diversion channel, where bedrock is not present, are comprised of glacio-lacustrine units overlying glacial tills. Characteristics of the soils are described in the response to IR525.

   For channel sections at structural risk from scour and erosion, riprap channel lining is provided. These areas include utility crossings, bridge foundations, the emergency spillway and areas of embankment that retain channel flow and the reservoir. For areas of lower risk, vegetation is proposed. At these locations, vegetation is expected to provide erosion protection for diversion flow rates up to 370 m^3/s. This exceeds the diversion rate for a 1:50 year flood. This erosion is expected to be localized and will be repaired following a flood.

b. The diversion channel will cross one existing stream (unnamed tributary 1350) approximately 200 m upstream of Highway 22. This stream will be re-directed into the diversion channel and the section of the channel will be lined with riprap armouring. In addition, smaller drainage swales and ditches will be intercepted by the diversion channel downstream of Highway 22. Riprap armouring will be provided on the inlet side of the channel for these locations, as well.

c. The diversion channel invert at upstream end is El. 1,207.140 m. The channel slopes at -0.10% to Station 13+970.624 at El. 1,203.298 m, then -0.20% to the top of the stepped roller compacted concrete (RCC) grade control structure at Station 14+570 and El. 1,202.286 m. The overflow section at the bottom of the RCC grade control structure is located at Station 14+624.763 at El. 1,195.624 m. The design flow depth is about 6.4 m and at least 1.9 m of freeboard is provided along the channel.

The diversion channel invert is El. 1,203.944 m at Station 13+325 (start of emergency spillway). The overflow crest elevation for the emergency spillway is El. 1,210.750 m. The top of the emergency spillway berm is El. 1,213.500 m.

The requested elevations along the diversion channel are provided in Table IR463-1.

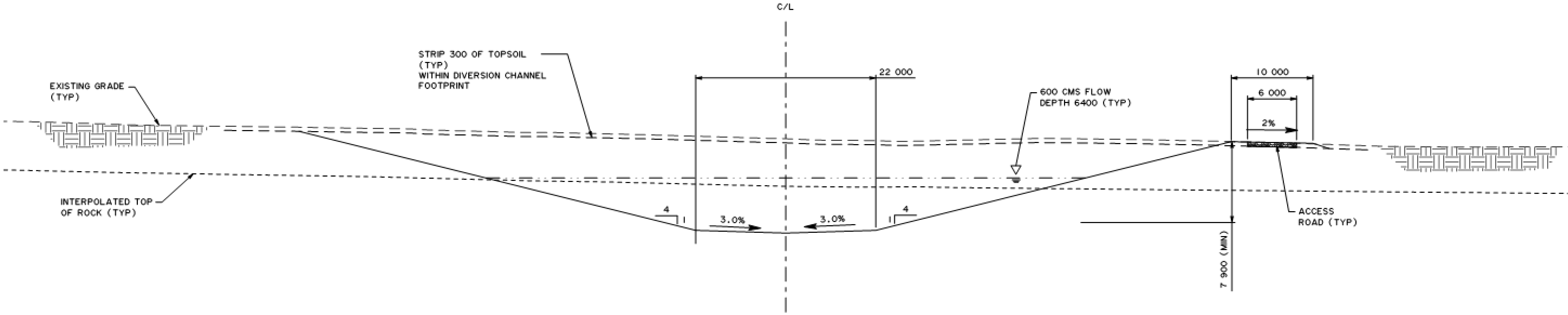
Table IR463-1 Diversion Channel Elevations

Location	Invert El. (m)	Top of Bank El. (m)	Full Service Level El. (m)
Upstream (diversion inlet near the river)	1,207.14	1,236.17	1,213.65
Emergency spillway	1,203.94	1,215.07	1,210.75
Downstream (diversion channel outlet near the reservoir)	1,202.29	1213.50	1210.75

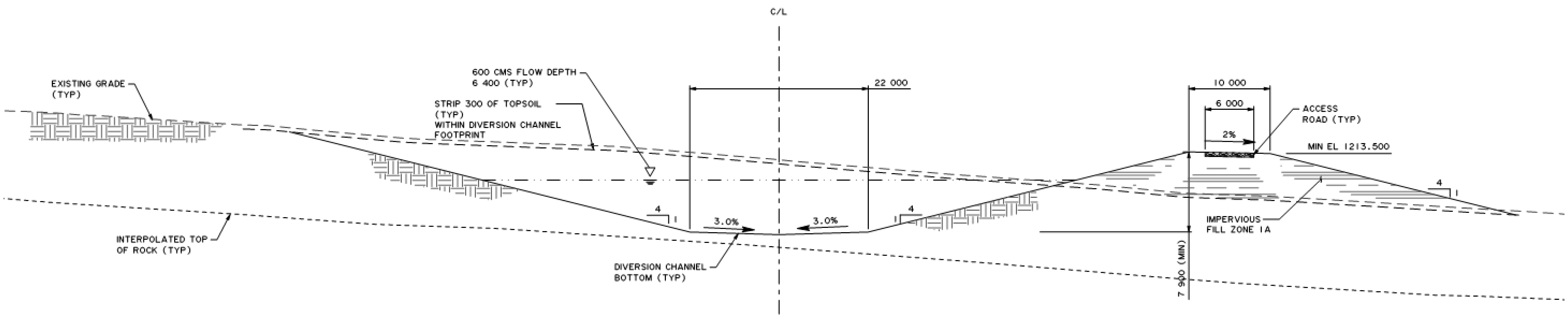
d. At the end of the diversion channel, the channel section transitions from a bottom width of 24 m to a width of 150 m, before it drops approximately 7 m over an RCC grade control structure into the reservoir. The width transition is lined with large diameter riprap armouring and the RCC control structure is designed to dissipate energy when the reservoir begins filling. The RCC structure has 600 mm tall steps arranged on an average slope of 5.5H:1V. The grade control structure has an integral stilling basin at the bottom to further dissipate energy. The grade control structure becomes partially to fully inundated as the reservoir fills.

The purpose of the channel widening and grade control structure is to spread flow and dissipate energy prior to discharge into the reservoir.

e. Figure IR463-1 provides typical diversion channel sections in cut and fill sections. These are representative for the channel through rock and/or soil.



A DIVERSION CHANNEL – TYPICAL CUT SECTION
1:250



B DIVERSION CHANNEL – TYPICAL FILL SECTION
1:250

0 3.75 7.5 15 22.5 30
Metres

ST-CAL-110773396-290 REV A

NAD 1983 3TM 114

Sources: Base Data- Government of Alberta, Government of Canada. Thematic Data - Stantec Ltd.

Disclaimer: This map is for illustrative purposes to support this Stantec project; questions can be directed to the issuing agency.

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ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESEVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Diversion Channel Typical Sections - March 2017

Figure IR463-1

f. Excavation and stockpiling requirements will conform with Civil Works Master Specifications (CWMS) *for Construction of Provincial Water Management Project, Section 02315* (Volume 4, Supporting Documentation, Document 13). Staging and spoil areas will be designated by the contractor. Excavation and embankment sequencing will be determined by the construction contractor within the guidelines established in the specifications.

g. The debris deflector (Canadian Environmental Assessment Registry [CEAR] #25) will retain most large debris in the river channel and limit passage into the diversion channel. There are no AEP requirements or guidelines that are applicable to the debris deflector.

   Furthermore, the pier spacing, and vertical height of the bridge openings are larger than the diversion channel inlet gate openings. Therefore, it is expected debris that passes through the gates should be able to pass through the bridges. In addition, the bridge piers are positioned such that they do not obstruct the center of the main channel of Elbow River.

h. Details regarding construction sequencing and the required temporary measures will be determined by the construction contractor. Prior to clearing and excavation activities, erosion and sediment protection measures will be installed. Clearing and grubbing will be performed according to CWMS for *Construction of Provincial Water Management Project*, Sections 01390, 02322 and 02930 (Volume 4, Supporting Documentation, Document 10 and Document 11). Topsoil will be stripped and stockpiled on site for later use. Excavation will be sequenced by the contractor for use in the embankment construction and to meet the requirements of the construction schedule. Dewatering will be performed, as needed, and will be directed to appropriate sediment controls prior to discharge to a local water course. Dewatering will be in accordance with *CWMS Section 02240 - Care of Water* (Volume 4, Supporting Documentation, Document 12) and *CWMS Section 02242 - Turbidity Barriers* (Volume 4, Supporting Documentation, Document 9).

REFERENCES

Canadian Environmental Assessment Registry (CEAR #525). 2018. Available at:
https://www.ceaaacee.gc.ca/050/documents/p80123/122722E.pdf

Question 464

<u>Volume 1, Section 3.2.3, Page 3.11</u>

Alberta Transportation states *The (emergency) spillway has a crest at the reservoir full service elevation of 1,210.75 m and a discharge capacity of 354 m³/s at 1.25 m of head*. The design capacity of the diversion canal is 600 m³/s, however, flows may exceed this amount during the PMF.

a. What is the maximum capacity of the emergency spillway and at what elevation does this occur?

Response 464

a. The design capacity of the emergency spillway is 354 m^3/s at a water surface El. 1,212.0 m, which occurs at the maximum pool elevation during the routing of the 24-hour probable maximum flood (PMF) for a scenario in which the diversion inlet gates fail to close. This is illustrated with the reservoir routing for the PMF presented in Figure IR464-1, where the peak reservoir elevation in the reservoir is reached after the maximum inflow rate occurs.

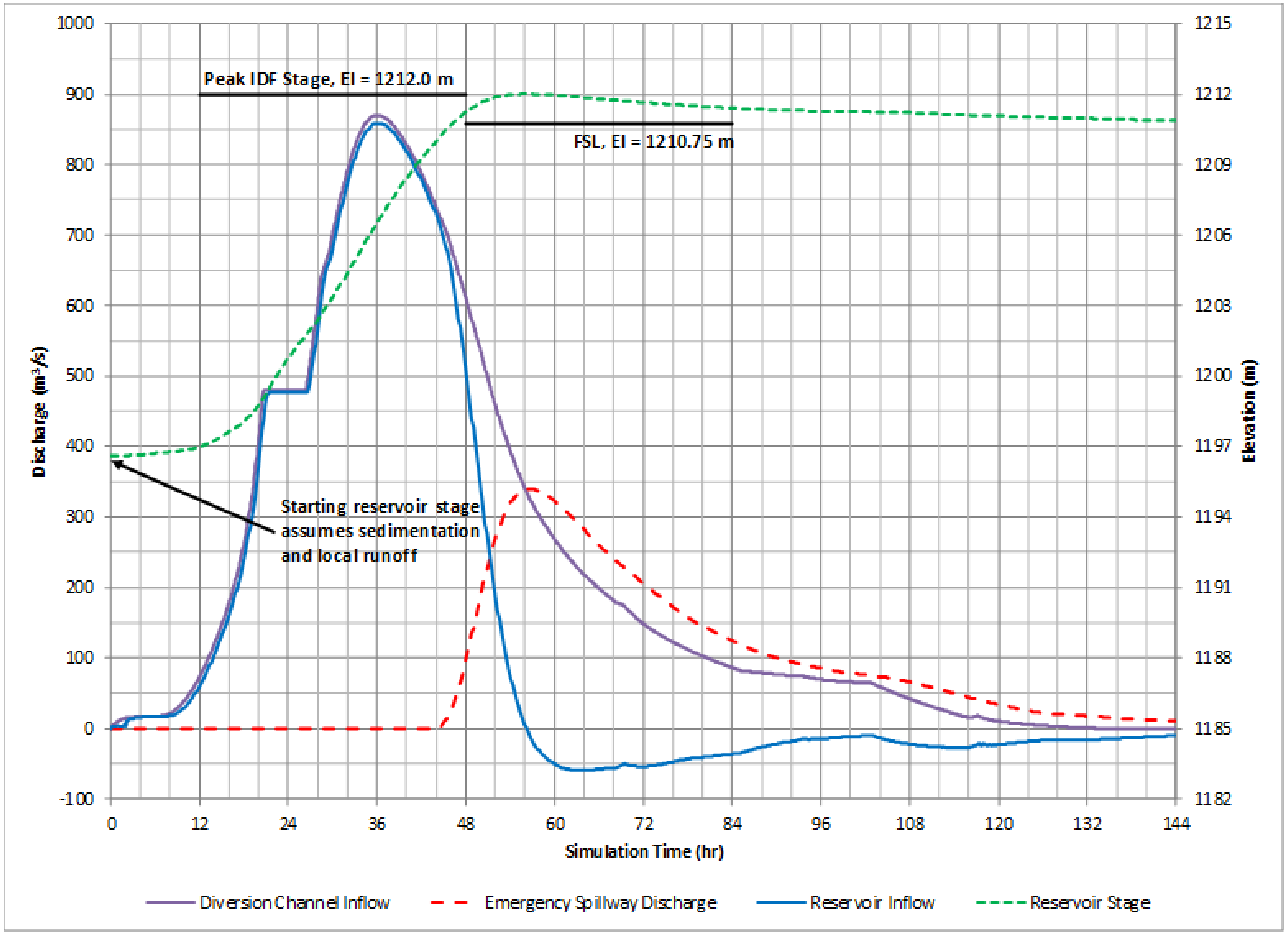


Figure IR464-1 Reservoir Inflow Routing for the Probable Maximum Flood with No Diversion Inlet Gate Closure

Question 465

Volume 1, Section 3.2.4, Table 3-3, Page 3.12

The residence time and release time for the 2013 design hydrograph is less than the 1:100 year flood. Alberta Transportation states *Table 3-3 shows the reservoir filling times for hypothetical floods and the 2013 design hydrograph. Actual residence time and release rates will vary depending on conditions downstream post flood, performance of the dam while storing water, and other factors*.

a. **Why are the residence and release times for the larger 2013 flood less than that for the 1:100 year flood?**

b. **Explain the reservoir evacuation criteria (e.g. levels and times) in case of an emergency at the dam, that are being used.**

c. **Provide the reservoir's operating rules with actual filling and releasing times/rates/levels corresponding to various flood events and conditions.**

Response 465

a. For the design flood, the gate is assumed to be fully opened. For the 1:100 year flood, the gate is assumed to be partially closed resulting in the off-stream reservoir holding water longer and increased residence and release times compared to a design flood. This approach provides more detail on the effects of a range of flow rates over a common period, in contrast to maximum release rates over different time periods. The actual operational release rate from the off-stream reservoir will vary, depending on circumstances at the time of diversion and release.

b. Alberta Transportation (2004) and CDA (2007) do not specify evacuation requirements for flood reservoirs. Therefore, the reservoir drawdown criteria are based on USBR (1990). The USBR method bases the proposed drawdown criteria on a combination of dam hazard classification and risk evaluation. The USBR hazard classification terminology differs from the CDA guidelines, wherein the highest level for USBR classifies as "High", while the CDA classifies as "Extreme". A "High" risk category was selected because of the facility's infrequent operations.

   Table IR465-1 provides the reservoir evacuation criteria (drawdown time) for a High Risk – High Hazard Dam according to the USBR. Target drawdown times are provided for various pool levels that are referenced to the maximum pool depth at the full service level. As an example using the first row of the table, the criteria requires the storage pool (i.e., the water in the reservoir) to be drawn down to 75% of the maximum depth in 20 to 30 days.

Table IR465-1 USBR Reservoir Drawdown Time Criteria

Reservoir Stage Relative to Depth at Full Service (flood pool stage) Level	Minimum Drawdown Time (Days)
75% Maximum Depth	20-30
50% Maximum Depth	40-50
25% Maximum Depth	50-60
10% Storage Volume	70-90

Figure IR465-1 presents the drawdown rates for the off-stream reservoir with the prescribed flood pool stage identified. The blue line on the figure represents the reservoir stage over time. At each of the defined criteria stages, the elevation and drawdown time are noted. As an example, the 75% stage (El. 1,204.3 m) is achieved in 19.5 days, which meets the minimum criteria of 20 to 30 days.

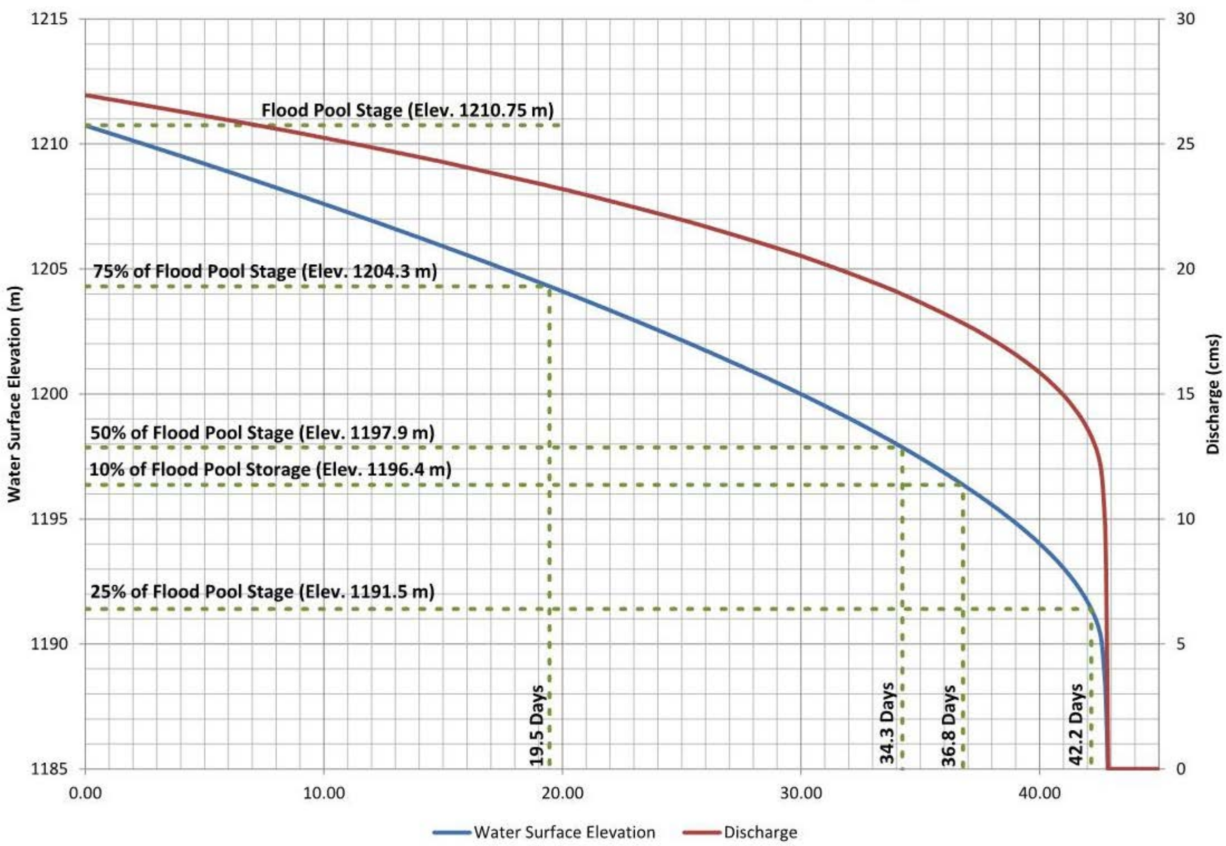


Figure IR465-1 Reservoir Drawdown from Full Service Level

c. As stated, flood release rates will vary depending on conditions downstream during post-flood operations. For instance, regardless of flows in the Elbow River, if post-flood repairs were occurring on the Elbow River or Bow River downstream of the Project, AEP Operations may hold water within the off-stream reservoir until such time that the repairs are complete and the additional flow can be added to Elbow River. Specific guidelines and operating rules have not been defined because these conditions are expected to vary for each flood.

REFERENCES

Alberta Transportation. 2004. Water Control Structures Selected Design Guidelines. Alberta Transportation, Civil Projects Branch. Edmonton, AB.

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

USBR (U.S. Dept. of the Interior, Bureau of Reclamation). 1990. ACER TM No. 3, Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works, Denver, Colorado.

Question 466

<u>Volume 1, Section 3.2.4, Page 3.12</u>

TOR 2.6[F] B requires Alberta Transportation to *Describe the construction activities for reservoir preparation, including: … methods for managing wood debris and shoreline stabilization during reservoir filling*. Alberta Transportation provided a discussion about debris management at the diversion structure but there is no discussion about shoreline stabilization within the reservoir.

a. Provide a discussion regarding shoreline stability during reservoir filling and drawdown.

Response 466

a. Shoreline instability may occur from the effects of reservoir filling, wave action and drawdown. Annually, prior to flood season, the reservoir area and potential shoreline will be generally inspected for slope stability.

During a flood and when the off-stream reservoir is retaining water, areas adjacent to structures susceptible to damage from shoreline instability will be monitored. Observed instabilities that pose a danger to critical structures and operations will be addressed immediately. Areas observed that do not pose an immediate risk to critical structures will be addressed after the reservoir is drained.

After reservoir drawdown, the off-stream reservoir area will be inspected for signs of slope instability and erosion.

Areas of identified instability during these operating conditions will be addressed through re-grading, re-vegetation and/or placement of armouring on slopes.

Question 467

Volume 1, Section 3.2.4, Table 3-2, Page 3.12

Table 3-2 does not indicate the reservoir level corresponding to each of the flood magnitudes.

a. Provide the reservoir levels corresponding to each flood magnitude.

Response 467

a. The reservoir levels for each of the flood magnitudes are:

- 1:10 year, El. is 1,192.5 m
- 1:100 year, El. is 1,203.6 m
- design flood (2013 hydrograph), El. is 1,207.5 m

Question 468

Volume 1, Section 3.2.5.1, Figure 3-8, Page 3.14
Volume 1, Section 3.2.5.4, Page 3.17

The foundation conditions including any variances that may occur underlying the offstream dam are not described. In particular, the differences in soil conditions between the majority of dam foundation and the areas at the dam abutments and No Name Creek are not described.

a. Provide a discussion on foundation materials underlying the dam and saddle dyke including what seepage control requirements are necessary and how these requirements might vary across the bottom of the valley and abutment areas. Describe or show the typical dam cross sections in the vicinity of the low level outlet near the No Name creek channel.

b. Explain why a nominal cutoff trench would be sufficient for part of the dam when a deep cutoff trench is required for the floodplain berm on the Elbow River and drains are proposed near the low level outlet structure.

Response 468

a. The subsurface soils beneath the proposed dam and saddle dyke are glaciogenic units consisting mostly of glacial lacustrine and glacial till soils. The following is a description of each soil unit and location, following standard geotechnical protocols, which may differ from soils science protocols.

- Glacial lacustrine (GL) is described as an olive green to brown, intermediate to high plasticity, and lean to fat clay. Particle-size distribution testing indicated that GL typically comprises between 50% and 70% clay-sized particles with a subordinate of (typically 30% to 50%) silt-sized particles. The thickness of GL typically ranges from 2 m to 6 m at the western and eastern ends of the dam. In the center of the dam, GL is thicker, ranging from 8 m to 15 m in thickness. GL is not present near the unnamed creek that runs through the reservoir.
- Glacial till (GT) is described as brown to grey, low to intermediate plasticity, and lean clay with sand and gravel. There is large variability and inconsistency in the thickness of the GT layer. Within the off-stream dam footprint, the thickness ranges from 1 m to 15 m, with the thickest layers generally are in the center portion of the off-stream dam.
- A layer of sand and gravel soils were encountered in the low-lying area of the unnamed creek. The sands are dense, brown, clayey sand with gravel and silty sand with gravel. The gravel is very dense, clayey and silty gravel with sand. The thickness of these soils ranges from 1 m to 7 m.

The foundation materials immediately underlying the reservoir and dam (GL or GT) have vertical hydraulic conductivity ranging from 3.0×10^{-11} m/s to 3.0×10^{-10} m/s. Based on these low permeability values and the temporary nature of the proposed impoundment of water, seepage protection measures are not required under most of the dam embankment. For the areas near the outlet structure, two rows of 1 m drains are extended to the sand and gravel soil layers and connected to the filter and drainage blanket. The drains relieve potential seepage gradients through the sand and gravel soil layers encountered near the unnamed creek (to become the flow path for water released from the reservoir and back into Elbow River). No direct connection has been identified between the reservoir and this layer because these soils were overlain by a clay till soil in each of the borings conducted during the geotechnical investigation.

Seepage analyses were performed to determine the steady-state phreatic surface at the inflow design flood (IDF) pool elevation (El. 1,212 m for retained water in the reservoir). This is a conservative assumption considering the temporary nature of the water to be retained in the reservoir. The headwater pool was modeled as a head boundary condition at El. 1,212 m. The tailwater pool was modeled as a head boundary condition to replicate the fall in the groundwater from the dam location to the river water elevation.

The exit gradients from the seepage model were evaluated. The average gradient in the upper one meter of the foundation soils just downstream of the toe of the dam was determined. The critical exit gradient for the foundation soil was calculated as 0.93. The factor of safety against piping due to heave was then calculated. For each of the design cross sections, seepage factors of safety were greater than 9, indicating no further mitigation measures were required.

Results of the seepage analysis for the typical cross section underlain by GL/GT is shown in Figure IR468-1.

Results of the seepage analysis for the cross section through the unnamed creek is shown in Figure IR468-2.

b. A cutoff trench at the floodplain berm is not needed.

However, analysis indicates seepage control beneath the off-stream dam is needed in the area where the sand and gravel occur in the foundation. Drains were deemed the most appropriate method of this control. Additional evaluations are being performed.

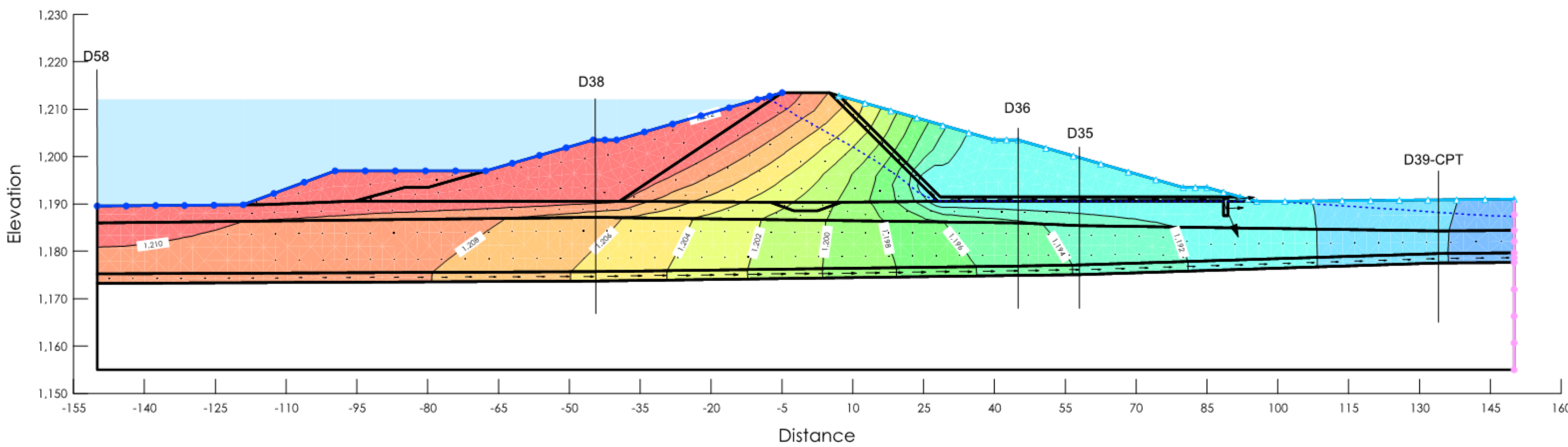


Figure IR468-1 Seepage Gradients for Sta. 22+925

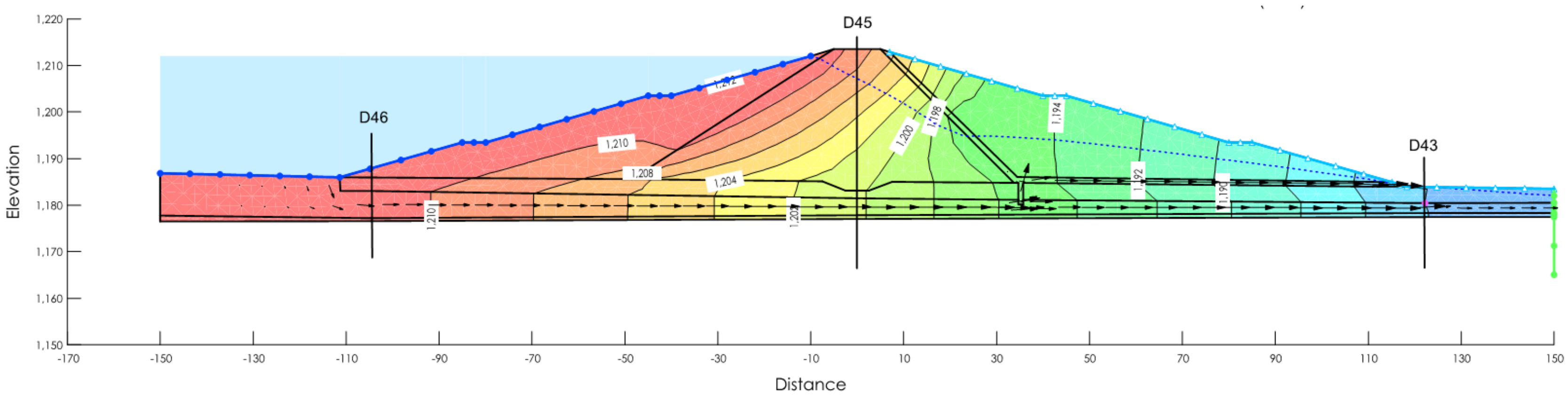


Figure IR468-2 Seepage Gradients for Sta. 23+175

Question 469

Volume 1, Section 3.2.5.1, Page 3.15

The composition of the dam includes an impervious core, random fill shells and a sand/fine filter internal drain. Quantities of the various construction materials are not provided. Other embankments such as the floodplain berm are described similarly with the exception that some areas of the embankments are also protected with riprap on the upstream slope.

a. **Provide the approximate quantities of material used to construct the dam and appurtenant structures.**

b. **Discuss the various zone types for fill materials to be used in the dam embankments including specification requirements.**

c. **Will high plastic soils or bedrock be encountered in the borrow areas and required excavations? Will these then be used in embankments? Explain why or why not.**

d. **What portion of the excavation and borrow materials will consist of high plastic materials?**

e. **If more than one type of random fill is used in the dam, how will the embankment be zoned for these materials which require different placement and compaction procedures?**

Response 469

a. For approximate quantities of material used for construction of the dam embankment and appurtenant drainage features, see Table IR469-1.

Table IR469-1 Dam Construction Quantities

Component	CWMS (Civil Works Master Specifications)[1]	Quantity (m^3)
Dam Embankment	Impervious Fill Zone 1A	1.5 million
Dam Embankment	Random Fill Zone 2A	3.4 million
Dam Embankment	Fine Filter Zone 3A	240,000
Dam Embankment	Topsoil	70,000
Dam Embankment (Drainage Channels)	Riprap Zone 6B	550
Inlet/Outlet Channel Armouring	Riprap Zone 6A	480
Dam Access Road	Road Gravel Zone 4B	1,700
SOURCE: [1] Alberta Transportation (2019)		

Quantities for the low-level outlet works are provided in the response to IR526.

b. The interior of the dam consists of a low permeability core and exterior embankment shells. A drain is located on the downstream face of the core and along the existing ground. A vertical toe drain and key trench are also included. The design includes provisions for a toe buttress on the upstream toe of the dam depending on performance of the foundation and pore water pressure dissipation during embankment construction.

The dam earthwork will be placed in accordance with Alberta Transportation (2019) for the specified zones with the exceptions noted below.

The embankment core will be constructed using Impervious Fill Zone 1A. Supplemental specifications for the Impervious Fill Zone 1A will be required to meet the design intent of the off-stream dam. Impervious Fill Zone 1A embankment core may include high plastic glacio-lacustrine clay (CH) soils compacted to a minimum of 95% of standard Proctor value and placed with an allowable moisture content ranging from optimum moisture content to plus 4%. The recommended minimum liquid limit is 50%, with a maximum particle size of 75 mm.

The embankment shell will be constructed using Random Fill Zone 2A. There will be three subclasses of Random Fill Zone 2A, based on the materials which are to be excavated from the diversion channel and the borrow source:

- For Random Fill Zone 2A (1), the selected soil embankment may include moderate to highly plastic glaciolacustrine clay soils or glacial till clay soils placed in the embankment shell and compacted to a minimum of 95% of standard Proctor value and placed in maximum 250 mm lifts with an allowable moisture content ranging from -2% to +2% of optimum moisture content.
- For Random Fill Zone 2A (2), the non-durable rock/soil embankment will consist of soil and weathered, non-durable bedrock (Slake durability index [SDI] <85) placed in maximum 200 mm lifts. Large rock fragments will be broken down into pieces less than 150 mm (in any dimension) or removed from the lift. Non-durable rock will be broken down and watered to the satisfaction of the engineer prior to compaction. All Zone 2A (2) materials will be approved by the engineer and compacted to 95% of the standard Proctor value, or as required by the engineer.
- For Random Fill Zone 2A (3), the rock fill embankment will consist of sound, durable sandstone and shale rock fill within the embankment shell zones with a minimum SDI value of 85. The maximum lift thickness will be 600 mm (with a maximum particle size of 450 mm).

The filter and blanket drain will be constructed with sand fill using Fine Filter Zone 3A. Due to the very fine nature of the base soils, the filter criteria window is relatively narrow and will require the filter gradation to be carefully controlled during construction. A suitable filter for both the core and the downstream foundation are provided by a standard fine aggregate commonly used for bituminous paving mixtures (AASHTO M 29-96, Grading No. 3.).

c. High plastic glacial lacustrine soils (plasticity index greater than 20) and bedrock will be encountered in the borrow source materials and diversion channel excavated materials. Materials will be used in the dam embankment construction in accordance with Alberta Transportation (2019) and noted supplemental specifications described above. The GL and rock materials combined makeup more than half of the material to be excavated from the diversion channel. When placed according to the design specifications, they are suitable materials for construction of a dam embankment. Given these materials are suitable, the use of GL and rock from the required channel excavation reduces borrow source requirements which in turn reduces the total impact of the Project on the environment and provides the most cost-effective materials for dam construction.

d. Approximately 40% of the excavated channel and borrow source materials will be high plastic clay. Specifications allow for placement of this material within both Zones 1A and 2A; however, the specific volume and composition of the embankment will be determined by field conditions and sequencing during construction.

e. Three types of random fill are anticipated. These consist of (1) moderate to high-plastic glacio-lacustrine or glacial till, (2) non-durable rock consisting of soil and weathered bedrock and non-durable shale, claystone, and mudstone bedrock, and (3) rock fill consisting of durable sandstone and shale rock. Placement requirements will vary for each of the three types of random fill. Random fill will be placed as unique types in horizontal lifts and different types will not be comingled.

REFERENCES

Alberta Transportation. 2019. Water Management Structures Selected Design Guidelines. Accessed at: https://www.alberta.ca/water-management-structures-selected-design-guidelines.aspx

Question 470

<u>Volume 1, Section 3.2.5.2, Page 3.15</u>

Alberta Transportation states *Since the reservoir will not have a permanent pool, wave wash protection will not be necessary. In addition, any flood pool will be a temporary condition*.

a. Since residence time in the reservoir is in the order of 40 days and can be delayed longer if downstream damages are significant thereby limiting outflows that may be released, how much erosion could occur during the period of impoundment?

b. Is there a critical elevation range near FSL that should include riprap protection? Explain.

c. Is there any infrastructure or buried lines that will be present on the upstream slope in the vicinity of the low level outlet that may be exposed or damaged due to a wave attack? If so, how will these facilities be protected?

d. If there is slope erosion during the period the reservoir is inundated, how would the corresponding sediment accumulation in the bottom of reservoir while the gates are shut affect gate operation of the low level outlet when they are eventually opened?

Response 470

a. Short-term erosion potential from storm-induced wave development was evaluated for the embankment including the 1:2 year and 1:1,000 year sustained wind events. Calculations show that, for a sustained wind event of four (4) hours, potential erosion is less than 47 mm for the 1:1,000 year event and 36 mm for the 1:2 year event. These erosion depths are acceptable because they are less than the expected vegetated turf root depth.

   The erosion estimates provide a reasonable range of potential erosion during the period of impoundment. Given the referenced recurrence intervals, it is unlikely for multiple events with sustained winds of equal or greater magnitude to occur while the pool is impounded and prior to inspection and repair.

b. No, the maximum reservoir levels will vary for each flood based on the severity of flood and the diverted volumes. The higher reservoir elevations with greatest potential for wave propagation will occur less frequently. The maximum reservoir elevation will not be maintained over a long period of time. Given these factors, riprap protection on the upstream slope is not cost effective when compared to repairing potential surficial erosion.

Further recommendations from Hughes (2010) for earthen levee systems conclude "with a reasonable degree of certainty that the flood-side armoring is not required … where (a) the earthen levees are constructed of good quality clay and (b) significant wave height is not expected to exceed 1.5 m."

Note the maximum wave height in the off-stream reservoir for the 1:1,000 wind event is 1.49 m which, based on Hughes (2010), indicates flood-side armouring is not required.

c. Hydraulic lines and vent piping are to be buried within the embankment at a minimum depth of 1,500 mm. Erosion from waves is not anticipated to reach the elevation associated with this depth. Vegetation and soil cover are sufficient to protect these utilities from erosion.

d. Slope erosion is not expected to affect the opening of the outlet structure gates.

REFERENCES

Hughes, Stephen A. 2010. Flood-Side Wave Erosion of Earthen Levees: Present State of Knowledge and Assessment of Armoring Necessity. Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center. Vicksburg, MS.

Question 471

Volume 1, Section 3.2.5.2, Page 3.16

Alberta Transportation states *The design also includes a storm drainage channel along the toe of the dam, upstream and downstream. The channel is sized to convey runoff from the 100-year storm and is grass lined*. There is no mention that gravel armour or riprap would be placed in ditches on the steep sections of the toe ditch such as the abutment slopes.

a. Is there a requirement to include erosion protection for some sections of the toe ditch where gradients may be excessive for grass protection? Is there a specific gradient where erosion protection would change from grass lining to armour?

Response 471

a. The design of the toe ditches, including erosion protection, will be updated during final design. Erosion protection for these ditches will be provided for the 1:100 year flow rate using the methods established in Alberta Transportation *Erosion and Sediment Control Manual* (Volume 4, Supporting Documentation, Document 6). Where vegetation is not sufficient for the channel gradient, gravel armour or riprap will be provided to protect against erosion.

Question 472

Volume 1, Section 3.2.5.3, Table 3-4, Page 3.16

Alberta Transportation states that *Table 3-4 identifies the evaluated load cases and the required factors of safety* for the dam embankment. Alberta Transportation uses the terms *instream design flood, inflow design flood, design flood*, and *design flow*. According to the glossary (Section 9), the first two terms have different meanings, but the referenced table equates the design flood with the IDF.

a. Clarify that the design meets the minimum required factors of safety and indicate calculated values.

b. Provide a discussion to differentiate between the various terms.

c. Clarify the definition of the IDF in the footnote of the referenced table and the list of definitions.

Response 472

a. Table 3-4 contains incorrect terminology. The term "instream design flood" should be the "inflow design flood" (IDF). The IDF meets the minimum required factors of safety. The calculated factors of safety for the dam embankment are presented in Table IR472-1. Updates to the design will meet or exceed the target factors of safety indicated within the load case column of the table.

Table IR472-1 Stability Analyses Results – Recommended 3.5H:1V Exterior Slopes

Load Case	Section	Factors of Safety (FOS)	
		Upstream	Downstream
End of Construction – Total Stress Analysis (Target FOS = 1.3)	21+050	1.6	1.8
	21+975	1.8	1.6
	22+925	1.5	1.4
	23+175	1.6	1.5
End of Construction – B-bar Analysis (Target FOS = 1.3)	21+050	1.4	1.6
	21+975	1.3	1.4
	22+925	1.3	1.3
	23+175	1.4	1.5

Table IR472-1 Stability Analyses Results – Recommended 3.5H:1V Exterior Slopes

Load Case	Section	Factors of Safety (FOS)	
		Upstream	Downstream
Long Term Drained (Target FOS = 1.5)	21+050	1.6	1.7
	21+975	1.8	1.6
	22+925	1.6	1.6
	23+175	1.7	1.6
Flood Load – USBR Method (Target FOS = 1.2)	21+050	-	1.5
	21+975	-	1.4
	22+925	-	1.3
	23+175	-	1.6
Flood Load – USACE Method (Target FOS = 1.4)	21+050	-	1.8
	21+975	-	1.6
	22+925	-	1.4
	23+175	-	1.5
Rapid Drawdown (Target FOS = 1.3)	21+050	1.6	-
	21+975	1.7	-
	22+925	1.5	-
	23+175	1.6	-
Seismic - Pseudostatic (Target FOS = 1.0)	21+050	1.1	1.2
	21+975	0.9	0.9
	22+925	0.9	0.8
	23+175	0.9	0.9

b-c. The "instream design flood" should be replaced with "inflow design flood." This equates to the probable maximum flood (PMF) for the "Extreme" consequence structures and 1/3 between 1:1,000 year and PMF for the "High" consequence structures.

The "design flood" refers to the June 2013 flood (approximately 1:200 year flood) and not the IDF. The June 2013 flood served as the basis for the design of the flood diversion capacity and volume of temporarily retained water in the off-stream reservoir.

Question 473

Volume 1, Section 3.2.4, Table 3-2 Page 3.12
Volume 1, Section 3.2.4, Figure 3-7, Page 3.13
Volume 1, Section 3.2.5.1, Figure 3-8, Page 3.14
Volume 1, Section 3.2.5.1, Figure 3-9, Page 3.15
Volume 1, Section 3.2.5.3, Table 3-4, Page 3.16

The terms for *design flood* shown on Figure 3-7 and *IDF pool level* at El. 1212.0 m shown on Figure 3-8 are confusing. The IDF pool level is higher than the emergency spillway crest elevation of 1210.75 m.

a. Confirm if this is correct. Clarify the terminology between IDF and the 2013 Design Flood. Update the sections as required so they are consistent.

Response 473

a. It is correct that the inflow design flood (IDF) pool level is higher than the emergency spillway crest elevation. CDA (2007) defines the IDF as “the most severe inflow flood (peak, volume, shape, duration, timing) for which a dam and its associated facilities are designed.” This equates to the probable maximum flood (PMF) for the “Extreme” consequence structures and 1/3 between 1:1000 year and PMF for the “High” consequence structures.

The active flood storage pool in the reservoir, below the full service level (El, 1,210.75 m), is based on the June 2013 flood, which is referenced as the design flood (as distinct from the “IDF”) in the EIA.

IDF pool level (El. 1,212.0 m) is based on a hydrologic routing of the IDF through the reservoir, uses storage capacity above the full service level, and results in discharge through the emergency spillway.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 474

Volume 1, Section 3.2.5.5, Page 3.17
Volume 3D, Section 2.3.2.2, Page 2.6
Volume 3D, Section 2.3.2.2, Page 2.7

Alberta Transportation states that *The dam stability was assessed for an Earthquake Design Ground Motion (EDGM) with an Annual Exceedance Probability of 1/10,000, in accordance with CDA Dam Safety Guidelines (2007) and the Extreme hazard classification*. Volume 3D, Section 2.3.2.2, Pages 2.6 and 2.7 identify measures to mitigate the risk of induced seismic events such as establishing an exclusion zone around the Project for commercial operators.

a. Was dam embankment stability assessed taking into account the effects of induced seismicity or is the design contingent upon establishing an exclusion zone?

Response 474

An “induced earthquake” or “induced seismicity” is defined as an earthquake caused by human activity such as hydraulic fracturing (typically related to oil and gas production) or operation of waste injection wells.

a. An exclusion zone was presented as a possible risk management strategy; however, the design of the dam is not contingent upon the establishment of an exclusion zone.

In accordance with CDA (2007), a probabilistic seismic hazard assessment (PSHA) was performed to define ground motion parameters for use in seismic design for the dam. Existing evidence of induced seismicity was incorporated into the PSHA model through inclusion of induced events in the regional and local source models; this is accounted for in the seismic design of the dam. Based on the deaggregation of the seismic hazard, the design considers a magnitude six (6 Mw) earthquake, which is larger than any induced earthquake recorded in the Province of Alberta (Macias-Carrasco et al. 2011; Stern et al. 2016). The earthquake motion was applied in proximity (less than 25 km) to the dam. This produced a peak ground acceleration of 0.28 g, which was incorporated into the design of the dam.

During the annual dam safety review and inspection, publicly available records of recent earthquakes (natural or induced) within 25 km of the dam will be reviewed. Should new trends in activity be observed, potential impacts to the design assumptions and operations of the facility will be reviewed and mitigation strategies, if necessary, evaluated and enacted.

References

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Macias-Carrasco, M., A. Fereydouni, K. Goda, and G. Atkinson. 2011. Canadian Composite. Seismicity Catalogue (CCSC11) for Western Canada.

Stern, V.H. Schultz, R.J. Shen, L. Gu, Y.J. Eaton, D.W. 2016. Alberta Earthquake Catalogue, version 4.0 (GIS data, point features). Alberta Energy Regulator, AER/AGS Digital Dataset 2013-0017.

Question 475

Volume 1, Section 3.2.6, Figure 3-10, Page 3.17

Alberta Transportation states that *The low-level outlet structure (Figure 3-10) consists of an approach channel, discharge gate, gatehouse, discharge conduit and outlet channel. The gate is operated locally by the gatehouse*. Figure 3-10 is labeled Low Level Outlet but only shows the basin at the downstream toe. The photo in this figure is a picture of another project with a similar basin but with twin conduits that are round instead of the actual conduit shape for this project.

a. Provide the discharge capacity (e.g. rating curve).

b. Describe the main structure components and provide the principal dimensions and elevations.

c. Provide a section showing the entire configuration of the low level outlet, including upstream gate, gate control location in the gate house, control lines, vent pipe, conduit including joints, and outlet structure. Provide the backfill configuration including measures to prevent seepage/piping and drainage measures.

d. Describe erosion protection measures for the low level outlet.

Response 475

a. The preliminary rating curve for the low-level outlet works (LLOW) is provided in Figure IR475-1.

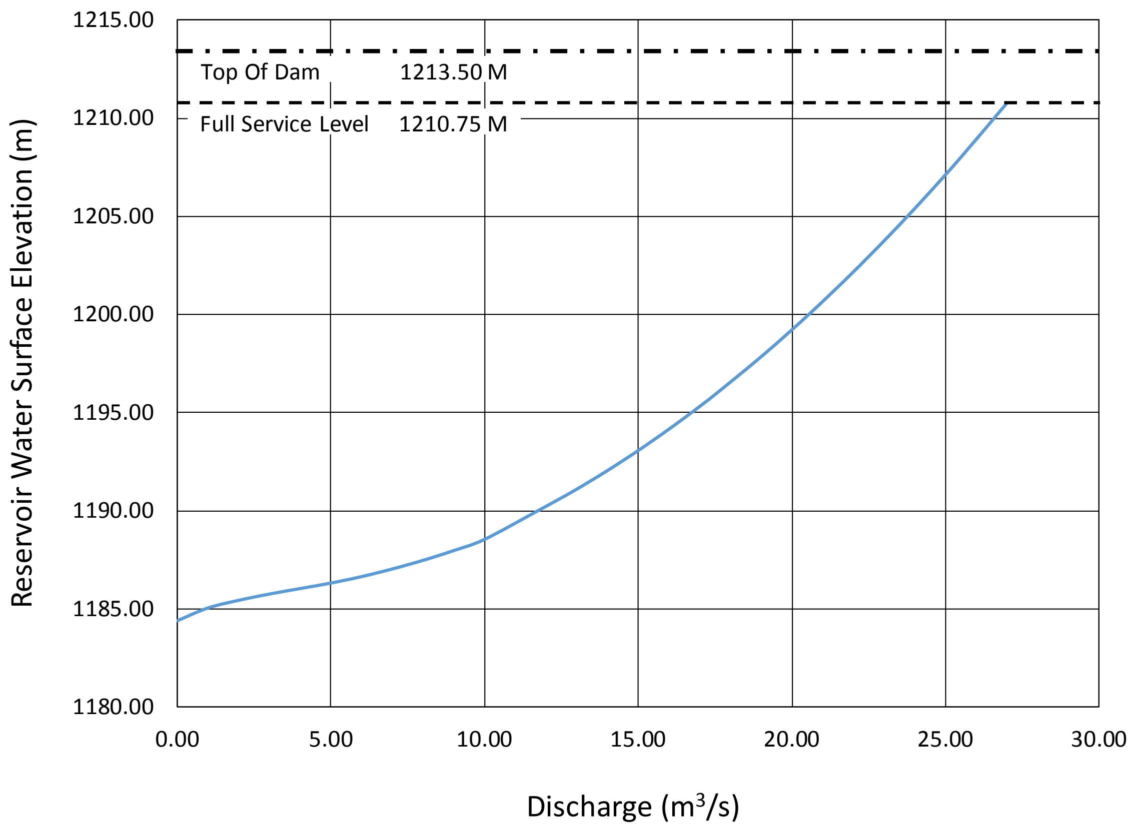


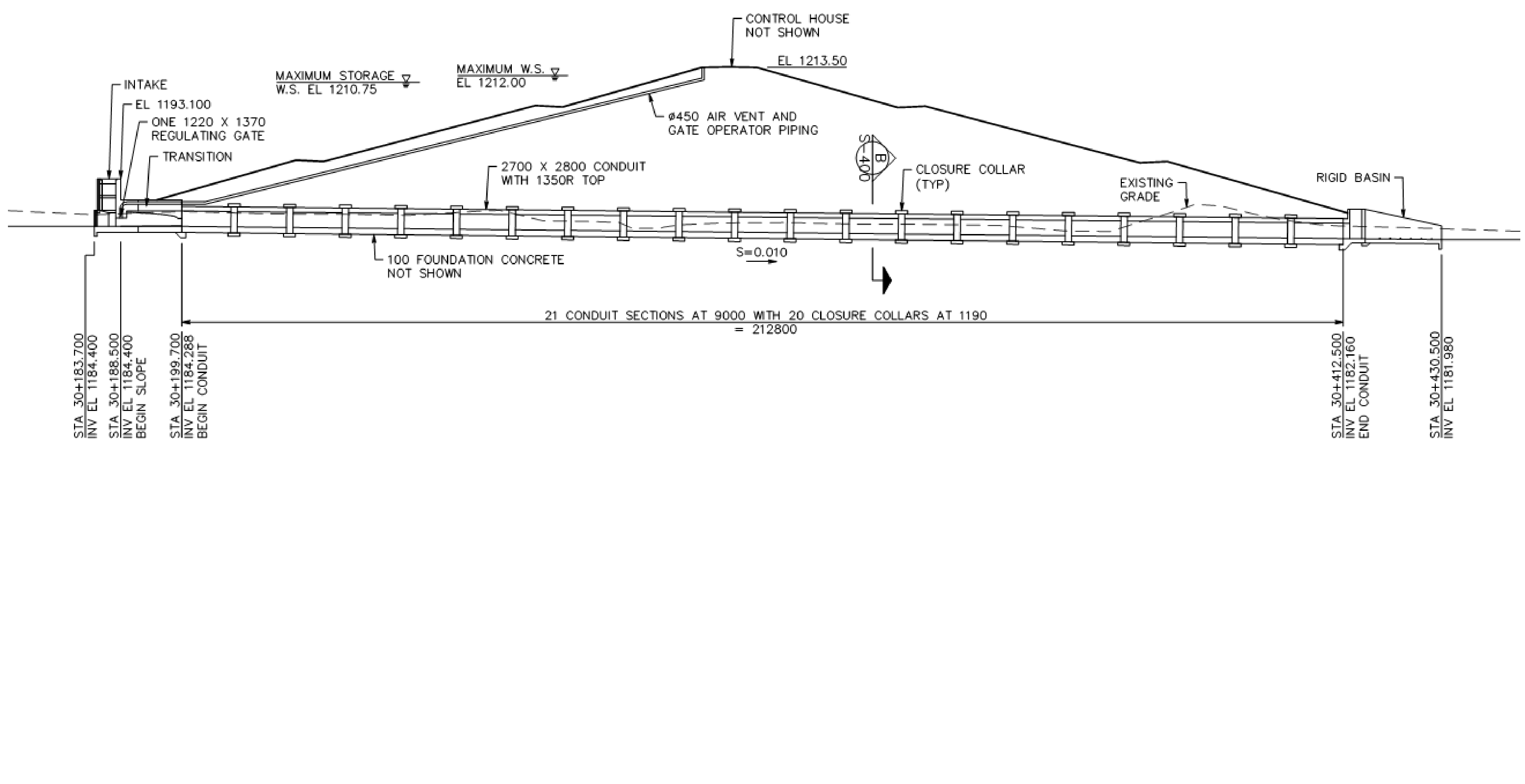
Figure IR475-1 Preliminary Rating Curve for Low Level Outlet Works

b. The LLOW is a gated gravity drainage structure. Primary elements of the LLOW are:

- A reinforced concrete Intake structure incorporates eight 2,500 mm by 2,500 mm trash rack panels and a 1,200 mm wide by 1,500 mm high sluice gate with hydraulic operator that can continue operating when submerged.
- A reinforced concrete conduit will provide the hydraulic transition from the intake structure gate opening to the main conduit section over a length of 10.5 m.
- A reinforced concrete 2,700 mm wide by 2,800 mm high modified horseshoe shaped conduit with a length of 212.8 m set on a 0.010 slope runs through the embankment dam.
- A reinforced concrete, 18 m long, rigid stilling basin located at the downstream end of the conduit and downstream toe of the embankment will provide at-grade energy dissipation of flow releases.
- A designed channel will be excavated from the stilling basin to the unnamed creek.

c. A section through the LLOW is provided in Figure IR475-2.

Backfill and seepage control measures will be developed and refined as design progresses.



Sources: Base Data - Government of Alberta, Government of Canada. Thematic Data - Stantec Ltd.

Disclaimer: This map is for illustrative purposes to support this Stantec project; questions can be directed to the issuing agency.

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Cross Section of the Low Level Outlet Works - March 2017

Figure IR475-2

d. The designed channel—a trapezoidal shape with a 5.4 m bottom width (to match the width of the stilling basin), 3H:1V side slopes, and a 0.5% bed slope— will be graded downstream of the outlet structure for a length of approximately 70 m to limit erosion and scour potential at the stilling basin and near the toe of the embankment for the outlet structure peak discharge. After 70 m, the bank height gradually transitions into the natural topography where flows are conveyed to the unnamed creek.

   The designed channel will be lined with riprap to control erosion. At the terminus of the channel, a riprap toe will extend to the depth of maximum scour to prevent progression of erosion upstream.

Question 476

Volume 1, Section 3.2.6, Page 3.18

Alberta Transportation states that *The gate system controls are situated in a lit gatehouse at the inlet to the low-level outlet*. The gatehouse is located on the dam crest which is offset from the submerged inlet structure. The method of operating the gate is not discussed in this section of the EIA. However, it is understood from the preliminary engineering design report that the gates are operated by hydraulic lines buried on the upstream slope between the inlet and gatehouse. As well, a vent pipe is buried in the dam embankment on the upstream slope. The entire low level outlet conduit is downstream of the gate and will be unpressurized when the gate is closed and the reservoir is full or partly full.

a. Provide details regarding the gate operation for the low level outlet.

b. What protective measures will be provided for the hydraulic lines buried on the slope?

c. What type of hydraulic fluid will be used in the event there is a leak or rupture?

d. Can the vent pipe and hydraulic lines accommodate settlement due to the embankment?

e. Describe if any backup measures or redundancies were incorporated should the gate be inoperable due to problems with the hydraulic system or the gate itself?

f. What are the backfill requirements for the section of the low level outlet conduit upstream of the impervious core? In particular, how will the joints be protected against the infiltration of backfill material due to the seepage gradient towards the unpressurized conduit during the period when the reservoir is inundated and the gate is closed?

g. What other gate arrangements were considered and explain why the proposed upstream gate arrangement is considered preferable?

Response 476

a. The outlet structure gate will be open during dry operations to allow flow (groundwater, precipitation) to pass through to the unnamed creek.

   Prior to operation of the diversion structure, the gate will be closed to impound water within the off-stream reservoir. The gate will remain closed during flood operations.

   Once Elbow River and the infrastructure downstream (such as Glenmore Dam, bridges) are ready to receive additional flows, the outlet structure gate will be opened to release flow from the off-stream reservoir back into the Elbow River by means of the low-level outlet works and the unnamed creek.

b. The hydraulic lines will be placed within a protective conduit and buried at sufficient depth (1,500 mm) to be protected from erosion.

c. A biodegradable, non-toxic hydraulic fluid will be used within the system; however, the exact fluid has not been specified at this time.

d. The vent and hydraulic lines will be designed to accommodate settlement of the embankment through use of flexible pipe, joints or fittings.

e. Provisions for backup power and redundant hydraulic systems will be provided at the control building. Addition of a second, “guard” gate is being considered and may be included in the design. If included in the final design, the second guard gate will be constructed within the current footprint of the dam embankment.

f. The conduit will be constructed at, or above, grade and constructed prior to embankment placement. Embankment material consistent with the zone in which the conduit is located will be placed against and above the conduit to required specifications (Alberta Transportation 2006).

   The cast-in-place concrete conduit within the embankment will be protected against infiltration from retained flood water as well as water seepage from conduit flowing into the fill using several methods. The primary means for seepage mitigation is installation of a waterstop embedded in the joint between the cast-in-place concrete conduits and providing continuous reinforcement across the joint to restrain joint movement. For conduits with a large amount of fill over the conduit, cast-in-place concrete collars with waterstops will surround the joint between conduits allowing small joint movement.

g. Four gate and control structure arrangements were considered:

1. intermediate control structure located at the crest of dam with upstream pressurized and downstream non-pressurized conduit
2. intermediate control structure located upstream of dam crest with upstream pressurized and downstream non-pressurized conduit so as to minimize penetration of core zone
3. upstream control tower with access bridge and non-pressurized conduit downstream
4. upstream control structure with mechanical gate operator that can be submerged and nonpressurized conduit downstream

Alternative 4 was selected for the following reasons:

- It is the least costly option of the four arrangements to construct because it eliminates the control tower.
- The conduit is uniformly constructed across the foundation and through the embankment with no break for an intermediate control tower. This reduces construction time for the conduit so that embankment placement can begin and reduces risks during construction.
- There is easy access for personnel and their equipment to the regulating gate from the reservoir for inspection and servicing.
- The intake structure is located outside of the core zone of the embankment dam, thus reducing potential seepage issues, and allowing differential settlement on a flexible soil foundation and allowing construction sequencing of conduit and intermediate tower with the embankment construction.
- The conduit remains non-pressurized during an impoundment event, which reduces the risk of conduit leakage into the embankment material.

Based on geotechnical exploration, updates to the outlet structure are currently underway. These updates may result in a change to the gate and control structure configuration as design advances.

REFERENCES

Alberta Transportation. 2006. CWMS, Section 002331, Fill Placement. Accessed at: https://www.transportation.alberta.ca/Content/docType125/Production/02331Fill_Placement.doc

Question 477

Volume 1, Section 2.2.6.1, Page 2.27

Alberta Transportation states that *Design Option 1 raises Highway 22 above the reservoir design flood level in the location of the future southbound lanes (twinning to the west side). The design elevation allows 0.5 m for freeboard and 1.0 m for the pavement structure depth above design flood level, which results in an embankment height of approximately 5 m at the Springbank Road intersection*.

a. Clarify which design flood level is referred to for the design crest level of the road.

b. Is the design crest elevation of the road a minimum of 1.5 m above the design flood level or 0.5 m or 1.0 m above the design flood level? It is not clear from the above wording. What are the actual elevations of the flood level and design crest level of the roadway?

c. Will the road be inundated during a PMF?

d. For the proposed culvert size, what is the head differential to maintain the required flow during filling and draining?

e. How are the exterior slopes and any buried utilities in the road embankments protected against wave attack? Clarify whether there is armour or riprap protection provided?

Response 477

a. The design flood level identified in the referenced section is consistent with the full service level of the reservoir (El. 1,210.75 m).

b. The design crest elevation varies along the road profile to accommodate road design grades. The minimum crest elevation is 1,212.75 m, which is 2 m above the design flood level. The maximum crest elevation is 1,230.75 m, which is 20 m above the design flood level.

c. The diversion inlet gates will be closed when the reservoir reaches its full service level. Should the gates fail to close during the probable maximum flood (PMF) event, the emergency spillway is designed to limit the water surface elevation within the reservoir to 1,212.0 m. Given the minimum crest elevation of 1,212.75 m, the roadway would not be inundated during a PMF, even with gate failure.

d. The culverts were designed to limit head differential across the embankment to 1 m during filling and draining.

e. The exterior slopes of the road embankments, including buried utilities, will be protected by vegetation against wave wash. No additional armour or riprap protection is provided. Further details on the short-term potential of wave wash are discussed in the response to IR470.

Question 478

Volume 1, Section 2.2.6.2, Page 2.30

Alberta Transportation states that *The road embankment would be classified as a dam under the Dam and Canal Safety Guidelines (Alberta Environmental Protection 1999), leading to higher engineering, construction, safety, maintenance, and licensing costs than for a typical roadway.*

a. Clarify on what basis the embankment would be classified as a dam.

b. Does the embankment act as a barrier to flow or is there a culvert?

c. Would the preferred option still be Option 2 for Springbank Road if the road embankment is not classified as a dam?

d. If Alberta Transportation considers Springbank Road as a dam, then is the Highway 22 embankment also considered a dam? If not, what are the differences?

Response 478

a. During operation of the reservoir for the design flood, the roadway embankment would temporarily maintain a hydraulic height exceeding 2.5 m and store more than 30,000 m^3, which meets the Province of Alberta's definition of a dam.

b. Three existing culverts connect the area north of Springbank Road and east of Highway 22 to the reservoir area to the south. These culverts provide local drainage, but they are not of sufficient size to pass the needed design flows to maintain a balanced head across the embankment.

c. Yes, the preferred option would still be Option 2, if it is not classified as a dam. The design concerns associated with retaining water with a large hydraulic head would dictate the same level of construction quality and care, with or without the classification.

d. The Highway 22 area is at higher elevation with a smaller volume behind it to impound water. The existing 3.0 m culvert is of sufficient size to maintain a hydraulic head differential across the embankment of less than 1 m of water. Because the roadway will not retain water and does not cause an imbalance of head across the embankment, the Highway 22 embankment is not considered a dam.

Question 479

Volume 1, Section 3.2.8, Page 3.20

Regarding utilities:

a. Discuss utilities that will be required for the Project components including their alignment (e.g. power for operation of the gates, etc.).

Response 479

a. The diversion structure requires three-phase electricity service. Coordination with the utility providers is currently in process. Electric service will likely be provided by Fortis from existing power lines located along Highway 22 with the service line routed along the access road adjacent to the diversion +channel. In addition, natural gas service is being considered for the backup power generators. Existing gas service is available along Township Road 242. Gas service would be extended from the intersection of Township Road 242 and the diversion channel access road and routed along the access road to the diversion structure control building.

   Permanent water service and sanitary sewer service are not proposed. These will be provided through temporary facilities.

   The outlet structure requires singe-phase electricity service. Electric service will likely be provided by Fortis from existing power lines located along Springbank Road.

Question 480

Volume 1, Section 3.2.8.1, Figure 3-12, Page 3.21

Figure 3-12 shows several pipelines crossing the reservoir. Alberta Transportation states that retrofitting including weighting of the pipe for flotation will be carried out for pipelines crossing the reservoir.

a. What provisions are included for erosion protection for the pipelines on the valley slopes in reservoir areas exposed to wave action during inundation?

Response 480

a. Pipelines currently located within the off-stream reservoir will be abandoned (and removed), or relocated, or retrofitted (if they remain in areas that could potentially be flooded). The response to IR533 provides more details about how third party pipelines, currently in the PDA, will be managed.

   Because wave action is not anticipated to pose a risk to these utilities, vegetation and soil cover are sufficient protection from erosion on the slopes in the off-stream reservoir. Additional details on wave action during inundation is provided in the response to IR470.

Question 481

Volume 1, Section 3.3, Page 3.22

Alberta Transportation states that *On the north side of Elbow River, the main access to the PDA will be a gravel road on the southeast side of the diversion channel, with gated approaches on both sides of Highway 22*. The diversion canal excavation slopes are up to 37 m deep on the northwest side of the canal channel and over 20 m on the southeast side. Access roads for the diversion canal are situated at the top of the cut slopes and there is no access berm at lower elevations on either side.

a. How will maintenance activities including possible clean out of slump debris occur in the diversion canal during operation with no access route for equipment at lower elevations?

Response 481

a. Provision for cleanout during flood operations at lower elevations is not provided at these deeper sections, because substantial freeboard is available within the diversion channel to overcome increases in water surface elevations associated with potential “slumps” and blockages. At locations where freeboard is reduced and areas around structures, such as bridges and spillways, a long-reach excavator may be used to clear debris and blockages. Access to the diversion channel is provided at numerous ramps along the length of the channel for maintenance.

Question 482

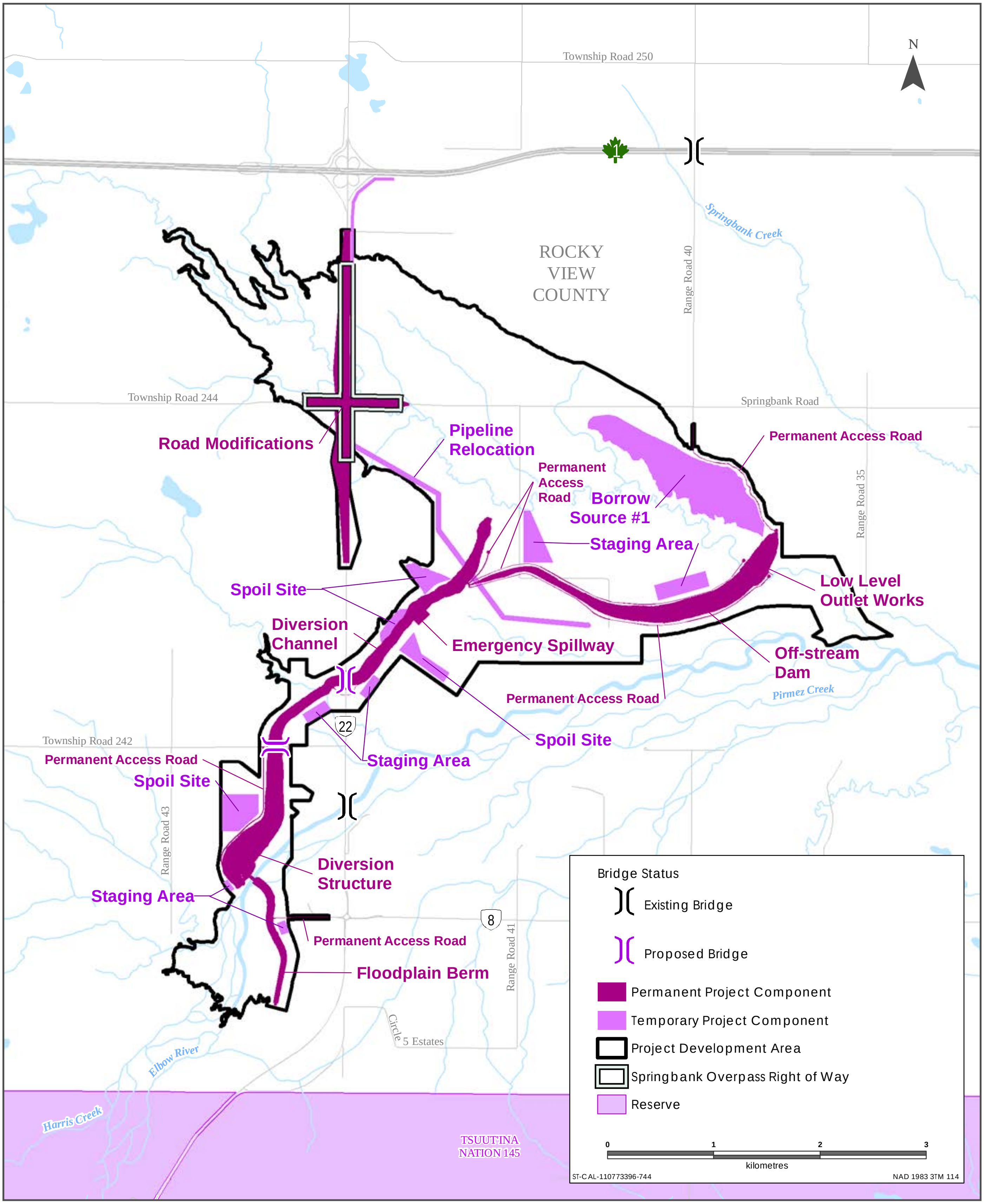
Volume 1, Section 3.3, Figure 3-1
Volume 1, Section 3.3, Page 3.22

Alberta Transportation states *See Figure 3-1 for the locations of temporary construction laydown areas, which typically include a site trailer, toilet facilities, and areas for parking, fueling, waste and recycling bins, and storage of equipment and materials*. Temporary construction laydown areas are not shown on Figure 3-1.

a. Provide details for locations of the temporary construction laydown areas. Update the EIA to reflect the proper locations.

Response 482

a. The reference to Figure 3-1 is incorrect. The reference should have stated Volume 4, Appendix D, Figure 2-2. Figure IR482-1 (a duplicate of Figure 2-2) illustrates the temporary construction laydown areas (staging areas).



Sources: Base Data - ESRI, Natural Earth, Government of Alberta, Government of Canada
Thematic Data - ERBC, Government of Alberta, Stantec Ltd

Permanent and Temporary Features of the Project

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ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESERVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Figure IR482-1

Question 483

Volume 1, Section 3.3.1.1, Page 3.23

Alberta Transportation states that *The direct material haul to the floodplain berm construction site from the diversion channel will require installation of a temporary bridge across the diversion channel.*

a. Clarify if the temporary bridge is across the diversion channel or the Elbow River.

b. Provide details showing the location of the temporary bridge crossing and the proposed access route on the valley slope of the Elbow River.

Response 483

a. A temporary bridge, if one is used, will be constructed across Elbow River, as determined by the construction contractor. If a temporary bridge is not needed, construction materials for the floodplain berm would be hauled to the site using the existing roadway network, including Township Road 242 and Highway 22.

b. Details regarding the location of a temporary bridge crossing will be determined by the construction contractor and are not available at the time of this submission. The bridge would be constructed across or upstream of the service spillway. Access to the bridge would be through the diversion channel excavation. A proposed access route on the valley slope of Elbow River is not planned.

Question 484

Volume 1, Section 3.3.1.2, Page 3.23
Volume 1, Section 3.2.5.3, Table 3-7, Page 3.31

Table 3-7 indicates that the schedule for the floodplain berm extends from September to November 2019 followed by two additional periods in May to June and August of 2021. Section 3.3.1.2 does not include any explanation for the staged construction of the floodplain berm nor does it describe the interim condition of the embankment after the 2019 construction season.

a. What is the condition of the floodplain berm after the 2019 construction season (i.e. what portions are completed)?

b. What components of the floodplain berm are constructed during each of the construction stages shown on the schedule?

c. **Is part of the floodplain berm construction intended to be for the auxiliary spillway? Identify which parts of the schedule apply to the auxiliary spillway.**

d. **Clarify if the floodplain berm construction between September and November 2019 includes the RCC auxiliary spillway? If so, explain how cold weather placement of the RCC will be carried out.**

e. **What temporary works are required for protection of the auxiliary spillway foundation excavation during operation of the service spillway? Explain how the auxiliary spillway excavation will be dewatered in close proximity to the service spillway during periods of river flow in the service spillway even if the left bay of the spillway takes most of the normal flow.**

Response 484

a. Subject to regulatory approvals, the first construction season will complete the majority of the floodplain berm earth embankment, including riprap placement. While the floodplain berm may be complete, the system will not be functional until completion of the auxiliary spillway and service spillway.

b-c. The schedule includes the roller compacted concrete (RCC) auxiliary spillway as part of the floodplain berm line item in the schedule (Volume 1, Section 3.2.5.3, Table 3-7). The construction of the auxiliary spillway will occur in the second construction season following completion of the service spillway.

d. Construction of the floodplain berm does not include the RCC auxiliary spillway.

e. Temporary berms or sheeting will be required to isolate the auxiliary spillway construction from flow in the river during construction. Prior to excavation of the auxiliary spillway foundation, the service spillway will be completed, including the right abutment transition. This concrete structure will be constructed on bedrock and will form the tie-in point for the required cofferdam. The coffer dam would be constructed starting at the abutment and then tying into high ground or the floodplain berm on the south extent of the auxiliary spillway. Specific means of isolation and dewatering will be designed by the contractor, but all construction work will be done in the dry and in compliance with Civil Works Master Specifications, Section 01390 (Volume 4, Supporting Documentation, Document 10) and Section 002240 (Volume 4, Supporting Documentation, Document 12) and all applicable environmental and water quality permit conditions.

Question 485

Volume 1, Section 3.3.1.3, Figure 3-1,
Volume 1, Sections 3.3.1.3, 3.3.1.4, 3.3.1.5, Page 3.24
Volume 4, Appendix D, Figure 2-2., Page 2.7

Alberta Transportation states that *Rock or soil materials that are unsuitable for construction will be left as spoil near the diversion structure (see Figure 3-1)*. Locations of spoil piles are not shown on Figure 3-1 but are shown on Volume 4, Appendix D, Figure 2-2.

a. Provide layout details for areas of disturbance at spoil pile locations on Figure 3-1.

b. Provide details related to spoil pile volumes, heights, side slopes, compaction requirements and drainage details.

Response 485

a. See the response to IR482, Figure 482-1.

b. The layout and details related to the spoil areas will be selected for potential use by the construction contractor, if needed. Actual volumes to be placed will be determined during construction and will be based on the quality and condition of excavated materials and the timing of material excavation with placement needs.

   Spoil Area 1 is located west of the diversion channel and south of Township Road 242. The maximum limit of Spoil Area 1 has an approximate area of141,400 m^2 and an approximate volume of 1.1 million m^3. The volume is based on a maximum height of 30 m with side slopes of 5:1. Drainage from the spoil pile will be routed at a 2% slope to the southeast quadrant of the pile. Drainage will be rerouted along the spoil pile through a series of ditches that approximately match existing drainage paths and into the diversion channel.

   Spoil Area 2 is located east of the diversion channel and east of Highway 22. The maximum limit of Spoil Area 2 has an approximate area of 78,600 m^2 and an approximate volume of 0.4 million m^3. The volume is based on a maximum height of 15 m with side slopes of 5:1. Drainage from the spoil pile will be routed at a 2.5% slope to the northeast quadrant of the pile. Drainage will be rerouted along the spoil area through a series of ditches that approximately match existing drainage paths and towards the emergency spillway channel.

   Waste fill placement including compaction requirements will be done in accordance with Alberta Transportation (2017).

REFERENCES

Albert Transportation. 2017. Waste Fill Placement. Accessed at: https://www.transportation.alberta.ca/Content/docType125/Production/Section02332.pdf.

Question 486

Volume 1, Section 3.3.1.4, Figure 3-1, and Page 3.24

Alberta Transportation states that *A temporary laydown/stockpile area to support construction will be set up within the reservoir area, near the dam in a location accessible from the existing road network (Figure 3-1)*. Locations of the laydown/stockpile are not shown on Figure 3-1. This section also refers to borrow areas located in the reservoir area and shows their locations on Figure 3-1.

a. Provide layout details for areas of disturbance for the laydown/stockpile areas.

b. Provide details for the borrow areas approximate depths and side slopes and indicate whether they become part of the reservoir. Refer to Figure 3-1 where locations are shown.

Response 486

a. See the response to IR482, Figure 482-1.

b. Alberta Transportation initially identified two potential locations for borrow sources. Upon further review, Alberta Transportation may use only one borrow source should excavation of the diversion channel yield insufficient material for constructing Project components. Borrow Source 1 is located within the reservoir (see the response to IR482, Figure 482-1) and will become part of the reservoir. Borrow Source 1 has a combined disturbance area of 618,000 m^2 and maximum excavation depth of 13 m. Volume calculations assume a maximum side slope of 5:1, which results in an overall available borrow source volume of 1,600,000 m^3. Drainage for Borrow Source 1 will be routed at 1% cross drain slope to existing water courses, which will not result in a sump or permanent pool.

Question 487

Volume 1, Section 3.3.8, Table 3-7, Page 3.31
Volume 1, Section 5.1.1, Page 5.2
Volume 1, Attachment A, Section A.2.1.1, Page A.6

A river diversion with cofferdam embankment will be used to allow the diversion inlet and service spillway to be constructed *in the dry*. The river diversion extends through the RCC auxiliary spillway.

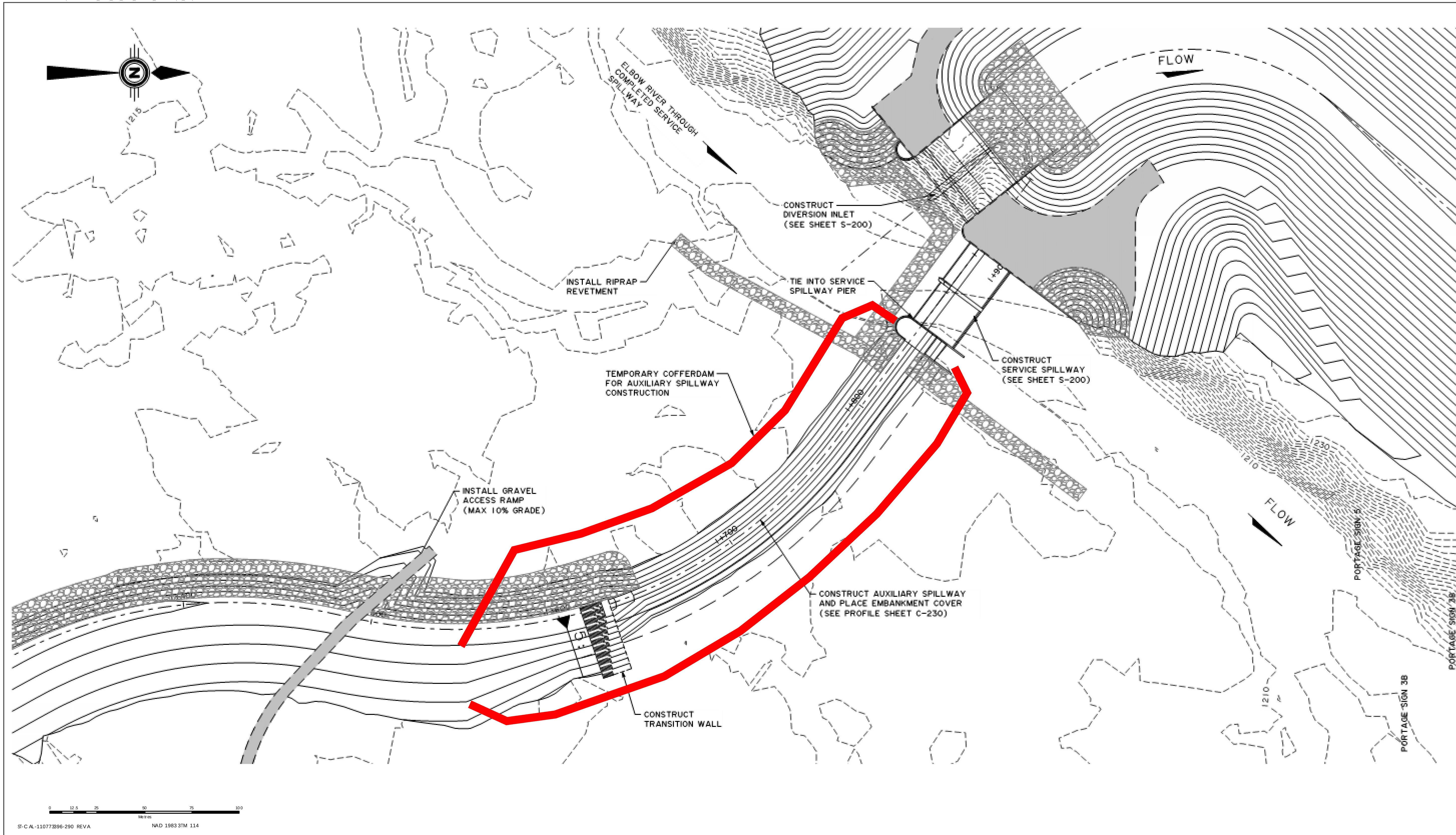
a. Explain the basis for selection of the 1:10 year flood event for the cofferdam.

b. Given that the auxiliary spillway abuts with the service spillway, clarify the water management requirements and construction staging for the auxiliary spillway considering instream construction windows.

Response 487

a. A 1:10 year flood was selected based on the relative low consequence of failure and short exposure duration. Damages associated with failure of the cofferdam are limited to the in-progress construction works, which are primarily cast-in-place concrete structures. The cofferdam will be in place for one flood season of Elbow River.

b. Construction of the service spillway will include the right abutment pier. Following construction of the service spillway, the river channel will be re-diverted through the service spillway and the isolation works for the auxiliary spillway will tie-in with the right abutment pier, which is constructed to an elevation higher than the 1:10 year flood water surface elevation. Work within the water course is anticipated to occur within the approved instream construction windows. Figure IR487-1 provides the proposed location of cofferdams for the auxiliary spillway.



Sources: Base Data- Government of Alberta, Government of Canada. Thematic Data - Stantec Ltd.

Disclaimer: This map is for illustrative purposes to support this Stantec project; questions can be directed to the issuing agency.

Stantec

ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESEVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Diversion Structure Overview - March 2017

Figure IR487-1

Question 488

Volume 1, Section 3.3.6.1, Page 3.29

Alberta Transportation states that *The integrity of the dam will be tested during the construction phase*.

a. Clarify this statement. What testing will be carried out during the construction phase to ensure the integrity of the dam?

b. What testing of backup power supply systems (e.g. standby diesel generators) will be provided if backup is supplied?

Response 488

a. The referenced statement refers to construction quality control testing and geotechnical instrumentation monitoring to be performed during construction. This will include moisture-density testing, permeability, settlement plates, pore-pressure, and inclinometers.

b. Testing of backup power supply systems will be conducted along with testing of the primary systems during system commissioning and annual maintenance–operations inspections.

Question 489

Volume 1, Section 3.4.1, Table 3-10, Page 3.33

Following flood deposition, removal of debris and sediment will be required within the river. The connection points and bladders for the Obermeyer gates are located in the river and will likely be submerged.

a. What are the regulatory approval requirements, timelines and the implications on completing removal of debris and sediment in the river?

b. Explain how inspections of the gate connections and bladders will be conducted?

c. What are the regulatory approval requirements, timelines and implications on carrying out repairs to the gates when required?

Response 489

a. Volume 1, Section 3, Table 3-10 specifically addresses removal of sediment and debris from the service spillway and debris deflector. Removal of sediment and debris from the upstream Elbow River channel is considered separately.

   Post-flood sediment and debris removal from the river bed upstream of the diversion structure will only occur if the sediment or debris is affecting the serviceability of the Project, or if it impedes navigation, fish passage or aquatic connectivity. If required, this activity is expected to occur in the post-flood period. Debris and sediment may be removed in-the-dry by work completed from the banks above the flow, or by diverting flow, or by isolating work areas with coffer dams. This activity will require planning and may require regulatory approval.

   Prior to removal, an evaluation will be done to determine the areas from which the material needs to be removed. AEP would then seek approvals under the *Water Act*, *Fisheries Act* and a Provincial Fish Research License, as required, and dependent upon the nature of the removal. The duration of this activity is expected to take up to one week for a heavy sediment and debris load. However, planning and approval requirements may take longer than the actual activity and only completed, if required, following operation for a flood that would generate sufficient amount of sediment and debris to require cleanup.

   The service spillway is designed to reduce the ability of sediment and debris to accumulate and reduce the risk of such accumulations affecting operation. The debris deflector prevents woody debris from entering the diversion channel during operation. Should it be determined that sediment and debris in the service spillway and the debris deflector have the potential to affect the operation of the Project, removal must be done immediately following a flood to make the Project operational again. This activity would be expected to take a few days to complete and AEP regulatory approval to do so is being applied for and will be included as part of the facility's license to operate.

b. In general, the exposed portions of an Obermeyer gate consist of a steel leaf protecting a pneumatic bladder, both secured continuously along one edge with a clamping plate to the spillway floor concrete. Air connections to the bladder and sensor connections are made through conduits embedded in the spillway concrete. They connect to the bladder just downstream of the clamping plate embedment and where the bladder rests permanently on the concrete surface.

   Following the diversion of a flood, the Obermeyer gates would typically be in the open (lowered) position with the leaf resting on the spillway and protecting the collapsed bladder beneath it. River flow would be passing over one or both of the lowered gates, depending upon sedimentation and debris conditions.

Two side-by-side 24 m-long Obermeyer gates are available so that one could be inspected while the other one is passing river flow. The gate and its actuator are designed to lift sediment, boulders, and debris up to one metre thick that rest on top of the lowered gate leaf following flood passage. However, it is not expected that sediment and debris will build up in the lowered crest gate leaves, based on the hydraulic model study results. These results indicated little sediment buildup in the gate leaves themselves as the flow is accelerated through the narrowed spillway channel over the leaves.

Obermeyer gate inspection is accomplished by raising one gate and visually inspecting both downstream and upstream sides of the gate leaf and bladder. Personnel can perform this inspection using chest waders. Removal of sediment needed for inspection will use a pump and fire hose.

Should the water depth be too great for upstream inspection of the gate leaf, a small-height cofferdam (less than 3 m) could be placed upstream of a single spillway gate bay to isolate the gate. The upstream temporary cofferdam could consist of commercially available systems such as Port-a-Dam (liner supported by A-frames), water dams (water-filled bladders), or “Super Sacks” consisting of one cubic meter bags filled with cohesionless material and stacked like sand bags. Once the temporary cofferdam is installed, a fish rescue would be conducted behind the cofferdam prior to dewatering. Once Isolated from Elbow River, the Obermeyer gate can be inspected or serviced in a dry environment.

After one gate has been inspected and serviced, it is lowered to allow passage of river flow and the other gate is raised for inspection. At no time is the river flow interrupted when passing through the service spillway for inspection or servicing.

c. Inspection and maintenance of the gates and related infrastructure within the footprint of the service spillway will be covered under the facility’s operating licence. This would include the provision to cofferdam and rescue fish from behind a cofferdam, when required. Approval for these activities in the licence would also apply to post-flood activities. Inspection and repair are necessary immediately following a flood; these activities be completed in a timely manner so that the Project is ready to operate. For this reason, post-flood activities within the service spillway, including the post-flood removal of debris and sediment, will be part of the Project’s approval.

Question 490

Volume 1, Section 1.2.1, Page 1.6
Volume 1, Section 1.2.1, Figure 1-5, Page 1.7
Volume 1, Section 3.3.8, Table 3-7, Page 3.31

Alberta Transportation states that *At present, the Project is scheduled to be functionally operational (able to accommodate a 1:100-year flood event) for floods in the spring of 2021, and be completely constructed (able to accommodate the design flood) for the spring of 2022*. However, the schedule on Table 3-7 on Page 3.31 shows several unfinished items of work in the spring of 2021 including completion of the diversion channel excavation, offstream dam embankment and floodplain berm which are all scheduled for the summer of 2021.

a. Describe the extent of unfinished work in the spring of 2021 at which time it is stated that a 1:100 year flood can be accommodated?

b. Discuss the flood handling procedures during the construction period prior to all of the works being completed by the end of 2021?

c. Clarify the extent of temporary/permanent power and control systems that will be provided to operate the facilities in the spring of 2021.

d. Explain how flow would be diverted to the offstream storage reservoir in the spring of 2021 if the diversion canal is not complete?

e. Explain how the low level outlet gate will be operated if the offstream dam is only partly completed and the hydraulic controls not yet installed and functional?

f. Explain the potential dam safety challenges/issues related to the project while partly completed in 2021 and identify the proposed mitigation measures.

Response 490

a. By second quarter of Year 3 after start of construction, the following items will be partially constructed to facilitate an interim protection level of a 1:100 year flood:

- The off-stream dam will be constructed for the full cross section to El. 1,204.5 m. The final 9 m of embankment will not be completed.
- The control building at the crest of the dam will not be completed and the outlet works gate will not have controls. The gate may be installed but would remain open.

- The diversion channel will be excavated to its final invert elevation for the full length with a minimum 6 m bottom width with 3H:1V side slopes. The channel would operate at depths comparable to the final configuration (approximately 6 m). This geometry provides sufficient capacity to convey the peak diversion flow (360 m^3/s) for the 1:100 year flood. Riprap will be placed at the design locations to protect the bridges, utilities and constructed embankments.

The following Project components will be completed and commissioned by the end of the second quarter of Year 3:

- diversion structure including floodplain berm, auxiliary spillway, service spillway and diversion inlet
- embankments on low side of the diversion channel
- emergency spillway (or temporary dike to contain channel flows)
- diversion channel structure including flare from 6 m width to the full 120 m width
- temporary emergency spillway located at the interim pool level and cut through earth
- outlet structure including inlet and outlet works and designed channel from the outlet

b. At the completion of Year 2 of construction, the diversion structure including the diversion inlet, service spillway and control building will be commissioned and handed over to AEP Operations for management of water diversion.

Construction of the diversion channel and dam embankment will continue through the summer of Year 3, when the facility is ready for flood protection. Alberta Transportation and its contractor will coordinate with Alberta Environment and Parks Operations group for flood forecasting and operations.

During the flood season (May through July), the contractor will not be permitted to locate staging areas, fuel storage, temporary offices and equipment laydown within the off-stream reservoir. If a potential flood producing rain event is forecast, the contractor will prepare the diversion channel and off-stream reservoir to receive flow. Sharp grade transitions and at-risk work will be protected with riprap. Construction equipment and materials will be removed from the channel and reservoir. Rock and soil materials will be staged for potential repairs during flood operations.

The Project will be operated, as necessary, up to a maximum diversion rate of 360 m^3/s. The maximum pool elevation in the reservoir for the interim condition is El. 1,201.5 m, which is 3 m below the maximum water elevation.

c. All permanent and backup power and control systems for the diversion structure will be fully installed and commissioned prior to flood season in the second quarter of Year 3. This will include primary and backup power to the control building and gate systems for the diversion inlet and service spillway. Power and control systems for the outlet structure works will not be installed.

d. The diversion channel will be excavated to its final invert elevation for the full length with a minimum 6 m bottom width with 3H:1V side slopes. The channel would operate at depths comparable to the final configuration (approximately 6 m). This geometry provides sufficient capacity to convey the peak diversion flow (360 m^3/s) for the 1:100 year flood. Riprap will be placed at the design locations to protect the bridges, utilities and constructed embankments.

e. The outlet structure works inlet is designed to provide flow control based on the concrete structure geometry. The flow rate capacity of the outlet works at interim El. 1,201.5 m in the reservoir is 21.6 m^3/s.

f. The interim condition of the Project will meet all required dam safety criteria prior to flood operation. The dam safety issues/challenges associated with the final condition will apply to the interim condition. Where vegetation has not been established along the diversion channel and within the reservoir, regular inspections and repairs will be performed throughout operation.

Question 491

<u>Volume 1, Section 5.0, Page 5.2</u>

Alberta Transportation indicated a hazard classification (under CDA guidelines) of *Extreme* for the offstream storage dam and *High* for the floodplain diversion berm. A rationale for the classification of the offstream storage dam has been provided, but there is no equivalent discussion for the floodplain diversion berm. Alberta Transportation also states that *A dam breach inundation study was completed for the off-stream dam and is discussed in Section 5.4*.

a. Provide the rationale for the consequence classification of the floodplain diversion berm, considering that the effect of the berm failure is not only a breach flood wave but also a potential reduction in the flood mitigation effectiveness of the entire SR1 system if the breach reduces the capacity of the diversion.

b. Similarly provide the consequence classification and rationale for other project components including the diversion channel.

Response 491

a. From Volume 1, Section 5.0, Page 5.2, failure of the diversion structure auxiliary spillway during the inflow design flood (IDF) will have minimal impact downstream of the structure. According to breach routing results, such a failure would increase the peak discharge in Elbow River immediately downstream of the diversion structure from 2,770 m^3/s to a peak of 3,103 m^3/s for less than 30 minutes. The spike in flow corresponds to approximately a 0.2 m increase in the water surface elevation. At the Highway 22 bridge, which is located approximately 1 km downstream of the diversion structure, the increase in water surface elevation due to the breach is less than 0.1 m. Based on these results, the model shows negligible change to inundation limits.

   Thus, failure of the diversion structure, including the floodplain diversion berm, during a flood would produce minimal increases in discharge and water surface elevation downstream. However, the breach wave caused by a failure of the diversion structure may carry concentrated debris that could damage the Highway 22 bridge, which is located a short distance downstream. Based on the potential for economic losses associated with the bridge and the diversion structure itself, a dam class of “High” consequence is justified.

b. Classification of the Project components are grouped, based on their related systems. The service spillway, auxiliary spillway and floodplain berm are grouped into the diversion structure system and collectively classified as “High” consequence. The diversion inlet, diversion channel, emergency spillway, off-stream dam, low-level outlet works and designed channel are part of the off-stream dam system and are collectively classified as “Extreme” consequence. All components within the system are designed in accordance with the referenced guidelines and in compliance with the required load cases and factors of safety for the corresponding hazard classification.

Question 492

<u>**Volume 1, Section 5.1.2, Tables 5-2, Page 5.3**</u>
<u>**Volume 1, Section 5.1.2 Table 5-3, Page 5.4**</u>

Tables 5-2 and 5-3 describe potential dam safety issues at the Diversion Inlet and the Service Spillway.

Table 5-2 indicates that the gates for the diversion inlet *were selected based on their mechanical reliability* while a similar statement is not provided on Table 5-3 for the service spillway gates.

a. Explain if potential issues that could affect the required operation of the diversion inlet (e.g. debris or sediment) and/or service spillway (e.g. debris or non-operable gates) to divert flood flows and thereby provide flood protection to the City of Calgary have been examined and what mitigation measures have been identified. If these potential issues have not been examined explain why not.

b. Describe the reliability of the service spillway gates relative to the diversion inlet gates. Provide information on the long-term performance of the type of gate selected for the service spillway in rivers with significant sediment transport. Explain their performance due to sediment accumulation on or against them.

c. Explain how future major maintenance, which requires removal of the diversion inlet or spillway gates, would be carried out (e.g. equipment, access, etc.).

Response 492

The diversion inlet and service spillway gate systems serve fundamentally different purposes for operation during flooding and as dam safety measures.

- The diversion inlet gates are guard gates and not intended for regulation of flow. They are closed during dry operations and fully opened for flood operation. The critical dam safety feature for the diversion inlet gates is their ability to close reliably and shut off flow to the off-stream reservoir if the reservoir is full or there is a potential dam safety issue.
- The service spillway gates are regulating gates that must reliably operate during a flood to control the upstream water surface elevation and achieve the desired diversion flow rates. The gate operators must be protected from sediment and debris flows. During a dam safety condition, such as instability of the floodplain berm/auxiliary spillway or failure of the diversion inlet gates to close properly, the gates must reliably "open" (i.e., lower) to pass flows downstream and reduce the water surface elevation upstream.

a. Potential issues that could affect required operation of the diversion inlet and service spillway to divert flood flows and provide flood protection have been considered; however, the referenced tables pertain specifically to issues for dam safety and directly relate to mitigation for safely passing or containing flood flows, so as not to risk the integrity of the floodplain berm, diversion channel, and dam.

   Specific mitigation measures for flood operation reliability <u>of the diversion inlet</u> are:

   - a diversion capacity 25% greater than that required to meet criteria for the design flood to address diversion capacity impacts associated with debris or sedimentations.
   - debris deflector to reduce effects of debris collecting on gates and gate intake

- sufficient gate bay width and height to pass debris that enters the gate bay
- backup power system

Specific mitigation measures for flood operation reliability <u>for the service spillway</u> are:

- sufficient bay width that allows for passage of debris
- selection of Obermeyer Hydro, Inc. pneumatic crest gates that:
  - allows for increased bay width
  - has no flow obstructions (such as hydraulic operating arms or superstructure) to catch debris
  - includes an air bladder on the underside of the gate leaf that prevents debris and sediment accumulation below or underneath the gate
  - provides a resilient system with each gate leaf operated by multiple bladders with independent air supply lines, such that failure of one bladder does not prevent operations
- flexible operations with two independent crest gates
- backup power system
- backup air supply system that can operate without power

b. As described in a., the diversion inlet and service spillway gate systems serve separate and distinct purposes for the Project and their relative reliability are not comparable.

According to information provided by Obermeyer Hydro, Inc. (OHI), OHI has been operating for over 25 years with over 300 installations of their pneumatically operated bottom hinged spillway gate. OHI gate installations have been successful in high bed load systems including mountainous environments such as British Columbia and Nepal. Successful operation of a similar system was observed by Alberta Transportation and design staff in western Wyoming at the Greybull Valley Irrigation District. That system is operated in a river and foothills environment similar to the Project.

A large amount of sediment deposition on the gates in the open position is not expected because the service spillway gate width is sized to hydraulically match the river at bankfull width, thereby maintaining the sediment transport capacity of the river at preflood stages. If sedimentation over the gate were to occur, the sediment could be “lifted” by the pneumatic bladder during operations or would be washed out during flows during the rising limb of the hydrograph. Sediment accumulation is not expected to impact the ability to meet the necessary flood diversion rates.

c. The diversion inlet gates invert is positioned at an elevation that is generally higher than the water surface elevation in the river during non-flood flow conditions. Access for maintenance may be achieved from the following routes and locations: Highway 22 or Highway 242 via the access road through the parking area and the access bridge across the gate structure; from the diversion channel; and from the river side of the structure. During flood operations, maintenance access must come from the access road, parking area and access bridge.

   For the service spillway, the gates are located within the active Elbow River channel. Maintenance activities will be carried out during the approved seasonal wildlife and fisheries windows. Maintenance of gates may require a temporary cofferdam (e.g., sandbags or water dam) to be installed upstream of an individual gate bay with the second bay serving as the river channel diversion bypass. Access to the service spillway will be achieved from Highway 8 using the Project access road and then along the service road atop the floodplain berm to the service spillway location on Elbow River. A crossing of the floodplain berm using access ramps is included in the design to allow access to the upstream or downstream side of the diversion structure. In addition, for removal of the gates, a mobile crane may be operated from the diversion structure parking area on the northwest side of the river.

Question 493

Volume 1, Section 5.1.2, Table 5-2, Page 5.3

Alberta Transportation states that *The gate hoists are planned as wire rope hoists. They will close by gravity, controlled by a braking system. Should the rope and/or brake system fail, the hoist brakes can be released and the gates lowered*. If the ropes or brakes fail, the gates will likely drop and may be damaged or rendered ineffective.

a. Clarify how the gates will be lowered without damaging the gate if the wire ropes fail.

Response 493

a. text in the second row, second column of Volume 1, Section 5, Table 5-2 is not correct. It should read as follows: "Reactive: The gate hoists are planned as wire rope hoists. They will close by gravity, controlled by a braking system. Should ~~the rope and/or brake system~~ the hoist motors fail, or power is lost, the hoist brakes can be released and the gates lowered.

   The wire rope hoists will be designed to hold the gate, even with the failure of several ropes. It is assumed that 12 wire ropes will be connected to the gate (6 wire ropes at each of the two drums). Additionally, a safety factor of approximately 5 is built into the safe working capacity of the wire ropes. The probability of simultaneous and complete failure of all 12 wire ropes is considered relatively low. Also, rope failure is typically progressive, such that individual rope strands start to fail and would be identified proactively with regular maintenance and inspection.

In an emergency failure mode, if all 12 wire rope hoists fail and the gates drop, some damage may occur to the gate and structure, but the primary objective of isolating the diversion channel from Elbow River flows will continue.

Question 494

Volume 1, Section 5.1.2, Table 5-2, Page 5.3

The consequences of one or more diversion gates failing to raise at the beginning of a high flow event have not been discussed.

a. If these gates cannot be raised, explain how the effectiveness of the diversion structure will be impacted and the corresponding consequences at the onset of a flood exceeding 160 m^3/s.

Response 494

a. The total capacity of the diversion inlet structure is 600 m^3/s at the maximum operating height of El. 1,215.8 m. If one gate fails to open, the diversion system would be reduced to 50% of total capacity, which would result in a maximum diversion capacity of 300 m^3/s. However, partial diversion of flood waters would continue.

   This reduced capacity is less than the required diversion capacity of 480 m^3/s for the June 2013 flood, but is greater than required for floods up to the 1:100 year flood.

   The dam safety systems, including spillway capacity, are designed to safely pass flows in the event of gate failure in the up or down position.

Question 495

Volume 1, Section 3.2.1, Figure 3-2, Page 3.4

Figure 3-2 shows a floating boom upstream of the diversion structure.

a. Describe the function and operation requirements for the boom.

Response 495

a. The floating boom has been removed from the design due to safety concerns. No floating booms are currently proposed upstream of the diversion structure. Figure IR495-1 is an update to Volume 1, Figure 3-2, which has the floating boom removed.

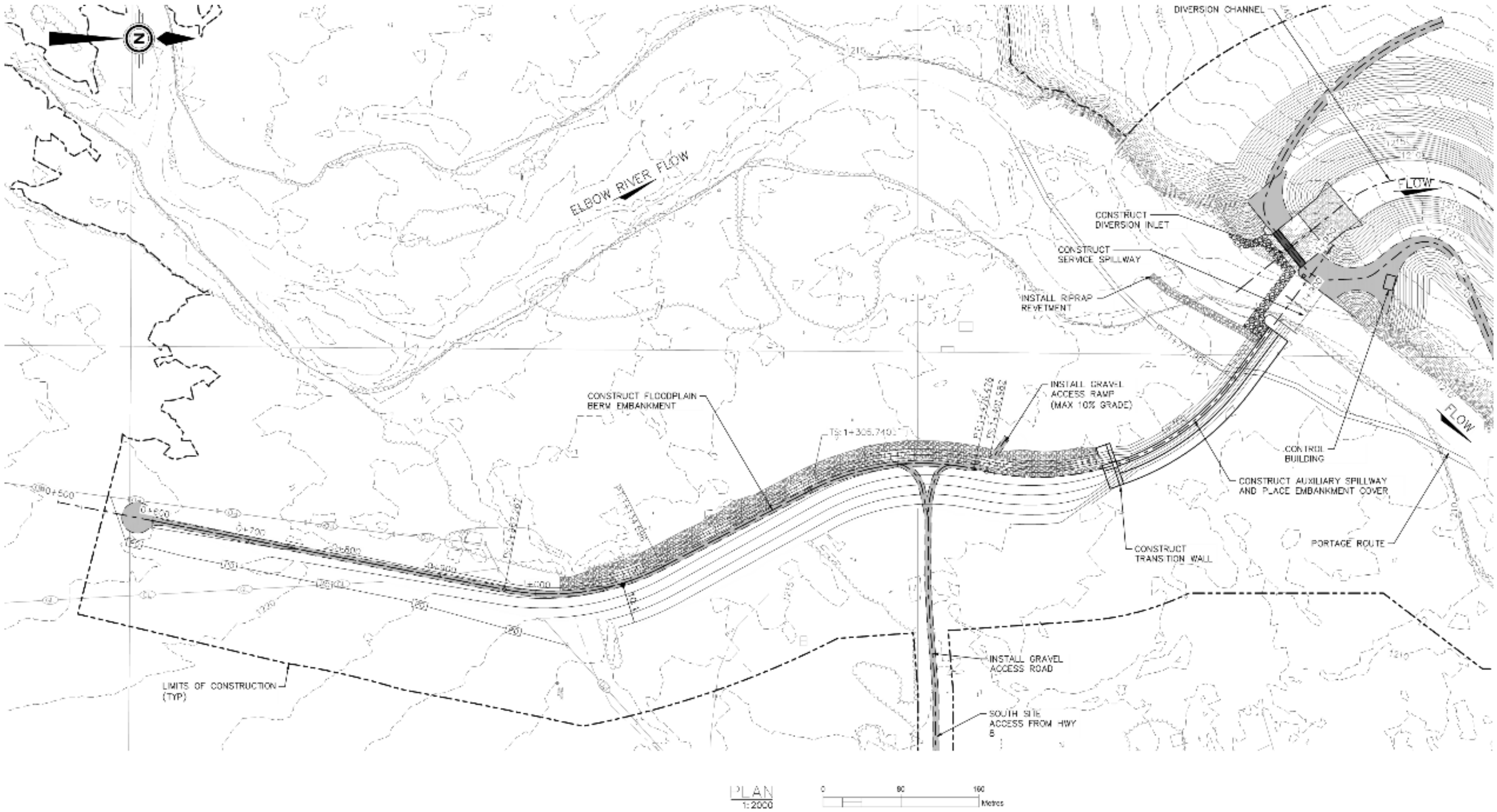


Figure IR495-1 The Diversion System, North End of Floodplain Berm

Question 496

<u>Volume 1, Section 5.1.2, Table 5-3, Page 5.4</u>

Alberta Transportation states that *The gates and pneumatic bladders* (of the Obermeyer gates) *are designed to fail open. That is, failure of the bladder would result in the gates opening and a reduced risk to dam safety*. If the gates fail in the open position during a flood and cannot be closed, less flow can be diverted to the reservoir, however, flows downstream of the service spillway will exceed the maximum permissible rate of 160 m^3/s to the Glenmore Reservoir.

a. What is the impact on diversion flows to the reservoir if one or both service spillway gates fail in the open position? What is the split of flow (i.e. service spillway versus diversion channel) if both structure gates are open for various flood magnitudes?

Response 496

a. To clarify, limiting flows into Glenmore Reservoir to 160 m^3/s is not a design criteria, as implied in the question. The design criteria is to limit flows *downstream* of Glenmore Reservoir to 160 m^3/s. The Project service spillway gates have continuously adjustable operating configurations. The flow splits can be calculated using the rating curves for a single bay presented in Figure IR496-1. For instance, at the design flow of 1,240 m^3/s, if the right bay were at El. 1,210 m (passing 540 m^3/s), the left bay could be set at El. 1,214.5 m (passing 100 m^3/s) and a diversion of 600 m^3/s would result.

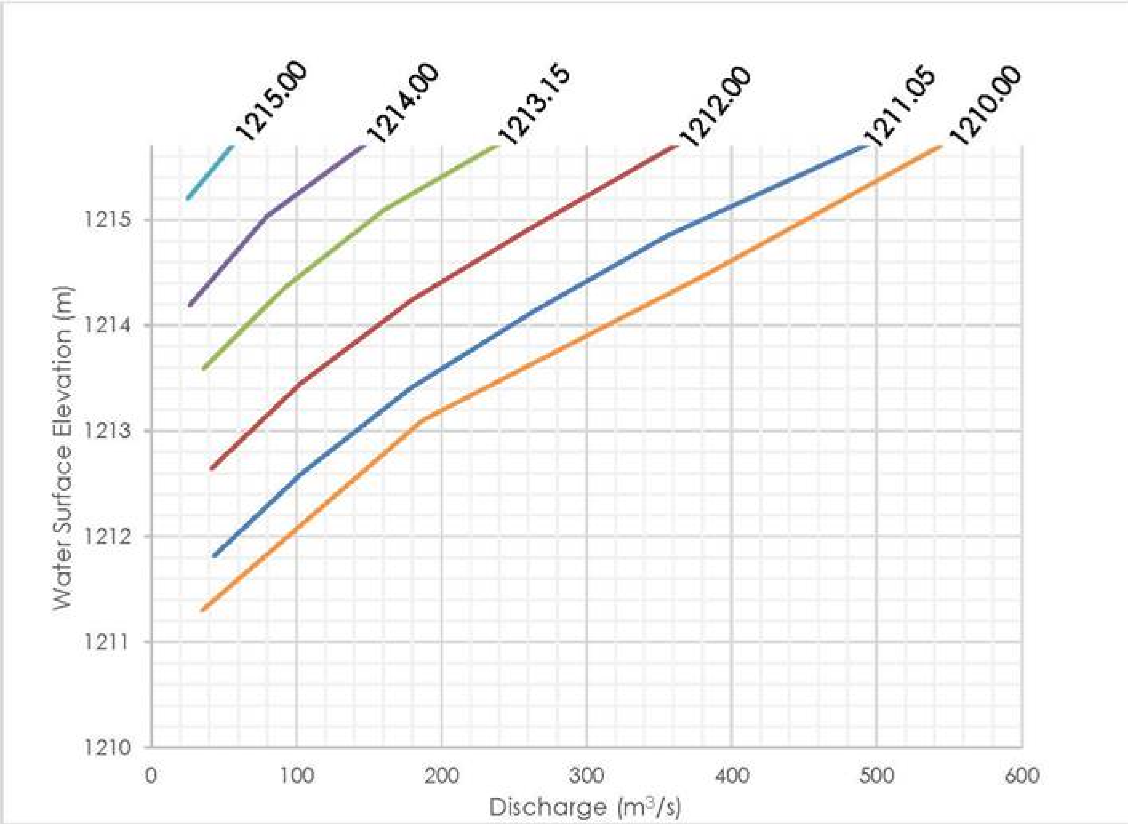


Figure IR496-1 Elevation-Discharge Curves for a Single Bay in the Service Spillway

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Question 497

<u>**Volume 1, Section 5.1.2, Table 5-4, Page 5.4**</u>

Table 5-4 does not discuss potential dam safety issues related to the operation of the low level outlet and there is no separate table for dam safety issues at the low level outlet.

a. **Identify mitigation measures relative to dam safety issues regarding a failure that prevents either opening or closing of the LLO gate.**

b. **Explain if measures for monitoring seepage/piping when the reservoir is inundated are proposed. Explain why or why not.**

Response 497

a. Table IR497-1 lists potential dam safety issues and proposed mitigation measures for the low-level outlet works (LLOW).

Table IR497-1 Mitigation Measures to Reduce Risk to the Low-Level Outlet Works

Identified Risk	Mitigation Measures
LLOW does not close prior to or during operations.	Preventive: Backup hydraulic systems and power are provided. Preventive: Routine inspection, operation and maintenance protocols are performed annually before flood season. Preventive: A secondary gate system is being considered that may include bulkheads or second gate. Reactive: The LLOW gate is not required to meet flood mitigation benefits. Flows into the off-stream reservoir through the diversion channel could be increased to offset the flow rate (up to 27 m^3/s) released through the LLOW during a flood. The diversion channel includes adequate capacity to achieve this increase.
LLOW does not open following operation of the diversion and water remains within the off-stream reservoir.	Preventive: Routine inspection, operation and maintenance protocols are performed annually before flood season. Reactive: Repairs to the gate to be made after diversion ceases using dive crews. Reactive: Syphons and pumping can be utilized to lower the water level in the reservoir, if necessary.

b. Seepage will be monitored when the reservoir is inundated. The operation manual will include a monitoring plan for monitoring seepage when the reservoir is in operation. The final groundwater monitoring plan (see the response to IR46, Appendix IR46-1 for the draft groundwater monitoring plan) may also involve the installation of piezometers located within the embankment and downstream of the LLOW (location of the piezometers will be determined in the final design).

Question 498

Volume 1, Section 5.2, Page 5.6

Regarding embankment breaches along the diversion channel:

a. Explain the potential for and the implications of a breach in the diversion channel bank (e.g. high fill section across an existing creek).

Response 498

a. The diversion channel has two locations along its length where embankments are used to form a portion of the conveyance cross section (water against the embankment during conveyance of diversion flows). Location 1 is approximately 200 m up the diversion channel (toward the off-stream dam) with an embankment height of approximately 8 m. Location 2, also referred to as the saddle dam, is located approximately 1,200 m up the diversion channel (toward the dam) with an embankment height of approximately 10 m (Figure IR498-1).

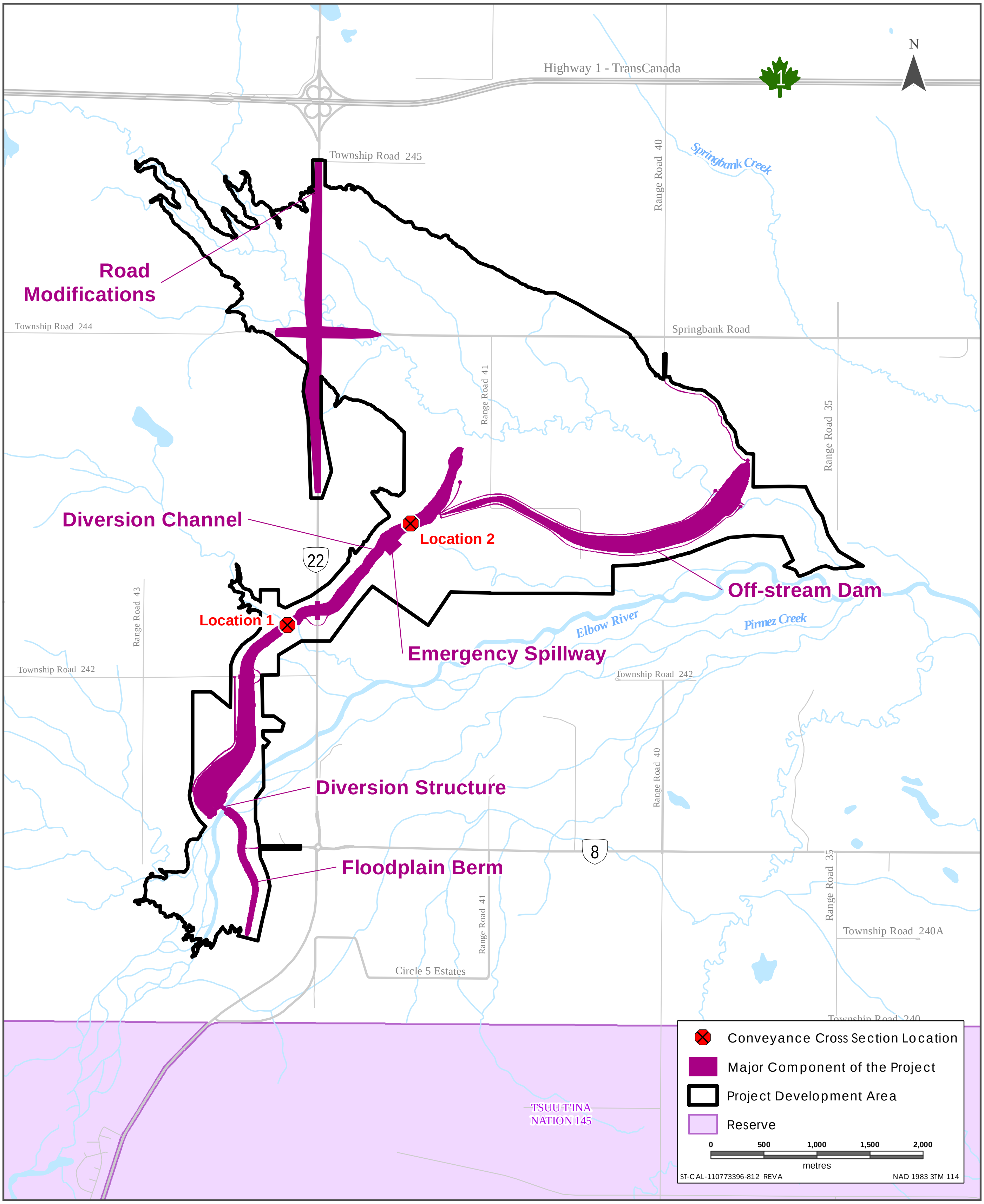
The channel is considered part of the off-stream dam system and, as such, is classified as an "Extreme" consequence system. Design of the embankments, including stability and freeboard, meet requirements of CDA (2007) for an "Extreme" consequence dam.

Volume 1, Section 5.2 describes the implications of a dam breach, including flooding within the City of Calgary. The total release volume from a breach of the off-stream dam (when the reservoir is full) is 77,900 dam^3. At Location 1, the invert elevation of the channel is 1,207.3 m. A breach at this location, with the reservoir at full service level (El. 1,210.75 m), would result in release of approximately 24,000 dam^3, which is approximately 30% of the total volume from the full dam breach. At Location 2, the invert elevation of the channel is 1,203.9 m. A breach at this location, with the reservoir at full service level (El. 1,210.75 m), would result in release of approximately 43,000 dam^3, which is approximately 55% of the total volume from a dam breach at full service level.

Areas between the potential breach locations and Elbow River are uninhabited farm land and riparian forest. Therefore, the unlikely event of a breach of a channel embankment would not cause a material increase in consequences when compared to the unlikely event of a breach of the dam, considering the less release volumes for a channel embankment failure.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.



Sources: Base Data - Government of Alberta, Government of Canada, Thematic Data - Stantec Ltd.

Conveyance Cross Section Locations within the PDA

Stantec

ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESERVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Figure IR498-1

Question 499

Volume 1, Section 5.2, Page 5.6

Four breach scenarios were considered but only three were analyzed.

a. **Clarify if the numbering of scenarios should be revised from 1, 1, 2, and 3 to 1, 2, 3 and 4.**

b. **A flood-induced failure of the off-stream dam was considered but not analyzed. Although this scenario may be of low probability due to the combination of two events (i.e. failure of the dam by piping in combination with failure of the diversion inlet to close), there is some risk of potential downstream inundation due to the dam failure during high inflows such as the PMF. Explain the timing of these two scenarios on the receding limb of the hydrograph and whether the downstream impact would be different with potentially different reservoir levels and inflow.**

c. **Scenarios 3 and 4 refer to flood-induced and post flood failures of the diversion structure. The term diversion structure appears to refer to the entire diversion system on the Elbow River. Clarify which components are susceptible to failure and how they each impact downstream inundation.**

d. **Scenario 3 refers to a flood-induced failure of the auxiliary spillway. Clarify if there is a risk that the floodplain berm could fail potentially due to piping. If appropriate, describe a floodplain berm failure and its impact on downstream inundation.**

Response 499

a. The numbering in Volume 1, Section 5.2 should be revised to 1, 2, 3 and 4.

b. The post-flood breach scenario presented results for a peak outflow of 17,309 m^3/s from a breach of the dam. This flow is reduced to 10,227 m^3/s at Glenmore Dam after approximately 2.5 hours of travel time. Downstream of Glenmore Dam, the peak flow is estimated at 2,971 m^3/s. The total release volume from the post-flood breach scenario is approximately 77,000 dam^3.

   Flood routings of the probable maximum flood (PMF) through the off-stream reservoir indicate that flow within Elbow River will be approximately 1,700 m^3/s (as a result of a breach) when the reservoir reaches the full service level (FSL) of 1,210.75 m and approximately 1,000 m^3/s (as a result of a breach) when the reservoir reaches the PMF elevation of 1,212 m.

   The PMF at Glenmore Dam is estimated to include a total flood volume of approximately 437,000 dam^3 with a peak flow of 2,830 m^3/s.

Failure during the PMF may result in a marginal increase in peak flows (about 5% to 10%) for locations upstream of Glenmore Dam, with a resultant increase in potential inundation area. Downstream of Glenmore Dam, and considering the influence of Bow River downstream confluence with Elbow River, relative effects of the breach during a PMF, compared to the post-flood event, are expected to be small.

c. For the purpose of the dam breach analysis, failure of the diversion structure assumes failure through the auxiliary spillway would release water downstream to Elbow River. Failure of the service spillway was not evaluated because, during the PMF, the gates would be in the lowered position and failure would not create any major change to discharges. Failure of the floodplain berm also was not evaluated because a failure of this structure would be less severe than a failure of the auxiliary spillway (the floodplain berm impounds water of less depth).

d. There is potential that the floodplain berm could fail due to piping; however, a failure of the floodplain berm will be less severe than a failure in the auxiliary spillway, so it was not considered as determining factor for downstream effects. The depth of water impounded by the diversion structure decreases in the floodplain berm, farther away from the diversion structure.

Question 500

Volume 1, Section 5.2, Page 5.7

The following information regarding the Glenmore Dam was obtained from Klohn Crippen Berger Ltd. who are currently working on the upgrades under construction at the Glenmore Dam.

- **Top of dam/bridge deck – El. 1083.906 m, Maximum dam height is 30 m.**
- **Spillway crest – El. 1075.353 m**
- **Top of closed 2.5 m high spillway gates – El. 1077.853 m**
- **Top of containment wall at the non-overflow section of the dam – El. 1079.92 m (existing) and El. 1080.55 m (after proposed upgrades)**
- **Top of high section of the Southeast Dyke – El. 1081.30 m**
- **Top of low section of the Southeast Dyke – El. 1080.44 m**

a. Clarify if the above information is consistent with that used to assess the cascade effects at the Glenmore Dam due to a failure of the Springbank off-stream dam.

b. Evaluate the cascade failure at the Glenmore Reservoir as a result of flood induced failure of the Springbank offstream dam.

Response 500

a. Glenmore Dam parameters were input into the model based on drawings and rating curves provided by the City of Calgary; these parameters describe existing conditions, not future conditions. Specifically, the rating curve was provided in 2014 and a 2014 Klohn Crippen Berger drawing entitled "Glenmore Dam and Spillway" was used to extend the rating curve to higher elevations. Using these data sources, the following values were used:

- top of dam /bridge deck = El. 1,083.7 m
- spillway crest = El. 1,075.33 m
- top of containment wall at non-overflow section of the dam = El. 1,079.92 m
- lowest crest elevation of southeast dyke = El. 1,080.4 m
- average crest elevation of southeast dyke = El. 1,081.1 m

b. The Final Terms of Reference for the Project includes the following requirements in Section 3.2 "(F): Describe the possibility of cascade failure and its impacts*.*"

The evaluation in the EIA considers the possibility of cascade failure and concludes that the likelihood of Glenmore Dam (a concrete gravity structure) breaching to be low. A breach of the earthen saddle dyke on the eastern side of the off-stream reservoir is presented and discussed, as a likely cascade failure scenario. If the Glenmore Dam were to breach, the flows downstream of the dam would exceed the assumptions.

Question 501

Volume 1, Section 5.2, Page 5.7

Alberta Transportation evaluated the potential for downstream cascading failures at the Glenmore Reservoir as a result of the failure of the Springbank off-stream dam for the post-flood scenario.

a. Was the planned bridge and embankment at Stoney Trail included in the hydraulic model? Why or why not? What are the implications of the construction of those works on the potential for cascade failures?

b. Explain the work/basis with regards to the statement *Failure of the concrete gravity dam is considered unlikely to occur in combination with a full breach of the Southeast Dyke*. Explain whether the CDA (2007) guidelines, which specify that the evaluation of the consequences of a dam failure should include the consequences of the failure of downstream dams caused by the failure of the subject dam, are appropriately accounted for in this statement and describe the differences in consequences of failure of the Glenmore Dam versus the SE Dyke.

Response 501

a. The planned bridge and embankment at Stoney Trail were not included in the hydraulic model. The construction was not complete at the time the hydraulic model was run and, therefore, as-built drawings for the proposed structure were not available. A bridge and embankment constructed across the floodplain would likely increase water surface elevations upstream of the structure. Without specific information regarding the bridge geometry and hydraulics, the potential for the roadway to overtop and the effects from a cascade failure downstream cannot be determined.

b. CDA (2007) guidelines address dam breach and inundation analysis for the purposes of risk assessment and dam consequence classification. Breach of the off-stream dam, without consideration of a cascade failure of Glenmore Dam, results in sufficient consequences to warrant the highest dam consequence classification of “Extreme”. Further consideration of additional impacts from a failure of Glenmore Dam are, therefore, not necessary for the dam consequence classification.

   Failure of concrete gravity dams, such as Glenmore Dam, and the resultant breach events are complex processes. Potential breaches may result from a limited structural failure of the dam as one or more monolith sections are forced apart or through sliding of the full dam section with a breach forming at the dam connection with the abutments. The potential for these failures requires major structural assessments of Glenmore Dam and there would still be high levels of uncertainty in the results.

   In contrast, failure mechanisms of earthen embankments, such as the southeast saddle dyke of the off-stream dam are well understood. Overtopping of an earthen embankment by more than 1 m has a high probability of initiating a breach event. With that, if a failure of the off-stream dam was to occur, there is a high probability of failure of the southeast saddle dyke of Glenmore Dam as a result. Therefore, the cascade failure scenario considered the more likely and well understood mechanisms; these are documented in Volume 1, Section 5.2.

   Breach of Glenmore Dam resulting from the breach of the off-stream dam would likely result in a secondary floodwave downstream of Glenmore Dam; however, this secondary flood wave would not result in changes in classification of the off-stream dam or design of the off-stream dam due to the already “Extreme” classification.

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 502

Volume 1, Section 5.2, Table 5-5, Page 5.8
Preliminary Design Report (draft), Appendix C.5 Breach Analysis and Inundation Mapping Report, Section 4.1.2, Table 4, Page 4.10

Alberta Transportation replicated Table 4, Page 4.10 from the *Breach Analysis and Inundation Mapping Report* (Appendix C.5 of the Preliminary Design Report), but modified the footnotes.

a. Clarify whether the footnotes are correct on Table 5-5 of the EIA or Table 4 of the *Breach Analysis and Inundation Mapping Report*.

Response 502

a. The footnotes from Table 4 of the Breach Analysis and Inundation Mapping Report are correct. The footnotes for Volume 1, Section 5.2, Table 5-5 are not correct and should be revised as follows in Table IR502-1.

Table IR502-1 Correction to Footnotes in Table 5-5 (Volume 1, Section 5.2)

Location	Arrival Time after Start of Breach (hr:min)	No Cascade Failure		Glenmore Reservoir Southeast Dyke Cascade Failure	
		WSE[1] (m)	Discharge (m^3/s)	WSE[1] (m)	Discharge (m^3/s)
Elbow River at Breach (Station 44,946)	0:00	1,180.96	17,309	1,080.96	17,309
Elbow River at Sarcee Bridge (Station 19,779)	2:20	1,086.88	10,227	1,086.88	10,227
Glenmore Reservoir Southeast Dyke Overtopping	2:40	1,082.47	2,445	1,082.23	3,314
Elbow River at Glenmore Dam (Station 11,417)	2:40	1,082.14	4,433	1,081.93	4,188
Elbow River at Elbow Drive Bridge (Station 7,206)	3:00	1,059.68	2,971 [2]	1,059.43	2,820 [2]
Elbow River at 1st St (Patterson) Bridge (Station 2,954)	3:20	1,050.68	1,688 [2]	1,050.58	1,611 [2]
Elbow River at 9th Ave Bridge (Station 287)	3:30	1,044.48	2,132 [2]	1,044.48	2,063 [2]
Bow River at 17th Ave (Cushing) Bridge (Station 44,288)	4:00	1,037.86	4,131	1,037.33	3,730
Bow River at Glenmore Trail (Graves) Bridge (Station 37,138)	4:40	1,023.76	3,648	1,023.50	3,282

Table IR502-1 Correction to Footnotes in Table 5-5 (Volume 1, Section 5.2)

Location	Arrival Time after Start of Breach (hr:min)	No Cascade Failure		Glenmore Reservoir Southeast Dyke Cascade Failure	
		WSE[1] (m)	Discharge (m^3/s)	WSE[1] (m)	Discharge (m^3/s)
Bow River at Highway 2 Bridge (Station 18,031)	5:50	988.40	4,017 [3]	988.88	4,658 [3]
Bow River at Confluence with Highwood River (Station 0)	8:10	952.99	3,865	953.43	4,608

NOTE:
[1] Water surface elevation
[2] Discharge for 1D model segments only, does not include flow spread over 2D floodplain
[3] Discharge increases due to flow discharged over Glenmore Reservoir southeast dyke entering Bow River

Question 503

<u>**Volume 3A, Section 5.1.2, Page 5.3**</u>
<u>**Volume 3A, Section 5.1.3, Table 5-1, Page 5.4**</u>

In Section 5.1.2, Alberta Transportation states *As of January 1, 2018, no project-specific intangible concerns were identified with respect to hydrogeology*. In Table 5-1, Alberta Transportation states *Interactions between the Project and groundwater quantity can include: groundwater withdrawals for construction dewatering; groundwater seepage into open excavations; groundwater seepage into the diversion channel when dry.*

a. Specifically, what are the existing groundwater conditions in areas of significant excavation such as the diversion channel? What is the depth of excavation below the groundwater table?

b. After excavation of the diversion channel, what will the effect of groundwater seepage be on slope stability? Is this the same on both sides of the diversion channel or is one side more susceptible to seepage discharging on the slope?

c. How will groundwater seepage on the excavation slopes be controlled or mitigated to prevent erosion or instability?

d. What is the proposed groundwater level monitoring program?

e. What is the expected seasonal variation in groundwater level?

Response 503

a. The base of the excavation for the diversion channel is up to 12 m below the water table in some areas. The deepest portions of the excavation below the water table are located near the inlet of the diversion channel. Near the outlet end (into the off-stream reservoir) of the diversion channel, the base of the excavation is above the water table.

b. Slope stability may be negatively affected by groundwater seepage following excavation in areas that are cut below the water table, depending on the competency of the unconsolidated or bedrock material. Over time, as the groundwater levels reach a new lower equilibrium, the potential for instability will be reduced relative to conditions immediately following excavation. The hydraulically upgradient side of the diversion channel (generally the north side) excavation will be more prone to slope stability issues than the downgradient side (generally the south side) as a result of the water table gradient (sloped toward Elbow River) and the geometry of the excavation.

c. In areas of competent bedrock, no mitigation measures are planned for groundwater seepage. For excavation through unconsolidated deposits or weakly cemented bedrock, seepage will be observed as excavation progresses. In locations where seepage remains persistent and significant following excavation, a treatment scheme will be designed. The treatment is likely to consist of excavation and replacement with sand, stone and geotextile fabric which best addresses the specific conditions.

d. The monitoring is described in the response to IR46, Appendix IR46-1.

e. Seasonal variation in groundwater levels is described in the Hydrogeology TDR Update (see the response to IR42, Appendix IR42-1), Section 3.2.5.

Question 504

Volume 3A, Section 5.2.2.4, Page 5.26

A significant number of groundwater wells were identified in the RAA some of which were located in the LAA. Groundwater quantity and quality effects are expected during dry operation of the diversion canal.

a. How close are some of the water wells to the diversion canal excavation?

b. Which water wells, if any, are within the project development area and require abandonment and which wells are affected by the diversion canal excavation?

c. What is the effect and proposed mitigation measures if required for the wells in close proximity to the diversion canal?

Response 504

a. There are Project-related groundwater wells installed within the footprint of the diversion channel excavation. These wells will be decommissioned prior to construction. Project-related groundwater wells outside the excavation footprint may potentially be used for long-term monitoring, as discussed in the response to IR44.

   One water well location registered in the Alberta Water Well Information Database (AWWID) also is within the footprint of the diversion channel; however, the legal land description centroids are often entered into the Alberta Water Well Drillers Database rather than as surveyed coordinates. Therefore, the well records may not be positioned accurately. The record is for a domestic water well (AWWID #387571) drilled in 1955. This well has not been field verified. If the water well is identified in the field, it will be decommissioned prior to construction. The location of potential water wells in the LAA are presented in Figure 3-28 of the Hydrogeology TDR Update (see the response to IR42, Appendix IR42-1, Section 3).

b. There are 38 water well records within the PDA and 10 have been field verified. Water wells previously installed for domestic or agricultural supply will be assessed to determine the potential for them to act as a pathway for vertical migration of surface water into the groundwater system. Based on well locations reported in the AWWID, only one well falls within the diversion channel alignment. However, field verification of wells within the diversion channel alignment will be required to confirm this prior to construction. A discussion of the wells that will potentially require abandonment is included in the response to IR44.

c. The effects of construction and operation of the diversion channel on water wells close to the diversion channel are described in the Hydrogeology TDR Update (see the response to IR42, Appendix IR42-1, Section 2.3). No mitigation measures are predicted to be required, based on the numerical modelling results.

Question 505

<u>Volume 3A, Section 9.4.2.2, Page 9.33</u>
<u>Volume 3A, Section 9.4.2.3, Page 9.33</u>

Alberta Transportation states that *After standard construction mitigation, there would be a change in terrain stability along the excavated diversion channel banks, off-stream dam and at the diversion structure (Figure 9-11 and Table 9-12)*.

a. Other than a change in slope gradient for the excavation or embankment fill, what other specific factors will lead to the change in stability?

b. What mitigation measures will be incorporated for areas of instability? Section 9.4.2.2 on Page 9.33 does not appear to incorporate any measures to address other types of change in stability.

Response 505

a. During construction and dry operations, changes in groundwater (seepage along cut slopes) and surface drainage may cause local instability; however, these slope instabilities are anticipated to result in minor sliding/sloughing that may be repaired prior to flood operations.

b. Post-construction, there is not expected to be slope instability of the excavated diversion channel, dam embankment and at diversion structure. Steepened slopes at the borrow source area, waste storage sites and high road cuts have potential for minor sliding/sloughing; mitigation may include grading of steepened slopes, grass seeding for exposed slopes to reduce surface erosion, or slope stability site visit investigation as required.

   Seepage along cut slopes would need to be assessed at the site level, depending on where (e.g., along bedrock interface or along glaciolacustrine silt/clay-till contact) and how much seepage is occurring. Mitigation measures may include, but are not limited to, installation of drainage features or over-excavation and replacement of soil with free-draining granular material.

Question 506

Volume 3A, Section 9.4.2.3, Table 9-12, Page 9.35

Alberta Transportation states in Table 9-12 that there is a slight decrease in Stability Class IV terrain for potentially unstable slopes.

a. **Will the presence of groundwater and high plastic soils in the diversion channel excavation (with slopes up to 37 m high) result in some increase in areas of potentially unstable slope? Explain why or why not.**

b. **Similarly does the construction of the offstream dam embankment on the valley bottom sediments result in some increase in potentially unstable slope? Explain why or why not.**

Response 506

a. Yes, the diversion channel excavation will result in an increase in areas of potentially unstable slopes in comparison to existing conditions. However, the diversion channel excavation slopes are designed with consideration of short-term and long-term stability requirements. Slope stability calculations indicate long-term stability factors of safety equal to 1.5 or greater for conditions in which long-term high groundwater conditions are not present.

   It is difficult to predict the effect of the diversion channel excavation on the local groundwater regime, especially given the irregular nature of the Brazeau formation bedding and jointing and how the bedrock and soil groundwater regimes interact with each other. Based on the analyses conducted, locations where the soil slopes exceed 10 m in height above the invert of the channel and groundwater level remains high following development of a post-excavation groundwater equilibrium, groundwater control will be required. Groundwater control may include installation of drains or over-excavation of soil materials and replacement with free draining material, such as sand, gravel and riprap. Groundwater control may also be required at other locations along the channel. The nature of the groundwater control measure will be driven by the amount and persistence of water encountered at various locations.

   Slope stability will be monitored during construction and areas with groundwater seepage (enough seepage to cause instabilities) will be addressed. The actual locations and methods of groundwater control will be determined in the field during construction by observing slope conditions and using piezometer data.

Groundwater conditions can change with time and will be monitored as part of the operations and maintenance program. If groundwater levels rise, risk of sloughing and slope failures increase. Monitoring will allow observation of conditions requiring mitigation, if any, prior to sloughs or slope failures developing.

b. The off-stream dam is designed to meet a slope stability factor of safety of 1.5 for all long-term conditions. Based on this design criteria, slope stability failures are not expected for areas within the vicinity or influence of the off-stream dam.

In addition, the embankment will be monitored and inspected on a regular basis, including prior to any flood operations. Slope stability issues identified will be mitigated prior to operations.

Question 507

Volume 3B, Section 5.2.1, Figure 5-6, Page 5.11
Volume 3B Section 5.2.1, Figure 5-10, Page 5.17
Volume 3B, Section 5.2.1, Figure 5-14, Page 5.23
Volume 3B, Section 5.2.1, Figure 5-18, Page 5.29
Volume 3B, Section 5.2.1, Figure 5-22, Page 5.35
Volume 3B, Section 5.2.1, Figure 5-26, Page 5.41

Alberta Transportation indicates in the figures listed above that for cross-section B-B' at the diversion channel that the groundwater table is situated in the bedrock and the channel excavation only impacts the water table near the base of channel excavation.

a. Is there a groundwater table in the clay and till overburden that will be affected by the channel excavation? Explain.

Response 507

a. The diversion channel will intersect both the potentiometric surface in the bedrock unit as well as the interpreted water table surface. The walls of the diversion channel will allow seepage to occur. There would be a dewatering effect to both the bedrock aquifer and the unconsolidated deposits. Seepage into the diversion channel (when dry) has been estimated by the numerical groundwater flow model (see the response to IR42, Attachment IR42-1, Section 5.5.1). Estimates of seepage into the diversion channel when dry are based on the flux values of the modelling nodes with the diversion channel, and the estimated net seepage into the diversion channel is 0.013 m^3/s.

The local, potentially perched water table will be more significantly affected given that the head difference is greater (approximately 12 m above the diversion channel base) but drains a limited area between the bedrock ridge west of the diversion channel and the

Elbow River. Where the diversion channel intersects the basal silt, sand and gravel unit, seepage rates and area of influence will be greater. The Hydrogeology TDR Update (see the response to IR42, Appendix IR42-1, Section 5) presents results from the numerical groundwater model simulations, including a map (Figure 5-7) of simulated drawdowns expected in the area of the diversion channel (when dry).

Question 508

<u>Volume 3B, Section 5.2.2.1, Page 5.45</u>

Alberta Transportation indicates for the 2013 design flood that *the net change in head, at the point in time where the reservoir is filled, varies from an increase of 28 m (near the upstream toes of the dam) to a decrease of 7 m (in the diversion channel near the inlet structure)*. Lesser values for net change in head (positive and negative) are predicted for the smaller 1:10 and 1:100 year floods.

a. What are the impacts on slope stability in the reservoir during the periods of drawdown after the reservoir is drained?

b. If horizontal permeabilities are greater than vertical permeabilities in the bedrock, how will this influence the drawdown cone (i.e. net change in head) at the base of the diversion channel? Will this result in seepage discharge being higher on the slope? What will the effects be on slope stability? What will the freeze-thaw effects be?

Response 508

a. Residual pore pressure in the banks of the reservoir and dam structure have the potential to negatively affect slope stability until the elevated pressures can dissipate. The drawdown of water from the reservoir will progress in a controlled manner and at a rate intended to mitigate slope stability issues.

b. If horizontal permeability is greater than vertical permeability, then horizontal flow will dominate in both the saturation of and drainage from side slopes of the diversion channel. Any higher permeability zones will saturate more quickly and will see a more rapid reduction in hydraulic head when the channel empties. This could increase the hydraulic gradients within and across the lower permeability units. The short duration of diversion channel flow will limit the level of saturation within low permeability units.

   There is potential for discharge over the entire saturated portion of the slope of the diversion channel excavation. Anisotropy of hydraulic conductivity/permeability will affect the discharge rate, shape of the drawdown cone, and length of time required to reach a new equilibrium level. Soil formations along the diversion channel typically consist of low permeability clays. Accordingly, it is not expected that short-duration channel flows will result in significant saturation of the channel side slopes. High steady state phreatic levels are the more critical stability condition and determined the channel slope design. The bedrock units, while generally poor quality, contain enough fractures to preclude significant pore water retention and sufficient strength to remain stable for the short-term transient flows.

Freeze-thaw cycles will lead to surficial weathering of both soil and bedrock slopes within the diversion channel. The analyses performed considered the likely condition of the soil and rock material.

Question 509

Volume 3B, Section 5.2.2.3, Page 5.46

Alberta Transportation states that *Direction is adverse (in areas where net change in groundwater level is negative) and positive (in areas where net change in groundwater level is positive)*.

a. How is a positive net change in groundwater level (i.e. surcharge in water level) considered positive if there can be negative effects on stability, saturation of subsoil, etc.?

Response 509

a. Effects on groundwater quantity are assessed separately from effects on soils and terrain. As stated in Volume 3A, Section 5.1.6:

> "Residual effects on groundwater resources can in turn cause effects on other VCs. For example, long term changes in the groundwater table could lead to changes in the soil moisture profile, or changes in surface water interactions near wetland features. Where residual effects on groundwater resources could lead to secondary effects on other VCs, the significance determination for those secondary effects are presented in their respective VC sections."

Therefore, the effects on soil saturation and stability are considered in the assessment of the soils and terrain in Volume 3A, Section 9 and Volume 3B, Section 9. The determination of significance of effects on soils and terrain is provided in Volume 3B, Section 9.3.

Question 510

Volume 3B, Section 9.2.2.2, Page 9.5

Alberta Transportation states that *Key mitigation to reduce the effect of soil pore water pressure change within the reservoir during reservoir draining include:*

- ***drawdown of stored flood waters will be conducted in a controlled manner to avoid soil erosion and to maintain slope stability.***

Alberta Transportation also goes on to state that *The predicted residual effects on slopes after recommended post-flood mitigation would be a temporary imbalance in soil pore water pressure within the reservoir. This could result in minor, localized bank slumping immediately following reservoir draining and before the dissipation of pore water pressure*.

a. **Explain what control measures will be incorporated to maintain slope stability of the reservoir slopes during drawdown.**

b. **Will there be a limitation on the rate of drawdown (e.g. control the rate of flow release from the reservoir)? If so, what are the limitations and corresponding reservoir elevation ranges that might be considered critical?**

c. **If there is a limitation on the rate of drawdown in order to ensure that sufficient drainage occurs in the slope so that landslides do not occur, how will this affect the time to drain the reservoir? How will these delays affect other factors such as erosion of the dam slope with no riprap?**

d. **As indicated above Alberta Transportation states that *drawdown of stored flood waters will be conducted in a controlled manner to avoid soil erosion and to maintain slope stability*. Subsequently, it is also stated that drawdown will result in *minor, localized bank slumping*. Explain how slope instability is avoided yet bank slumping will occur. What are the differences?**

e. **Once bank slumping occurs, won't this increase in size over time? Explain why or why not.**

f. **Which reservoir slope areas are most susceptible to slumping? Explain.**

g. **What is the risk of slumping adjacent the dam abutments? Explain.**

Response 510

a. Slope stability analyses evaluated the performance of the reservoir slopes during rapid water release. The stability analyses were conducted assuming the instantaneous removal of the water. For this assumption, slope stability factors of safety are greater than 1.3. This value far exceeds the maximum flow through the low-level outlet. The analysis indicates that control measures would not be required; however, standard industry guidance recommends controlled release of water in a non-emergency (as is planned for this Project) in order to limit residual risk of slumping or slope failure.

b. The recommended maximum release rate of water from the reservoir is 0.3 m per day, based on typical industry guidance for soil embankment dams (see the response to IR465). This drawdown rate is equal to the capacity of the low-level outlet for the critical higher water levels in the reservoir (El. 1,210.8 m to El. 1,203.0 m). For the lower water levels in the reservoir, the control gates will limit the release rate. In the unlikely event of a dam safety emergency, (e.g., such as excessive seepage with evidence of piping) that requires a faster release, the gate will be fully opened.

c. The slope stability analyses do not indicate a need to limit the release rate for slope stability. If required, the reservoir can be drained within 40 days.

d. The referenced section states:

   "This **could** result in minor, localized bank slumping immediately following reservoir draining and before the dissipation of pore water pressure." (bold emphasis added)

   Analyses indicate that the appropriate factors of safety are met for slope stability associated with a rapid release of water from the reservoir. However, these analyses represent generalized conditions. Anomalies may occur and could result in minor, localized bank slumping. The reservoir perimeter will be monitored during release of water and potential issues will be addressed with maintenance and repair.

e. The dam embankment and reservoir perimeter will be monitored during release of water. Areas of slumping will be identified and repaired. Repairs will consider risk for future slope instability development and mitigate future instability, as warranted.

f. Reservoir stability was assessed at sections along the reservoir perimeter. Steeper slopes in the deepest parts of the reservoir are considered more susceptible to slumping than shallower slopes.

g. The risk of slumping at the dam abutments would be expected to be similar to other areas of the reservoir with similar soil profiles. Slope stability analyses for the rapid release of water from the reservoir indicate that factors of safety exceed the design criteria; that is, there is low risk of slumping.

Question 511

Volume 3B, Section 9.2.3.1, Page 9.7

Alberta Transportation states that *Both reservoir filling and reservoir drainage are expected to contribute to water erosion.*

a. Describe the potential for soil erosion on the reservoir slopes adjacent the dam abutments.

Response 511

a. Short-term erosion potential from storm-induced wave development was evaluated for the embankment and abutments, including the 1:2 year and 1:1,000 year sustained wind events. Calculations show that, for a sustained wind event of four hours, potential erosion is less than 47 mm for the 1:1,000 year event and 36 mm for the 1:2 year event.

   Areas of concentrated storm water flow will be rock-lined to prevent erosion from rainfall runoff.

Question 512

Volume 3B, Section 9.2.3.2, Page 9.8

Alberta Transportation states that *Riprap will be installed along some edges of the diversion channel side slopes in critical areas such as outside curves, and on the water face of the off-stream Storage Dam*.

a. Placement of riprap on the water face of the off-stream Storage Dam was not mentioned elsewhere in the project description. Explain where riprap will be placed on the upstream slope of the dam.

Response 512

a. The referenced section should be revised as follows: "Rip rap will be installed along some edges of the diversion channel side slopes in critical areas such as outside curves, and on the water face of ~~the off-stream Storage Dam~~ the embankments that form the channel sides and the saddle dam located between the emergency spillway and diversion channel outlet."

   No riprap will be placed on the upstream slope of the off-stream dam.

Question 513

Volume 3D, Section 1.5.1, Pages 1.18 and 1.19
Volume 1, Section 5.1 Page 5.2 to 5.5

Alberta Transportation very generally discusses the downstream impacts in the event of a PMF failure of the dam embankments and for lesser floods simply indicates that less damages would occur. Volume 1, Section 5.1 describes some of the challenges to dam safety which are potential failure modes.

a. Describe more specifically the extent and severity of damages and mitigations provided to reduce impacts for various levels of floods resulting from the different types of failures described in Volume 1, Section 5.1.

Response 513

a. Volume 1, Section 5.1 identifies potential failure modes at the diversion inlet, service spillway and off-stream reservoir. Additional information regarding failure scenarios and potential impacts is provided below.

DIVERSION INLET

For the diversion inlet, the primary dam safety risk identified is a failure of the gates to close during a flood.

For floods equal to or less than the design flood, the diversion inlet gates do not need to close. The water surface elevation within the river and the diversion rate is controlled by the service spillway gates. Sufficient volume is provided within the reservoir to retain the diverted volume. Failure to close the gates would have minimal consequences.

For floods that exceed the design flood up to the probable maximum flood (PMF), failure to close the diversion inlet gates would result in discharge through the emergency spillway. Discharge through the emergency spillway may result in erosion of the areas downstream of the spillway. Flows downstream of the diversion structure would be equal to or less than a non-failure scenario.

The design considers the potential for gate failure and mitigates the risk through sufficient freeboard for the dam crest and the provision of scour protection within the spillway.

SERVICE SPILLWAY

For the service spillway, auxiliary spillway and floodplain berm, the primary dam safety risk identified is an increase in water surface elevations upstream of the diversion structure. This could occur from failure to operate the service spillway gates as planned or from debris and sediment deposition within the river.

For floods equal to or less than the 1:100 year flood, an increase in water surface elevations upstream of the diversion structure is unlikely to affect planned operations. Sufficient capacity is available within the service spillway and diversion channel to mitigate a potential increase. Activation of the auxiliary spillway is unlikely. Consequences are low.

For floods greater than the 1:100 year flood, failure mechanisms that result in increases in the water surface elevations may result in activation of the auxiliary spillway. This may result in erosion of the cover soil and requirements for post-flood maintenance. Flood diversion would be expected to continue and the diversion volume goals for flood management would still be met.

For floods greater than the 1/3 between the 1:1,000 year and PMF, overtopping or circumvention of the floodplain berm could occur. A breach resulting from this scenario would have minor effects downstream, as described in Volume 1, Section 5.2.

OFF-STREAM DAM

The primary dam safety risk is failure through piping resulting in breach of the dam and release of water from the reservoir. Results for a breach of reservoir when it is full are presented in Volume 1, Section 5.2. Failure of the dam at the time of a smaller amount of water is in the reservoir would have lesser effects than those described, but effects may still result in substantial damages downstream.

Question 514

<u>Volume 3D, Section 1.5.1, Page 1.18</u>

Alberta Transportation states that *Accordingly, the off-stream dam has been designed to standards consistent with a hazard classification of "very high" per the AEP guidelines and "extreme" per the CDA guidelines (see Volume 1, Section 5.0 for details on dam safety design)*. AEP guidelines are no longer used for consequence classification.

a. Clarify the extent of consequences for each category of damages (e.g. loss of life) while recognizing that all of the consequence damage categories may not be of the same severity and state which categories dictate the overall classification for each component.

Response 514

a. The off-stream dam and floodplain berm are designed according to industry standards with load factors consistent with the consequence classifications Accordingly, a failure of either facility is considered a very low probability.

For the off-stream dam and associated facilities, "Extreme" consequence from CDA (2007)guidelines dictated the design standards. All three of the listed consequence damage categories led to the overall classification:

- population at risk, permanent
- loss of life, more than 100
- infrastructure and economics, extreme losses affecting critical infrastructure or services

For the floodplain berm system, including the service spillway and auxiliary spillway, "High" consequence from the CDA guidelines dictated the design standards. One of the listed consequence damage categories led to the overall classification: infrastructure and economics, high economic losses affecting infrastructure, public transportation and commercial facilities.

REFERENCE

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario.

Question 515

Volume 3D, Section 1.5.1 Page 1.18
Preliminary Design Report (draft), Section 10.1.2, Figure 39, Page 152

Alberta Transportation states that *Another design feature that serves-dual purpose in dam safety is the inclusion of breast walls above the diversion inlet gate bays. If the diversion inlet gates fail to close during flood operations, the breast walls would limit the flow of flood water into the off-stream reservoir to 600 m3/s, which can be managed by the emergency spillway*. Background documents (Preliminary Design Report Figure 39) indicate a peak diversion of over 850 m^3/s.

a. Confirm whether the design flow of 600 m^3/s for the canal is for the 2013 flood and explain whether flows can exceed this amount for larger floods such as the PMF.

Response 515

a. Discharge in the channel will not exceed 600 m³/s. Flows exceeding 600 m³/s would be the result of an operations failure.

   As an example, if an operator were to fail to close the diversion inlet gates during the probable maximum flood (PMF), 850 m³/s would enter the diversion channel at the peak of the flood hydrograph.

Question 516

Volume 3D, Section 1.6.2, Page 1.31
Volume 1, Section 3.2.1.5, Page 3.10

Alberta Transportation states that *The service spillway and auxiliary spillway have the capacity to pass up to 1/3 the flow rate between the 1:1000 year flood and the PMF, in accordance with the guidelines. The auxiliary spillway is designed to activate when inflow exceeds the capacity of the service spillway and diversion inlet*. Volume 1, Section 3.2.1.5, Page 3.10 states that *The auxiliary spillway is designed to withstand overtopping for flood events up to 1/3 between the 1:1000 and the PMF with an overtopping depth of 1.5 m*.

a. Clarify which portion of the design flood can be discharged through the service spillway and at what flood frequency/magnitude the operation of the auxiliary spillway is activated with and without diversion to the offstream reservoir.

Response 516

a. The service spillway has a capacity of 1,240 m³/s without activation of the auxiliary spillway and assuming no diversion. During diversion, the service spillway and diversion inlet, in combination, can convey flows up to 1,740 m³/s (an approximate 1:500 year flood) without activation of the auxiliary spillway.

   At the peak of the inflow design flood (IDF, 1/3 between the 1:1,000 year flood and the probable maximum flood), the flows are as listed in Table IR516-1.

Table IR516-1 Flows in Project Components During an IDF

Scenario	Diversion Inlet (m³/s)	Service Spillway (m³/s)	Auxiliary Spillway (m³/s)
Diversion gates open (failure to close)	746	1,215	249
Diversion gates closed	0	1,570	640

The 640 m^3/s is the highest discharge rate during the IDF, which results in an average overtopping depth of 1.5 m.

Question 517

Volume 3D, Section 1.6.2, Page 1.31

Alberta Transportation states that *The pneumatic system that raises the service spillway gates are designed to operate effectively if one or more air bags are damaged and fail in the downward position for safe passage of flow.*

a. Describe the number of air bags for each gate and indicate the minimum number of air bags to raise the gates in the event that one or more air bags fail.

b. Confirm that air supply lines for each air bag are separate and not affected by a malfunction or damage to one or more air bags.

Response 517

a. The service spillway gates consist of two 24 m-wide by 5 m-high crest gates manufactured by Obermeyer. Each crest gate consists of four 6 m-wide panels arranged linearly. Each panel consists of a 6 m-wide steel leaf and clamping plate supported by a dual-chamber bladder. The dual air chambers are oriented one on top of the other and connected in series to act together as one composite bladder for each panel.

   The four panels of an individual crest gate are intended to be inflated/deflated in unison. To provide a seal between panels, the steel leaves of adjacent panels are spliced together with a rubber seal connection. In the event of bladder failure or large differential pressure between adjacent panels, the seal connector will tear and allow a single panel to move independently without affecting the position of adjacent panel(s). With a failed bladder, the affected panel will move to the “fail-safe” down position while the remaining panels continue to divert flow. The crest gate, using the remaining three panels, can remain operable should one panel within the crest gate move to the down position.

b. It is confirmed that the dual-chamber bladder in each 6 m-wide panel is fed from a single air supply line contained within a conduit embedded within the concrete and would not be affected by a malfunction or damage to one or more adjacent air bags.

Question 518

Volume 3D, Section 2.3.3, Page 2.7

Alberta Transportation states that *Damage to infrastructure caused by wildfires, seismic events, or tornadoes during flood operations could result in dam failure or breach.*

a. Describe how wildfires and tornadoes would result in a dam failure or breach and what mitigation measures have been included.

Response 518

a. The text referenced in Volume 3D, Section 2.3.3, Page 2.7 is not correct. There is no risk of dam failure or breach associated with wildfires or tornadoes.

Question 519

Preliminary Design Report (draft) Appendix B.4-1 PMF Analysis Report, Section 4.3.1, Page 33

Alberta Transportation states that *a common practice in British Columbia (BC) and Alberta is to precede the PMP with a 100-year 24 hour rainfall leaving a period of three days between the storms.*

Alberta Transportation describes that antecedent event selection is a common practice in the United States, but is not a common practice in Canada. It recommends that the antecedent precipitation for Alberta projects be taken as the 1:10 year 48-hour rainfall that finishes five days before the PMP.

a. Discuss the implications for the PMF estimate of using a PMP with a different antecedent event.

Response 519

a. The primary implications associated with using a probable maximum precipitation (PMP) with a different antecedent event is the routing of the antecedent event through the dam structure. The selected event (1:100 year, 24-hour) is more conservative in nature and results in a larger antecedent volume and slightly higher peak flow for the subsequent probable maximum flood (PMF). This conclusion is based on the result of hydrologic modelling of both the 1:100 year, 24-hour and 1:10 year, 48-hour events preceding the PMF.

Using an 1:10 year, 48-hour antecedent event would not change the design recommendations for the Project because the peak flow reduction in the PMF (approximately 40 m^3/s) is less than 2% of the peak for the PMF using the 1:100 year, 24-hour antecedent condition.

Question 520

Volume 1, Section 3.1, Pages 3.1 Volume 1, Section 3.1, Page3.2

Alberta Transportation states that some of the project components are designed for the PMF, which was estimated using a PMP analysis and hydrologic modelling.

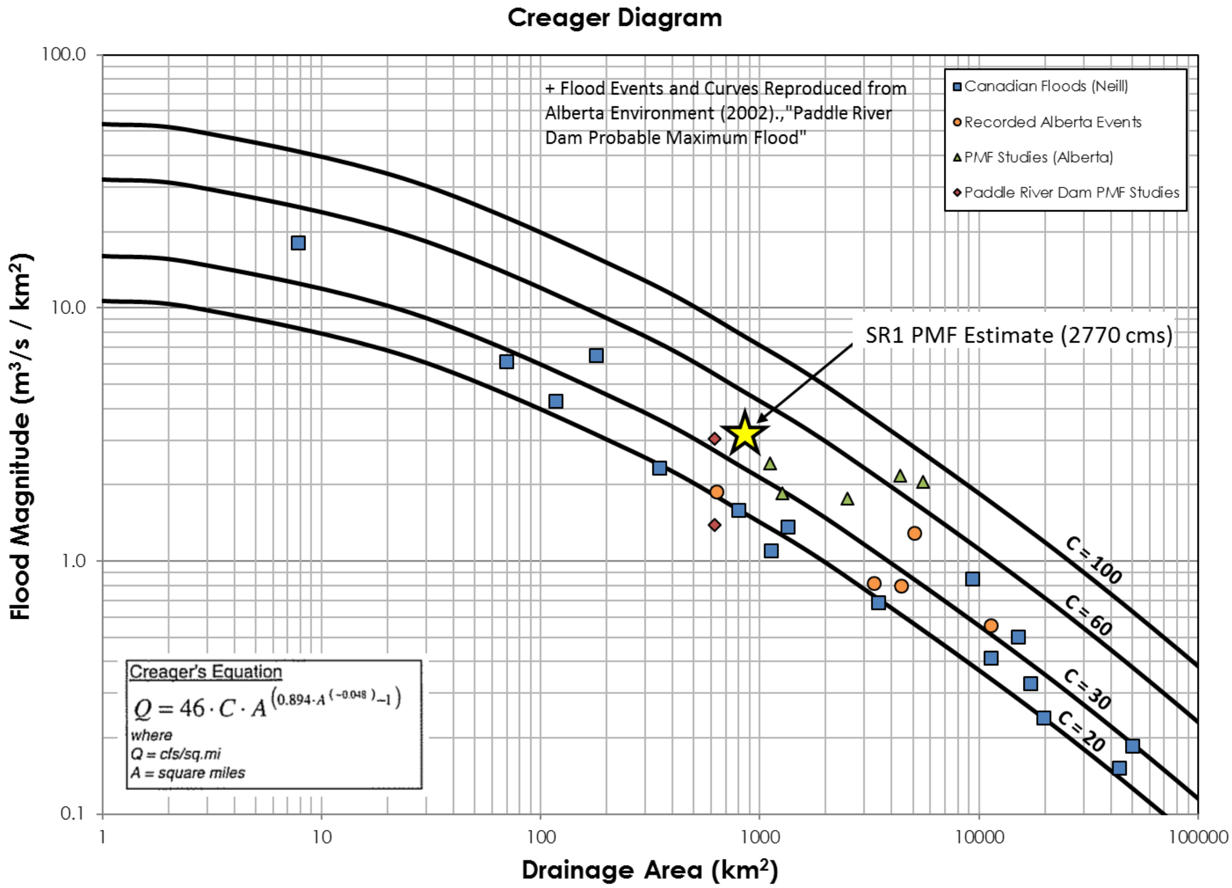
a. **Provide details on the derivation of the PMF.**

b. **Provide a comparison of the SR1 PMF estimate with PMF values estimated in other studies for sites in the same region (e.g. on a Creager plot).**

Response 520

a. Details regarding the derivation of the probable maximum flood (PMF) are provided in the report titled Probable Maximum Flood Analysis, attached as Appendix IR520-1.

b. Figure IR520-1 provides a comparison of the Project's PMF estimate to other studies in the region and the Creager plot. As demonstrated in the figure, the Project PMF estimate is equal to or greater in magnitude than any of the recorded Alberta or Canadian floods of historic record. The Project PMF estimate plots within the median range of past PMF studies in Alberta.



NOTE: cms = m^3/s

Figure IR520-1 Comparison of PMF Estimates

Question 521

Volume 1, Section 3.2.1.1, Page 3.6

For the diversion inlet:

a. **Discuss the structural design of the main structure components including design approach, loading conditions and parameters.**

b. **Discuss the stability of the main structure components including loading conditions, parameters, target and actual factors of safety.**

c. Discuss instrumentation to monitor performance (e.g. deformations).

d. Discuss anticipated quantities of construction materials required.

e. Describe construction requirements including temporary works (e.g. site clearing and grubbing, dewatering systems, foundation preparation, etc.).

Response 521

a. The design approach, loading condition and parameters are presented below.

DESIGN APPROACH

The hydraulic structures are grouped into individual structures (for the purposes of stability assessment, analysis and structural design):

- monoliths for mass concrete structures, such as gate crest monoliths
- reinforced concrete monoliths, such as retaining walls, based on structure geometry, size, joint location, and loading considerations

The structures were analyzed as either concrete gravity sections (gate structures, piers), retaining walls (wing walls, training walls, and abutment walls), or as independent reinforced concrete structures. Each structure was evaluated for global stability, strength, and serviceability.

Global stability was assessed using the rigid body analysis method and application of unfactored loads. This method uses the summation of forces applied to the structure to determine resultant location, foundation bearing pressures, sliding resistance along identified potential failure plane(s), and floatation.

Strength evaluation of individual elements or members of structures and monoliths was used to verify member sizes based on application of factored loads as described in USACE (2016). Additional elements evaluated as part of strength design included joint detailing, equipment anchorage, and embedded parts.

Serviceability includes limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR).

LOADING CONDITIONS AND PARAMETERS

Loading conditions and acceptance criteria are based on Alberta Transportation (2004), CDA (2007a; Table 6-4), and CDA (2007b; section 6.0). The load cases evaluated are grouped into five categories, as listed in Table IR521-1.

1. *USUAL CONDITION*

These are conditions under which the structure is intended to serve during normal operations and further defined as a condition that has a high likelihood of occurring within the design life of the structure. Usual load conditions include normal pool and winter conditions. For the hydraulic structures, this includes flood events up to the 1:100 year flood.

2. *UNUSUAL CONDITION*

These conditions occur infrequently and may stress the structure more, under certain aspects, than normal conditions and may occur within the design life of the structure. Unusual load conditions include:

- construction conditions
- maintenance conditions
- floods between the 1:50 year and 1:1,000 year floods
- infrequent earthquake events other than the maximum design earthquake (MDE)
- plugged drain conditions for usual load cases

3. *EXTREME – FLOOD*

Extreme load conditions have a very remote likelihood of occurring with the design life of the structure. For the Project, it is defined as those floods that occur from the 1:1,000 year flood up to the structure's IDF.

4. *EXTREME – EARTHQUAKE*

This load condition is the MDE because it has a remote likelihood of occurring with the design life of the Project. The MDE is applied to the usual condition load cases. The extreme – earthquake condition is used to establish post-earthquake condition of the hydraulic structure. Thus, there are no stability acceptance criteria for this condition.

5. *POST-EARTHQUAKE*

This condition assesses the stability of the hydraulic structure following the applied seismic event, based on earthquake induced cracking at the foundation structural interface and within the structure so that it can still capable of resisting the usual loading.

Table IR521-1 Acceptance Criteria for Hydraulic Structure

Loading Combination	Position of Resultant Force (Percent of Base in Compression)[1]	Normal Compression Stress[2]	Sliding Safety Factor	
			Friction Only	Floatation Safety Factor
Usual	Middle third of the base: 100% compression	<0.3 x f_c	≥1.5	≥1.5
Unusual	Middle third of the base: 100% compression	<0.5 x f_c	≥1.3	≥1.3
Extreme Flood	Within middle half of the base, and all other acceptance criteria must be met	<0.5 x f_c	≥1.1	≥1.1
Extreme Earthquake	Within the base, except where an instantaneous occurrence of resultant outside the base may be acceptable	<0.9 x f_c	Note [3]	
Post-Earthquake	Within middle half of the base	<0.5 x f_c	≥1.1	≥1.1

NOTES:

[1] Foundation bearing stress is compared to allowable stress determined from Geotechnical Investigation

[2] Where f_c = compressive strength of concrete

[3] The earthquake load case is used to establish post-earthquake condition of the structure

b. Stability analyses were performed in accordance with the structural design criteria described in response a). The stability results for the centre pier monolith (Table IR521-2) are presented for usual, unusual, and extreme loading conditions as representative of the potential range of conditions the structure will be exposed to during the design life.

Stability analyses indicate a stable structure within the limits of acceptance criteria. For all loading conditions considered, floatation factors of safety are above required; 100 percent of the base is in compression for all loading conditions and sliding factors of safety are above required. Stability results indicate that sliding stability is the primary concern due to the low friction angle at concrete/rock interface and rock/rock bedding planes, but adequate factors of safety were achieved assuming a horizontal failure plane while conservatively neglecting the stabilizing contribution of the upstream. The controlling load cases at this time are considered to be Load Case E4 (earthquake design ground motion applied during 1:100 year flood), and Load Case UN3 (1:1,000 year flood with no diversion).

Structural design of the diversion inlet structure continues and will meet all required design criteria.

Table IR521-2 Diversion Inlet Centre Pier, Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases										
U1 Normal Operation	1,212.1	1,208.5	12,976	1.5	4.14	1.5	3.78	163	120	100
U2 Diversion Operation *100 Yr. Flood*	1,215.8	1,214.1	24,970	1.5	2.33	1.5	3.69	147	84	100
Unusual Load Cases										
UN1 Diversion Operation *2013 Flood*	1,215.8	1,214.1	24,970	1.3	2.33	1.3	3.69	147	84	100
UN2 No Diversion *2013 Flood*	1,216.2	1,208.5	21,305	1.3	2.63	1.3	2.37	140	100	100
UN3 No Diversion *1000 Yr. Flood*	1,217.0	1,208.5	22,930	1.3	2.46	1.3	1.83	144	88	100
UN4 Construction/Maintenance	1,215.8	1,212.1	23,371	1.3	2.35	1.3	5.44	104	116	100

Table IR521-2 Diversion Inlet Centre Pier, Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Extreme – Flood										
E1 PMF without Diversion	1,218.0	1,208.5	24,962	1.1	2.28	1.1	1.36	150	71	100
E2 PMF with Diversion	1,216.6	1,215.1	27,395	1.1	2.17	1.1	3.87	145	76	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E3 EDGM applied to U1	1,212.1	1,208.5	12,976	1.1	4.01	1.0	1.26	199	72	100
E4 EDGM applied to U2	1,215.8	1,214.1	24,970	1.1	2.27	1.0	1.00	187	32	100

NOTES:
Analysis assumes horizontal sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

c. Monitoring of structures may include the following:

- Surface measuring points would be located on top of retaining and training walls using embedded stainless steel carriage bolts or cap screws located 150 mm on either side of a vertical formed joint. This provides measurable differential movement between walls at the joint and joint opening. In addition, wall deformations and block movement can be detected based on intermittent surveys of the measuring points.
- Water measurement weirs would be located at end of drain pipe outfalls that are used behind structure retaining walls. Sometimes measuring weirs are located within dry wells that serve as junctions for retaining wall drains.

Settlement, deformation and block movement can be detected based on intermittent surveys of the measuring points and other surface monuments.

d. Construction of the diversion inlet will require approximately 3,400 m^3 of reinforced structural concrete, 9,100 m^3 of mass concrete and 44,000 kg of structural metal framing. In addition, the construction will include installation of the fixed wheel lift gates, gate hoist system and controls.

e. The following is a list of activities that may be conducted as part of construction of the diversion inlet:

- Clearing and grubbing will be performed in conjunction with the excavation of the diversion channel.
- The excavation will be isolated from Elbow River using cofferdams and groundwater seepage will be pumped from the excavation.
- Foundation preparation will require cleaning and preparation of concrete/rock interface. Preparation will also include installation of foundation under drain to control uplift pressures.
- Anchors will be required to maintain adequate factors of safety against floatation in the stilling basin. The anchors will be drilled and grouted in a grid pattern prior to placement of the stilling basin concrete.
- The diversion inlet structure and gates will be functional before a tie-in with the diversion channel is made. The gates will be load tested to verify functionality of the diversion inlet gates and gate structure.

REFERENCES

Alberta Transportation. 2004. Structure Stability. Accessed at: http://www.transportation.alberta.ca/Content/doctype125/Production/S8DsgnGdln.pdf

CDA (Canadian Dam Association). 2007a. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario

CDA. 2007b. Technical Bulletin No. 8. Geotechnical Considerations for Dam Safety. Canadian Dam Association. Toronto, Ontario

USACE (US Army Corps of Engineers). 2016. Strength Design for Reinforced Concrete Hydraulic Structures. Accessed at: https://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-2104.pdf

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Question 522

Volume 1, Section 3.2.1.2, Page 3.7.

For the service spillway:

a. Discuss the structural design of the main structure components including design approach, loading conditions and parameters.

b. Discuss the stability of the main structure components including loading conditions, parameters, and target and actual factors of safety.

c. Discuss instrumentation to monitor performance (e.g. deformations).

d. Discuss anticipated quantities of construction materials required.

Response 522

a. The design approach, loading condition and parameters for the service spillway are the same as for the diversion inlet; see the response to IR521a.

b. Stability analyses were performed in accordance with the structural design criteria outlined above. The stability results for the center pier monolith (see Table 522-1) are presented for usual, unusual, and extreme loading conditions as representative of the potential range of conditions the structure will be exposed to during the design life.

Table IR522-1 Service Spillway Center Pier, Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases										
U1 Normal Operation	1,212.1	1,211.8	12,958	1.5	2.04	1.5	2.19	85	36	100
U2 Diversion Operation *1;100 year flood*	1,215.8	1,211.8	15,680	1.5	1.85	1.5	2.01	70	50	100
Unusual Load Case										
UN1 Diversion Operation *2013 Flood*	1,215.8	1,213.1	19,810	1.3	1.59	1.3	1.50	73	32	100
UN2 Diversion Operation *2013 Flood*	1,216.1	1,213.0	20,237	1.3	1.57	1.3	1.34	72	31	100
UN3 No Diversion *1:1,000 year flood*	1,217.0	1,214.7	22,651	1.3	1.51	1.3	1.65	55	50	100
UN4 Construction/Maintenance	1,215.0	1,212.5	18,182	1.3	1.50	1.3	0.99	60	22	100

Table IR522-1 Service Spillway Center Pier, Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Extreme – Flood										
E1 PMF without Diversion	1,217.8	1,215.3	24,280	1.1	1.47	1.1	1.33	58	44	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E2 EDGM applied to U1	1,212.1	1,211.8	12,958	1.1	1.98	1.0	0.82	97	17	100
E3 EDGM applied to U2	1,215.8	1,211.8	15,680	1.1	1.80	1.0	0.74	86	27	100

NOTES:

Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.

Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

Stability analyses indicate a relatively light structure that is sensitive to sliding instability when crest gates are used to retain water during diversion operation. Stability calculations indicate results within the limits of acceptance criteria utilizing an inclined base analysis. For all loading conditions considered, floatation factors of safety are above what is required; 100 percent of the base is in compression for all loading conditions and sliding factors of safety are above required. Stability results indicate that sliding stability is the primary concern due to the low friction angle at concrete/rock interface and rock/rock bedding planes. To ensure an inclined failure plane utilized in the analysis is valid, the upstream shear is designed as a structural element. The controlling load cases at this time are the Load Case E3 (earthquake design ground motion applied during 1:50 year flood), and Load Case UN4 (construction/maintenance dewatered during a 1:100 year flood).

Structural design of the service spillway structure continues and will meet all required design criteria.

c. Monitoring of structures are the same as for the diversion inlet; see the response to IR521c.

d. Construction of the service spillway requires approximately 2,300 m^3 of reinforced structural concrete and 10,200 m^3 of mass concrete. In addition, the construction will include installation of the steel crest gates, pneumatic bladder system and controls.

Question 523

Volume 1, Section 3.2.2, Page 3.11

For the diversion channel:

a. Discuss the stability of the canal slopes including the extent of toe erosion expected. Provide the loading conditions, parameters, and actual and target factors of safety.

b. Discuss the testing and suitability of excavated materials for use as fill.

c. Discuss anticipated quantities of construction materials generated and required.

Response 523

a. Slope stability calculations for the diversion channel slopes were performed in Geostudio Slope/W. The stability of the channel slopes was analyzed at the following cross sections (Figure IR523-1):

- Station 10+150 is the channel inlet from Elbow River with a benched profile to account for the proposed access roads on either side of the channel.
- Station 10+400 is a 30 m deep excavation into bedrock and overlying units.

- Station 11+000 is the utility crossing for the Nova Chemicals and Pengrowth pipelines and the AltaLink overhead transmission line.
- Station 11+400 is an excavation entirely in glaciogenic units.
- Station 11+900 is the utility crossing at the TransCanada Pipelines location.
- Station 12+400 is an excavation at the planned Highway 22 bridge.
- Station 13+600 is a side-long excavation on the north side of the diversion channel and construction of an embankment on the south side.

Loading Conditions

The possible consequences of a slope failure will depend on the location of the failure along the channel length and water level in the reservoir at the time of failure. The slope stability assessment made the following assumptions:

1. For water to reach the full service level in the reservoir, the diversion channel will be in operation for a maximum of 3.75 days. Due to the low permeability of the clay soils at the site, it is assumed that the excavated slopes will not become saturated from water within the channel during that time. Accordingly, rapid drawdown conditions are not considered critical to slope stability.
2. While the diversion channel must remain clear and open during diversion, the flood waters loading applied to the base of the slope would act as a stabilizing toe surcharge, increasing the stability over the dry channel condition.

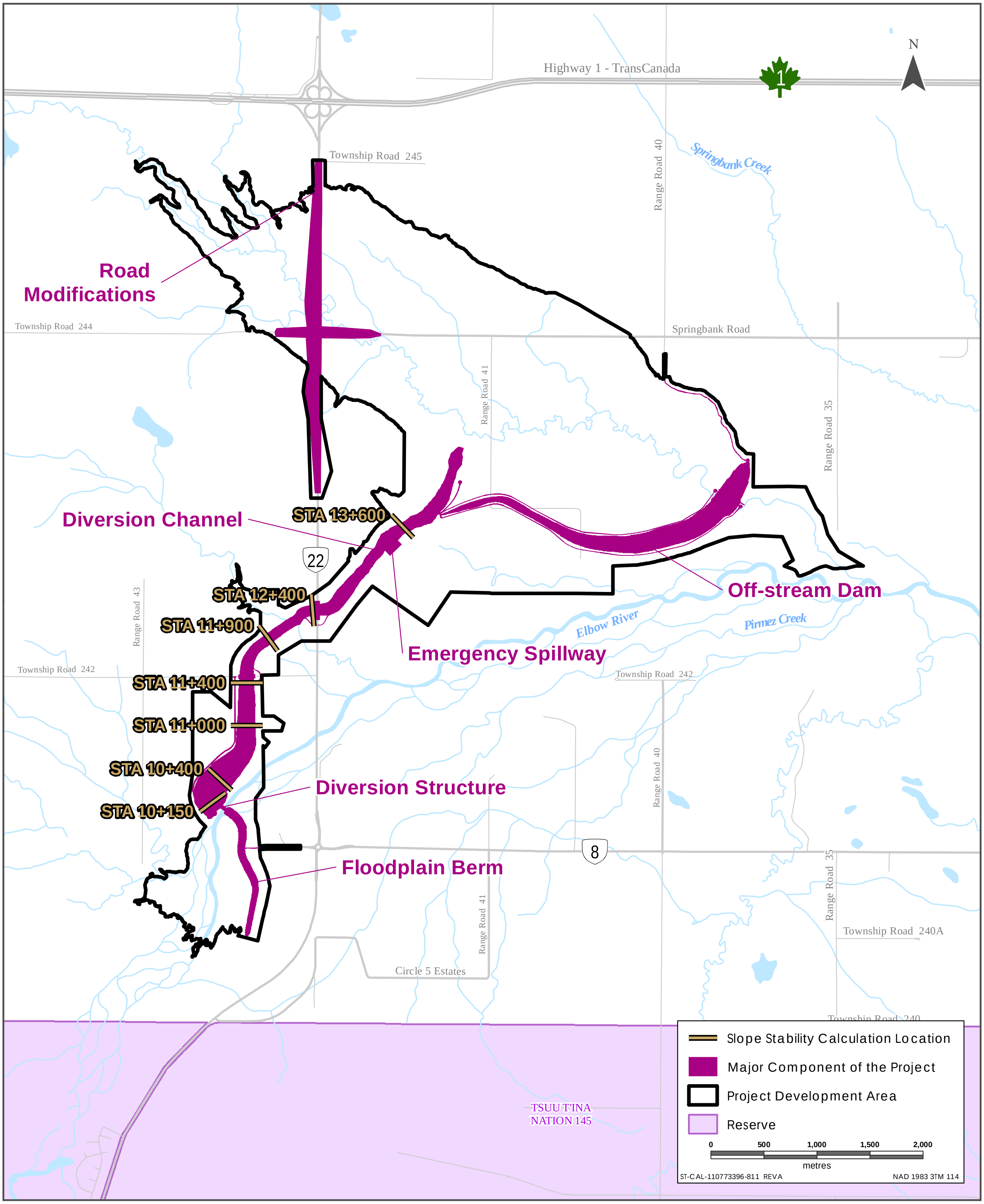
Therefore, the critical slope stability condition is the long-term condition (see Table IR523-1) with no channel flow and steady state groundwater conditions.

Table IR523-1 Critical Design Load Case for the Diversion Channel

Load Case	Reference	Reservoir	Foundation Behavior	Pore Pressures	FOS
Long Term	CFEM	None	Drained strength parameters	Assumed steady state phreatic surface	1.5

The design load case for the channel is in accordance with CDA (2007) and CGS (2007).

Erosion of the channel bottom is not expected within the excavated bedrock sections. Critical areas with the channel bottom excavated in soil—including bridges, utility crossings, hydraulic structures and embankments forming the sides of the channel—are protected from erosion with riprap. Minor toe erosion may occur in unlined soil sections of the channel. However, the associated impacts are not expected to affect operation of the channel and the risks are judged acceptable.



Sources: Base Data - Government of Alberta, Government of Canada, Thematic Data - Stantec Ltd.

Slope Stability Calculation Locations within the PDA

Stantec

ALBERTA TRANSPORTATION SPRINGBANK OFF-STREAM RESERVOIR PROJECT ENVIRONMENTAL IMPACT ASSESSMENT

Figure IR523-1

PARAMETERS

Input parameters were derived from laboratory testing, published literature and local experience. The parameters used in the analysis are summarized in Table IR523-2.

Table IR523-2 Slope Stability Analysis Input Parameters

Material	Input Parameters		
	Unit Weight	Friction Angle	Cohesion
Riprap	22	38	0
Glacial Lacustrine	18	23	0
Glacial Till	18	27	0
Basal Granular Till	22	34	0
BZF – Cross-bedding strength	21	28	*
CSF – Cross-bedding strength	21	*	*

SLOPE STABILITY RESULTS

Slope stability results for each section are summarized in Table IR523-3.

Table IR523-3 Slope Stability Results

Station	Failure Mechanism	Target Factor of Safety	Calculated Factor of Safety
10+150	Rotational material through GL/GT and underlying Brazeau	1.5	1.5*
10+400	Rotational failure through the glaciogenic (GL and GT) units	1.5	1.5*
11+000	Rotational failure through the glaciogenic (GL and GT) units.	1.5	1.5
11+400	Rotational failure through the glaciogenic (GL and GT) units.	1.5	1.6*
11+900	Rotational failure through the glaciogenic (GL and GT) units.	1.4	1.6*
12+400	Rotational failure through the glaciogenic (GL and GT) units.	1.5	1.5*
13+600	Rotational failure through the embankment into the underlying glaciogenic (GL and GT) units.	1.5	1.6

NOTE:
* Indicates groundwater lowered through natural drainage or through installation of drainage to facilitate slope stability.

The factor of safety (FOS) of the diversion channel slopes is dependent on the management of groundwater at appropriate elevations to produce satisfactory FOS. The analyses are based on idealized conditions, including specific uniform boundary conditions and homogeneous soil and bedrock conditions. The actual conditions are likely to vary from those assumptions and may include variations along the length of the channel. Accordingly, it is expected that at different locations along the channel, the groundwater level will be higher or lower than those predicted by the models at the sections presented. Therefore, flexibility will be included in the construction plans to allow modification of the groundwater control approach to adapt to the conditions encountered along each segment of the channel, including the installation of drains, as needed.

Design of the channel is ongoing, and results of the above analyses may change as a result. However, the design will meet the specified factors of safety under the required loading conditions.

b. Thirty-eight borings and three cone penetration test (CPT) soundings were completed along the diversion channel. The field activities consisted of advancing auger borings, sonic borings, and CPT borings. At certain locations, both CPT and auger borings were advanced near each other to calibrate CPT results. Boring logs, laboratory testing, and CPT data were reviewed to develop soil horizons and soil material properties.

Data from the borings were used to characterize the subsurface materials along the diversion channel. Soil samples from the diversion channel and borrow source boring locations were used to characterize the planned embankment materials, assuming the excavated soils will be used to construct the dam. These disturbed samples from the diversion channel were used to remold and test specimens in the laboratory to determine representative embankment properties.

The construction area is divided into 13 geographical zones. The zone locations are shown on Figure IR523-2. Borings and laboratory test results are organized by zone for the different soil types to determine soil parameters for use in the analyses. Zones 1 and Zone 2 indicate location of the floodplain berm and diversion structure. Zone 9 through Zone 13 indicate the construction footprint of the dam. Zone 1 to Zone 2 and Zones 9 to Zone 13 are not expected to provide substantial material for embankment construction. The footprint of the diversion channel is divided into six zones (Zones 3 through Zone 8). The diversion channel and borrow source borings were reviewed to determine characteristics for the material to be used to construct the off-stream dam.

Design soil parameters were selected for the seepage and stability analyses of the embankment dam and the diversion channel, based on the laboratory test results and field data. Rock parameters were selected for diversion channel slope stability based on both laboratory test results and field observations of the rock mass.

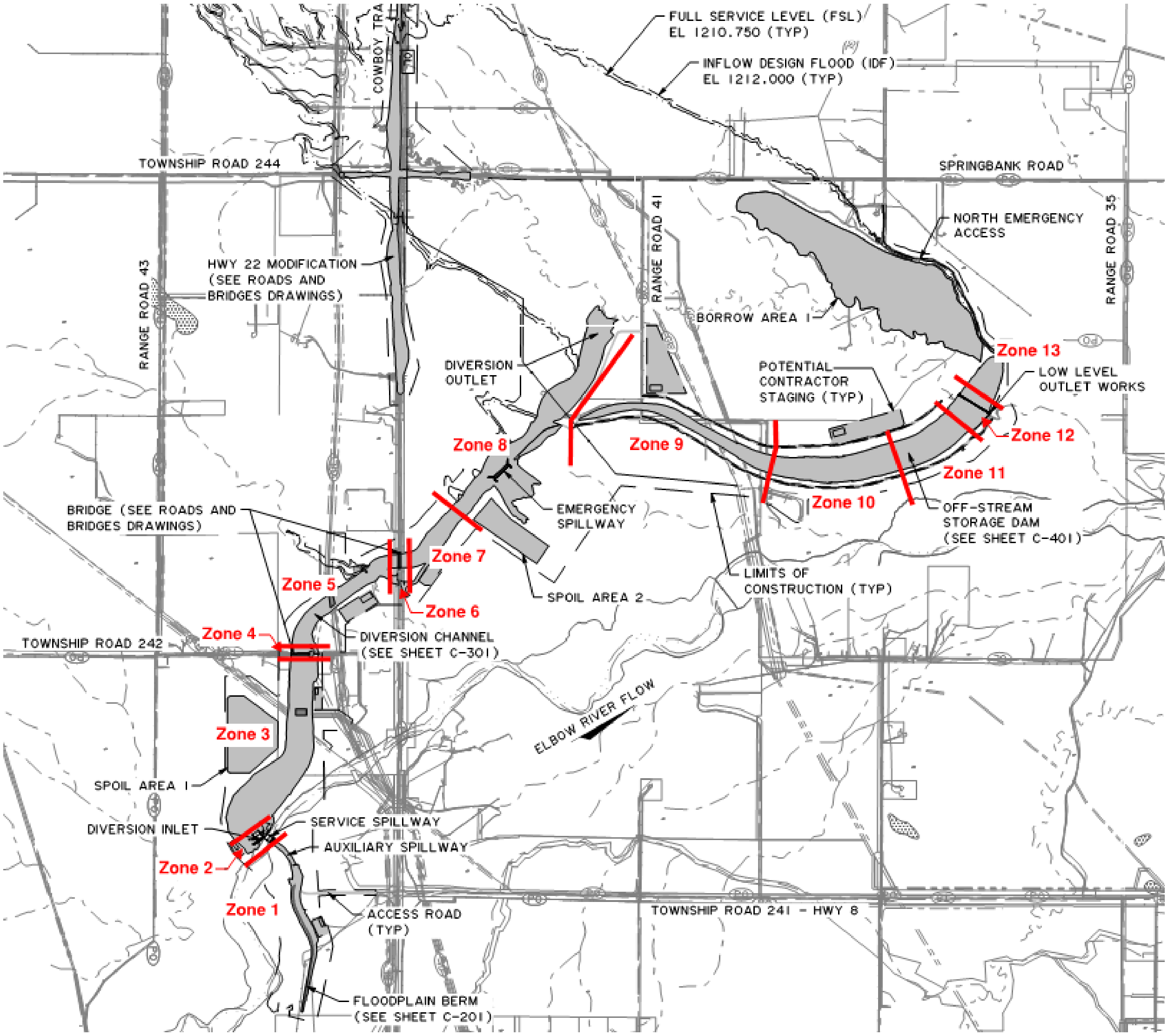


Figure IR523-2 Earthwork Construction Zones

The subsurface soils are glaciogenic units consisting mostly of glacial lacustrine and glacial till soils. The following is a description of each soil unit in the associated zones of the diversion channel:

- Glacial lacustrine (GL) is olive green to brown, intermediate to high plasticity, and lean to fat clay. Particle size distribution testing indicated that the GL is typically consists of between 50% and 70% clay-sized particles with a subordinate (typically 30% to 50%) of silt-sized particles. The thickness of the GL within Zone 3 ranges from 0 m to 3 m in thickness. GL was not present within half of the borings. In Zones 4 to Zone 8, GL ranged continuously from 2 m to 5 m in thickness, except for along a ridge near Station 14+000 where no GL was present.
- Glacial till (GT) is brown to grey, low to intermediate plasticity, lean clay with sand and gravel. There is large variability and inconsistency in the thickness of the GT layer. The thickness of the GT within Zone 3 to 5 typically ranges from 9 m to 20 m. In Zone 7 and Zone 8, the GL ranged from 2 m to 9 m.

It is anticipated that the majority of the soil excavated from the diversion channel can be used as Impervious Fill Zone 1A or Random Fill Zone 2A(1). This comprises GT and GL.

Bedrock excavated from the channel may be suitable for Random Fill Zone 2A(2) and Random Fill Zone 2A(3).

It is assumed that 35% of rock and 10% of soil excavated will be unsuitable for use as fill.

c. Excavation of the diversion channel will result in the following estimated quantities:

- top soil, 0.2 million m^3
- clay (GL/GT), 4.1 million m^3
- rock, 1.0 million m^3

The off-stream dam will require the following estimated quantities:

- top soil, 0.07 million m^3
- Zone 1A, 1.5 million m^3
- Zone 2A, 3.4 million m^3
- Zone 3A Fine Filter, 0.17 million m^3

REFERENCES

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario

CGS (Canadian Geotechnical Society). 2007. Canadian Foundation Engineering Manual. Fourth Edition. Canadian Geotechnical Society. Richmond, BC.

Question 524

Volume 1, Section 3.2.5.1, Page 3.14
Volume 1, Section 3.2.5.1, Figure 3-8, Page 3.14
Volume 1, Section 3.2.5.4, Page 3.17

The foundation conditions including any variances that may occur underlying the offstream dam are not described. In particular, the differences in soil conditions between the majority of the dam foundation and the areas at the dam abutments and No Name Creek are not described.

a. **Provide a discussion on foundation materials underlying the dam and saddle dyke including what seepage control requirements are necessary and how these requirements might vary across the bottom of the valley and abutment areas. Show or describe the typical dam cross section in the vicinity of the low level outlet near the No Name Creek channel.**

b. **Explain why a nominal cutoff trench would be sufficient for part of the dam when a deep cutoff trench is required for the floodplain berm on the Elbow River and drains are proposed near the low level outlet structure.**

Response 524

a. The subsurface soils beneath the proposed dam and saddle dyke are glaciogenic units consisting mostly of glacial lacustrine and glacial till soils. The soils description follows protocols of the geotechnical program, which may differ from protocols in soils science. The following is a description of each soil unit and location.

   - Glacial lacustrine (GL) is olive green to brown, intermediate to high plasticity, and lean to fat clay. Particle size distribution testing indicated that GL typically comprises between 50% and 70% clay-sized particles with a subordinate (typically 30% to 50%) of silt-sized particles. The thickness of GL typically ranges from 2 m to 6 m at the western and eastern ends of the dam. In the center of the dam, GL is thicker, ranging from 8 m to 15 m in thickness. GL is not present near the unnamed creek.
   - Glacial till (GT) is brown to grey, low to intermediate plasticity, and lean to fat clay with sand and gravel. There is large variability and inconsistency in the thickness of the GT layer. Within the off-stream dam footprint, the thickness ranges from 1 m to 15 m, with the thickest layers generally in the center portion of the off-stream dam.
   - A layer of sand and gravel soils were encountered in the low-lying area of the unnamed creek. The sands are dense, brown, clayey sand with gravel and silty sand with gravel. The gravel is described as very dense, clayey and silty gravel with sand. The thickness of these soils ranges from 1 m to 7 m in the area of the unnamed creek.

The foundation materials immediately underlying the dam (GL or GT) have vertical hydraulic conductivity ranging from 3.0 x 10^{-11} m/s to 3.0 x10^{-10} m/s. Based on these low permeability values and the temporary nature of the impoundment of water in the reservoir, seepage protection measures are not required under most of the dam embankment. For the areas near the outlet structure, two rows of 1 m drains are extended to the sand and gravel soil layers and connected to the filter and drainage blanket. The drains are included to relieve potential seepage gradients through the sand and gravel soil layers encountered near the unnamed creek, which will be the flow path for water released from the reservoir to return to the river. No direct connection has been identified between the reservoir and this layer. These soils were overlain by a clay till soil in each of the borings as part of the geotechnical investigation.

Seepage analyses were performed to determine the steady-state phreatic surface at the IDF pool elevation of water in the reservoir (El. 1,212 m). The headwater pool was modeled as a head boundary condition at Elevation 1,212 m. The tailwater pool was modelled as a head boundary condition to replicate the fall in the groundwater from the dam location to the river water elevation.

The average gradient in the upper 1 m of the foundation soils just downstream of the toe of the dam was calculated as 0.93. The factor of safety against piping due to heave was greater than 9, indicting no further mitigation measures are required.

Results of the seepage analysis for the typical cross section underlain by GL/GT is displayed in Figure IR524-1.

Results of the seepage analysis for the cross section through the unnamed creek is displayed in Figure IR524-2.

b. It has been determined a cutoff trench at the floodplain berm is not needed.

However, design analysis indicates seepage control beneath the off-stream dam is needed in the area where the sand and gravel occur in the foundation. Drains are considered the most appropriate method for this control.

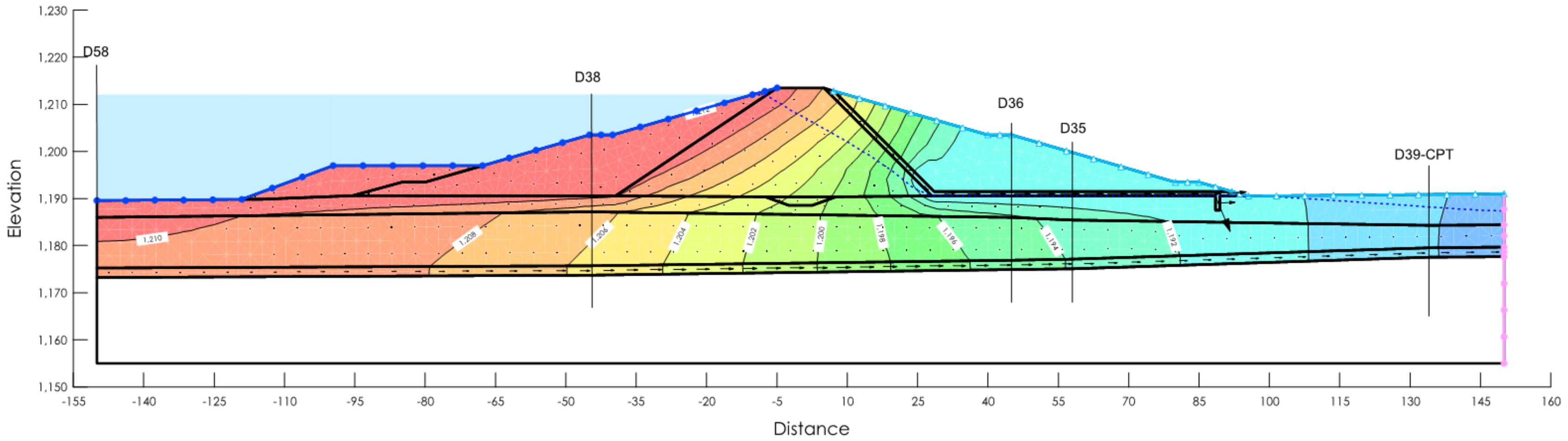


Figure IR524-1 Seepage Gradients for Sta. 22+925

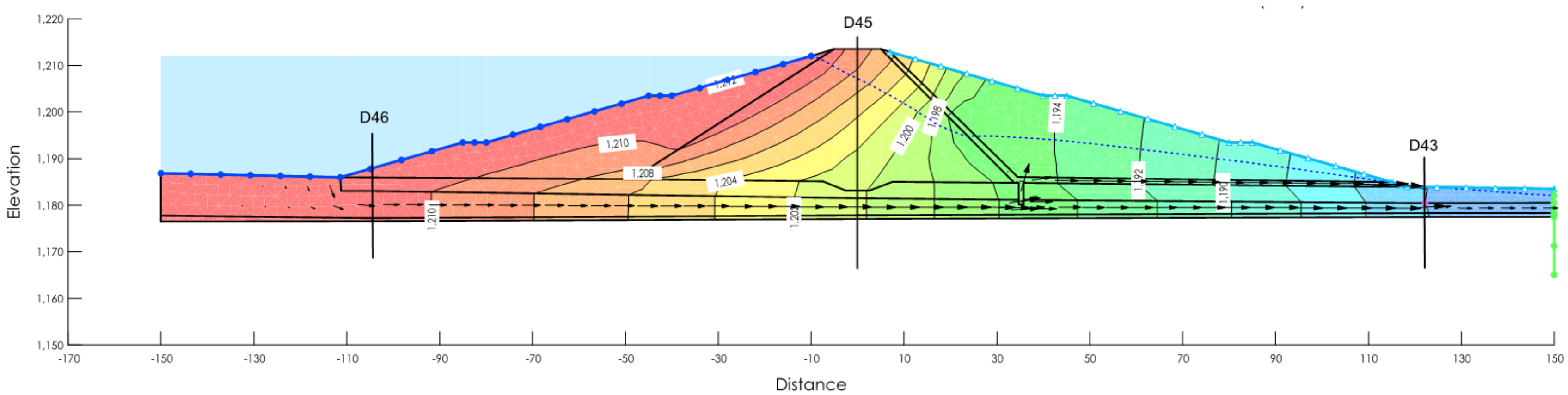


Figure IR524-2 Seepage Gradients for Sta. 23+175

Question 525

Volume 1, Section 3.2.5.1, Page 3.15

The composition of the dam includes an impervious core, random fill shells and a sand/fine filter internal drain. Quantities of the various construction materials are not provided. Other embankments such as the floodplain berm are described similarly with the exception that some areas of the embankments are also protected with riprap on the upstream slope.

a. **Provide a summary of the field and lab testing that has been performed to determine the suitability of the materials.**

b. **Provide the characteristics/geotechnical properties and design parameters of the in-situ and construction materials and describe their suitability for use as construction materials.**

c. **Clarify if the earthfill specifications for impervious and random fill will permit the use of high plastic soils? If so, how will the CWMS be modified to account for the use of high plastic soils?**

d. **Clarify if the stability analyses for the excavations and embankments account for the use of high plastic materials in the embankments and foundation soils?**

e. **Discuss availability of various fill materials from excavations and requirements for sorting, moisture conditioning or other processing requirements.**

Response 525

a. The initial geotechnical exploration was completed between March 21, 2016 and August 25, 2016 to characterize insitu soils that will form the foundations for structures and a source of construction materials for the embankment fills. The exploration consisted of 135 boreholes and 20 cone penetration tests advanced within the PDA. Subsequent laboratory testing included 390 Atterberg limits tests, 374 grain size analysis tests, 79 consolidated-undrained triaxial shear strength tests, 36 hydraulic conductivity tests, 19 consolidation tests, and 28 bedrock unconfined compressive strength tests.

b. The insitu soils are characterized primarily as glacial-lacustrine (GL) clays and silts or glacial till (GT). The GL was encountered beneath the dam footprint and the diversion channel within the exploratory boreholes. It was always encountered at the top of the glaciogenic sequence, near the existing ground level. Standard penetration test (SPT) N values indicated that the density of this unit was ‘stiff to hard’ with typical values between 15 and 25. The GL was typically encountered as olive brown to brown, medium to high-plastic, clay and silt. The thickness ranged between 0.5 m and 11 m. Index testing indicates that this unit has a medium to high plasticity clay with silt. Clay was the dominant fraction, typically comprising 50% and 70%.

The GT was encountered beneath the GL within the dam footprint and the eastern portion of the diversion channel. The GT was typically encountered as an olive brown to brown, medium plastic, clay and silt with increased sand content with depth. Silt was the dominant fraction, typically comprising 35% and 50%. The clay content was more variable, with values between 10% and 50%. The sand content ranged between 10% and 30% and the gravel content ranged from 10% to 20%.

Both GL and GT soil types are suitable for use as construction materials. The recommended soil properties of insitu and placed soil types are listed in Table IR525-1.

Table IR525-1 Recommended Field Soil Properties

Soil Type	k_v (m/sec)	k_h (m/sec)	c_v NC (mm²/sec)	c_v OC (mm²/sec)	C_c	C_r	e_0
Insitu GL	5.0E-10	2.5E-09	0.20	0.29	0.17	0.042	0.637
Insitu GT	5.0E-10	1.5E-09	0.27	0.36	0.13	0.028	0.538
Placed GL	1.0E-09	5.0E-09	0.38	0.58	0.12	0.035	0.607
Placed GT	5.0E-10	2.5E-09	0.51	0.72	0.09	0.023	0.524
Sand Drain	3.0E-06	3.0E-06	-	-	-	-	-
Rock Fill	3.0E-08	3.0E-08	-	-	-	-	-

Recommended strength properties of insitu and placed soil types are listed in Table IR525-2.

Table IR525-2 Material Parameters Recommended for Analysis

Material Name	Drained Strength		Undrained Strength	
	Cohesion (kPa)	Friction Angle (degrees)	Cohesion (kPa)	Friction Angle (degrees)
Embankment Shell (GL)	0	24	25	15
Embankment Core (GT)	0	28	80	19
Foundation Glacial Lacustrine	0	23	Spatial Mohr-Coulomb based on DSS Testing and S_u/σ_p Method	
Glacial Till	0	27	60	19
Drain	0	33	-	-
Rock Toe	0	33	-	-
Weathered Bedrock	0	35	-	-

c. The earthfill specifications for impervious and random fill will permit the use of high plastic soils. The supplemental language allowing for the use in the embankment zones follows:

- For Impervious Fill Zone 1A

  The embankment core may include high plastic glacio-lacustrine clay (CH) soils compacted to a minimum of 95% of standard Proctor value and placed with an allowable moisture content ranging from optimum moisture content to +4%. The recommended minimum liquid limit is 50%, with a maximum particle size of 75 mm.

- For Random Fill Zone 2A(1)

  The selected soil embankment may include moderate to highly plastic glacio-lacustrine clay soils or glacial till clay soils placed in the embankment shell and compacted to a minimum of 95% of standard Proctor value and placed in maximum 250 mm lifts with an allowable moisture content ranging from -2% to +2% of optimum moisture content.

d. The stability analyses account for the use of high plastic materials in the embankments and foundation soils.

e. Excavation of the diversion channel will result in the following estimated quantities:

- top soil, 0.2 million m^3
- clay (GL/GT), 4.1 million m^3
- rock, 1.0 million m^3

Based on the field exploration, sufficient material is within the diversion channel for construction of the dam. Through proper sequencing, the construction contractor should be able to place material directly from the excavation to the embankment without substantial sorting or “double handling.” Moisture conditioning, through mechanical means, may be required depending on the drainage of the excavation, care of water, weather conditions and placement area. This will likely occur within the dam placement footprint or in the immediate vicinity of the dam.

Question 526

Volume 1, Section 3.2.6, Figure 3-10, Page 3.17

Alberta Transportation states that *The low-level outlet structure (Figure 3-10) consists of an approach channel, discharge gate, gatehouse, discharge conduit and outlet channel. The gate is operated locally by the gatehouse.*

a. Discuss the structural design of the main structure components including design approach, loading conditions and parameters.

b. Where applicable, discuss the stability of the main structure components including loading conditions, parameters, and target and actual factors of safety.

c. Discuss instrumentation to monitor performance (e.g. deformations).

d. Discuss anticipated quantities of construction materials required.

Response 526

a-b. The outlet structure includes the following components:

- inlet gate structure (concrete approach apron, the concrete inlet structure with trash rack framing, the discharge gate, transition section)
- discharge conduit (concrete arched conduit through the dam)
- concrete energy dissipation structure and the designed channel

Each component was assessed for the applicable imposed loads to which they may be subject per the governing code criteria. These components include the inlet and gate structure, discharge conduit, and the designed channel. Secondary structures (such as gate hoists, gate operating mechanisms, ladders) will be designed to work with the supporting structure and to function as required by the code under which they fall. Each structure was evaluated, as applicable, for global stability, strength, and serviceability.

DESIGN APPROACH

Global stability was calculated based on a summation of forces to determine resultant location, foundation bearing pressures, and sliding resistance along the concrete/soil interface. A 3D finite element method (FEM) model was created to validate stability calculations and identify areas of stress concentration of the inlet/gate structure. Mathcad and Excel sheet templates were developed to assess the stability of the inlet structure and trash rack frame, transition, conduit, and stilling basin structures.

Strength evaluation was used to verify size of critical components such as walls, aprons, gate operating mechanism reaction ledge, training walls, and identify location of primary expansion/contraction joints. Strength evaluations also provided design values for sizing concrete reinforcement, joint doweling, gate anchorage, and gate hoist supports. The same 3D FEM model used for stability of the intake/gate structure were used to generate shear and moment values for strength design. A combination of Mathcad and Excel sheet templates and commercial software were used for design to meet strength requirements.

Serviceability includes limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR). The same 3D FEM used for strength evaluation was the primary method used to evaluate serviceability of the intake/gate structure. A combination of Mathcad and Excel sheet templates and commercial software were used to assess serviceability requirements of the inlet structure/trash rack frame, transition, conduit, and stilling basin.

Loading Condition, Parameters And Stability Results

Inlet/Gate Structure

The loading conditions that were considered in the stability analysis of the inlet gate structure are summarized in Table IR526-1.

Table IR526-1 Load Cases and Acceptance Criteria for Inlet Gate Structure Stability Analysis

Load Case	Description	Loading	Sliding Safety Factor*		Resultant Location**	Bearing Stress**	Floatation Safety Factor***
			Drained	Undrained			
LC01	Usual 1: Empty Reservoir + Snow	Dead + Earth + Uplift + Wind + Snow	1.5	3.0	Middle 1/3	<ALLOWABLE	1.5
LC02	Usual 2: Empty Reservoir	Dead + Earth + Uplift + Wind	1.5	3.0	Middle 1/3	<ALLOWABLE	1.5
LC03	Unusual 1: Construction	Dead + Earth + Uplift + Surcharge	1.3	2.0	Middle 1/3	<ALLOWABLE	1.3
LC04	Unusual 2: 10-Y Flood	Dead + Earth + Hydro + Uplift	1.3	2.0	Middle 1/3	<ALLOWABLE	1.3
LC05	Extreme 1: 10-Y Flood + Impact	Dead + Earth + Hydro + Uplift + Impact	1.1	1.3	Middle 1/2	<ALLOWABLE	1.1
LC06[1]	Extreme 2: Earthquake	Dead + Earth + Uplift + EQ[1]	n/a				

Table IR526-1 Load Cases and Acceptance Criteria for Inlet Gate Structure Stability Analysis

Load Case	Description	Loading	Sliding Safety Factor*		Resultant Location**	Bearing Stress**	Floatation Safety Factor***
			Drained	Undrained			
LC07	Post Earthquake	Dead + Earth + Uplift	1.1	1.3	Within Base*	<ALLOWABLE	1.1

NOTES:
1 Response Spectrum Analysis; EQ=max [24 combinations of EQx, EQy, EQz]
* CDA (2007; Section 6.7); Alberta Transportation (2004; Section 8.4)
** Alberta Transportation (2004; Section 8.4)
*** Alberta Transportation (2004; Section 8.5)

Stability analysis results for the intake structure are summarized in Table IR526-2. The intake structure meets the acceptance criteria for sliding, resultant location, floatation, and bearing capacity for the analysis load cases.

Table IR526-2 Summary of Stability Analysis Results for the Inlet Gate Structure

Load Case	Loading Type	Sliding Safety Factor		Floatation Safety Factor	Resultant Location	Bearing Safety Factor [1]
		Drained	Undrained			
LC01	Usual 1: Empty Reservoir + Snow	5.9	13.2	10.8	Middle Third	1.1
LC02	Usual 2: Empty Reservoir (Summer)	5.8	13.2	10.6	Middle Third	1.1
LC03	Unusual 1: Construction	3.1	7.0	10.6	Middle Third	1.1
LC04	Unusual 2: 1:10 year flood	5.4	16.2	2.0	Middle Third	1.6
LC05	Extreme1: 1:10 year flood + 1.5 kN Impact	5.3	14.5	2.0	Middle Half	1.2
LC07	Post-Earthquake	5.8	4.6	10.6	Middle Third	1.1

NOTE:
1 Except for LC06, bearing is compared to allowable. Bearing stresses from LCO6 are compared to the ultimate capacity.

DISCHARGE CONDUIT

Load cases for the discharge conduit were taken from Engineering Memorandum 1110-2-2902 *Conduits, Culverts, and Pipes* and Federal Emergency Management Agency *Technical Manual: Conduits through Embankment Dams*. The conduit was analyzed at the intake for floatation for two load cases described in Table IR526-3.

Table IR526-3 Load Cases and Acceptance Criteria for Discharge Conduit Stability Analysis

Load Case	Description	Loading	Floatation Safety Factor*
LC01	Unusual: 1;10 year	Dead + Earth + Hydro + Uplift	1.3
LC02	Extreme: maximum reservoir	Dead + Earth + Hydro + Uplift	1.1
NOTE: * Require minimum value as per Alberta Transportation (2004; Section 8.5)			

The stability analysis results for the conduit are summarized in Table IR526-4. The conduit meets the floatation acceptance criteria.

Table IR526-4 Summary of Stability Analysis Results for the Discharge Conduit

Load Case	Loading Type	Floatation Safety Factor
LC01	Unusual: 1:10 year	1.5
LC02	Extreme: maximum reservoir	1.1

Outlet Structure

The load cases used for the stability analyses are described in Table IR526-5.

Table IR526-5 Load Cases and Acceptance Criteria for Outlet Structure Stability Analysis

Load Case	Description	Loading	Sliding Safety Factor*	Resultant Location**	Bearing Stress**	Floatation Safety Factor***
LC01	Usual: Dry Condition	Ws + Wsoil + Fa + Fp	1.5	Middle 1/3	<ALLOWABLE	1.5
LC02	Unusual: 10-Year Flood	Ws + Wsoil + Fa + Fp + Fd + Ww + U	1.3	Middle 1/2	<ALLOWABLE	1.3
LC03	Unusual: Rapid Gate Closure	Ws + Wsoil + Fa + Fp + U	1.3	Middle 1/2	<ALLOWABLE	1.3
LC04	Unusual: Construction/ Maintenance	Ws + Wsoil + Fa + Fp + Fs	1.3	Middle 1/2	<ALLOWABLE	1.3
LC05	Extreme: Maximum Flow	Ws + Wsoil + Fa + Fp + Fd + Ww + U	1.1	Within Base	<ALLOWABLE	1.1

Table IR526-5 Load Cases and Acceptance Criteria for Outlet Structure Stability Analysis

Load Case	Description	Loading	Sliding Safety Factor*	Resultant Location**	Bearing Stress**	Floatation Safety Factor***
LC06[1]	Extreme: Earthquake	Ws + Wsoil + Fa + Fp + Qh + Qv	1.1	Within Base	<ALLOWABLE	1.1

NOTES:
* CDA (2007' Section 6.7); Alberta Transportation (2004; Section 8.4)
** Alberta Transportation (2004; Section 8.4)
*** Alberta Transportation (2004; Section 8.5)
1 Seismic coefficient analysis for three load combination as follows:
Case6a = 1kh + 0kv
Case6b = 1kh + 0.3kv
Case6c = 0.3kh + 1kv

The stability analysis results for the outlet structure are summarized in Table IR526-6.

Table IR526-6 Summary of Stability Analysis Results for the Outlet Structure

Load Case	Loading Type	Sliding Safety Factor	Resultant Location	Bearing Stress Safety Factor	Floatation Safety Factor
LC01	Usual: Dry Condition	10.1	Middle Third	2.3	N/A
LC02	Unusual: 1:10 year flood	5.9	Middle Third	2.7	3.3
LC03	Unusual: Rapid Gate Closure	3.9	Middle Third	3.6	2.1
LC04	Unusual: Construction/Maintenance	5.1	Middle Third	2.4	N/A
LC05	Extreme: Maximum Flow	5.1	Middle Third	2.8	2.6
LC06(a)	Extreme: Earthquake	1.6	Middle Third	3.5	N/A
LC06(b)		1.6	Middle Third	3.6	36.6
LC06(c)		3.3	Middle Third	3.5	11.0

c. Monitoring of structures may include the following:

- Surface measuring points would be located on top of retaining and training walls using embedded stainless-steel carriage bolts or cap screws located 150 mm on either side of a vertical formed joint. This provides measurable differential movement between walls at the joint and joint opening. In addition, wall deformations and block movement can be detected based on intermittent surveys of the measuring points.
- Water measurement weirs would be located at end of drain pipe outfalls that are used behind structure retaining walls. Sometimes measuring weirs are located within dry wells that serve as junctions for retaining wall drains.

Settlement, deformation and block movement can be detected based on intermittent surveys of the measuring points and other surface monuments.

d. Construction of the outlet structure will require approximately 3,852 m^3 of reinforced structural concrete, 135 m^3 of mass concrete and 8,800 kg of structural metal framing. In addition, the construction will include installation of the gates, vent piping, hydraulic operator and controls.

REFERENCES

Alberta Transportation. 2004. Water Control Structures Selected Design Guidelines. Alberta Transportation, Civil Projects Branch. Edmonton, AB.

CDA (Canadian Dam Association). 2007. Dam Safety Guidelines (Revised 2013). Canadian Dam Association. Toronto, Ontario

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Question 527

Volume 1, Section 5.1.2, Table 5-3, Page 5.4

Alberta Transportation states that *Sediment deposition simulations were performed to evaluate potential effects on water surface elevations and freeboard established appropriately* (at the service spillway). There is a discussion of the uncertainty involved in sediment transport calculations, and in particular the differences between various sediment transport models, but the uncertainty related to selection of a sediment size or gradation is not discussed. The sediment transport calculations rely on a single composite grain size distribution based on four bar samples.

a. Provide information on the sample locations relative to the site and the local morphology (e.g. where on the bar).

b. Provide information on the sample size.

c. Provide a plot showing individual gradation curves for all four samples.

d. Discuss the uncertainty inherent in the mathematical and physical models related to uncertainty and variability of bed material gradations, and the consequences of that uncertainty for the expected performance of the diversion system.

Response 527

a. Sediment samples were taken from depositional bars on Elbow River at the following locations:

- Sample 1–river right, bar upstream of the floodplain berm
- Sample 2–river left, bar at diversion entrance
- Sample 3–river right, bar downstream of diversion and floodplain berm
- Sample 4–river right, bar upstream of Highway 22

Figure IR527-1 displays the location of each bar sample.

Selection of the sample location within each bar followed procedures described in Rosgen (2006). Samples were taken from the downstream 1/3 of the bar, approximately halfway between the thalweg invert elevation and bankfull elevation.

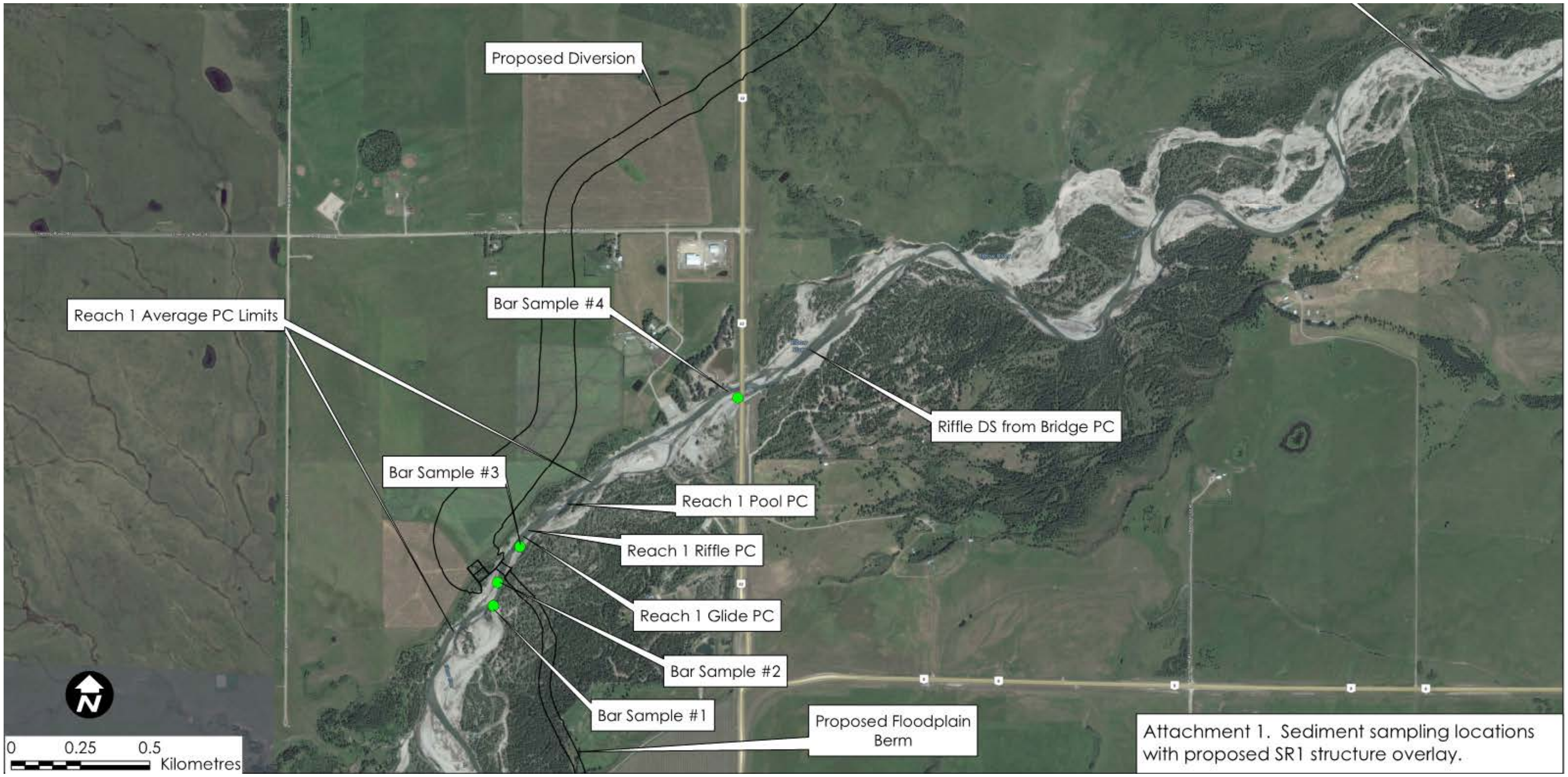

Figure IR527-1 Sediment Sample Locations in Elbow River

b. Volumetric sample collection followed procedures described in Rosgen (2006). Bar material was excavated within a bottomless 5-gallon bucket (approximately 541 cm^2) to a depth 2X the intermediate axis of the largest bar particle (14 cm to 24 cm) and retained for sieve analysis. Sample weight ranged from 16.2 kg to 20.3 kg (see raw data in Table IR527-1).

Table IR527-1 Sediment Sample Raw Data from Bar Location in the Elbow River

BAR SAMPLE RAW DATA

	Net Weight (kg)				
Sieve (mm)	Sample 1	Sample 2	Sample 3	Sample 4	Composite
63	2.5	1.0	0.8	1.7	6.0
31.5	1.9	5.5	5.1	7.9	20.4
16	2.7	4.6	5.1	1.3	13.7
8	2.1	2.6	5.5	4.4	14.7
4	1.4	1.1	1.5	0.2	4.2
2	0.9	0.5	0.7	0.2	2.1
Pan	2.0	0.3	1.9	1.0	5.3
Total Weight (kg)	16.2	18.3	24.1	20.3	70.4
D16 (mm)	3.2	11.4	7.2	11.2	7.6
D35 (mm)	12.8	22.7	14.1	30.8	15.1
D50 (mm)	25.2	32.2	23.0	43.4	26.0
D84 (mm)	76.4	76.0	66.6	80.9	61.0
D95 (mm)	85.8	106.3	103.3	100.9	99.9
D100 (mm)	110.0	120.0	120.0	110.0	120.0
Silt/Clay (%)	0	0	0	0	0
Sand (%)	13	1	8	5	7
Gravel (%)	57	78	75	70	79
Cobble (%)	30	20	17	25	14
Boulder (%)	0	0	0	0	0
Bedrock (%)	0	0	0	0	0

	Size (mm)	wt (kg)	Size (mm)	wt (kg)	Size (mm)	wt (kg)	Size (mm)	wt (kg)	Size (mm)	wt (kg)
Largest Surface Particles	90.0	0.9	120.0	1.9	120.0	2.1	110.0	1.7	120.0	1.9
	110.0	1.8	100.0	0.9	70.0	1.2	100.0	1.9	120.0	2.1

c. The four sample gradations are shown in Figure IR527-2.

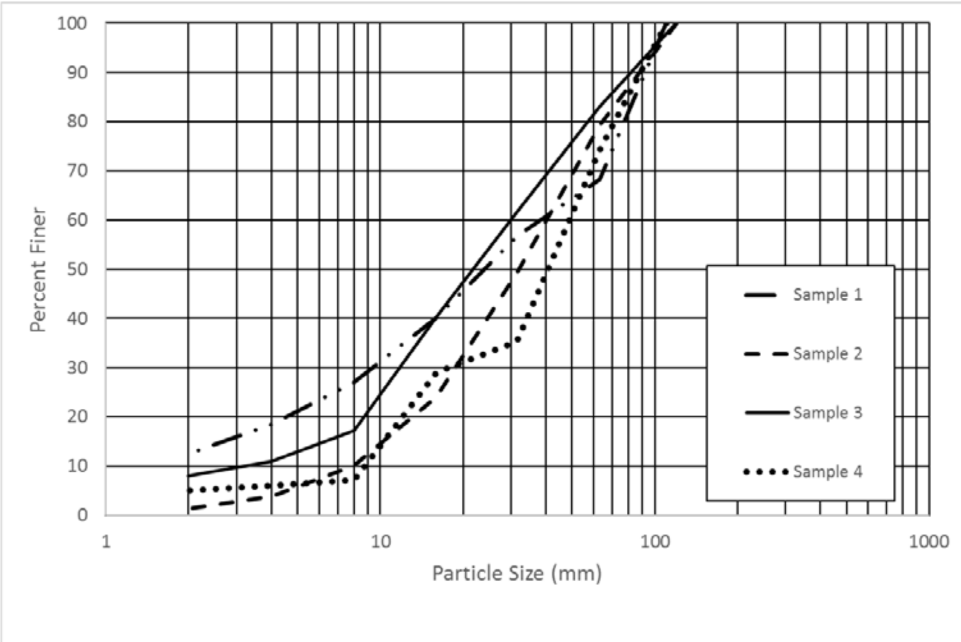

Figure IR527-2 Variation of Sediment Sample Particle Sizes

d. Considerable uncertainty is inherent in the representation of sediment transport with mathematical and physical models. Multiple sensitivity analyses were performed to evaluate the impacts of sediment size and channel roughness on modelling the performance of the diversion structure under loading.

Three mathematical relationships for bedload transport capacity were considered: Meyer-Peter and Müller (1948), Bagnold (1980), and Wilcock and Crowe (2003). Each of these relationships are sensitive to the mean particle size and channel roughness. Figure IR527-3 presents the sediment transport rating curve for each of the three methods, assuming the composite sediment gradation (median particle size (D50) = 26 mm) and using an average channel roughness value (n) of 0.045 for each of these models.

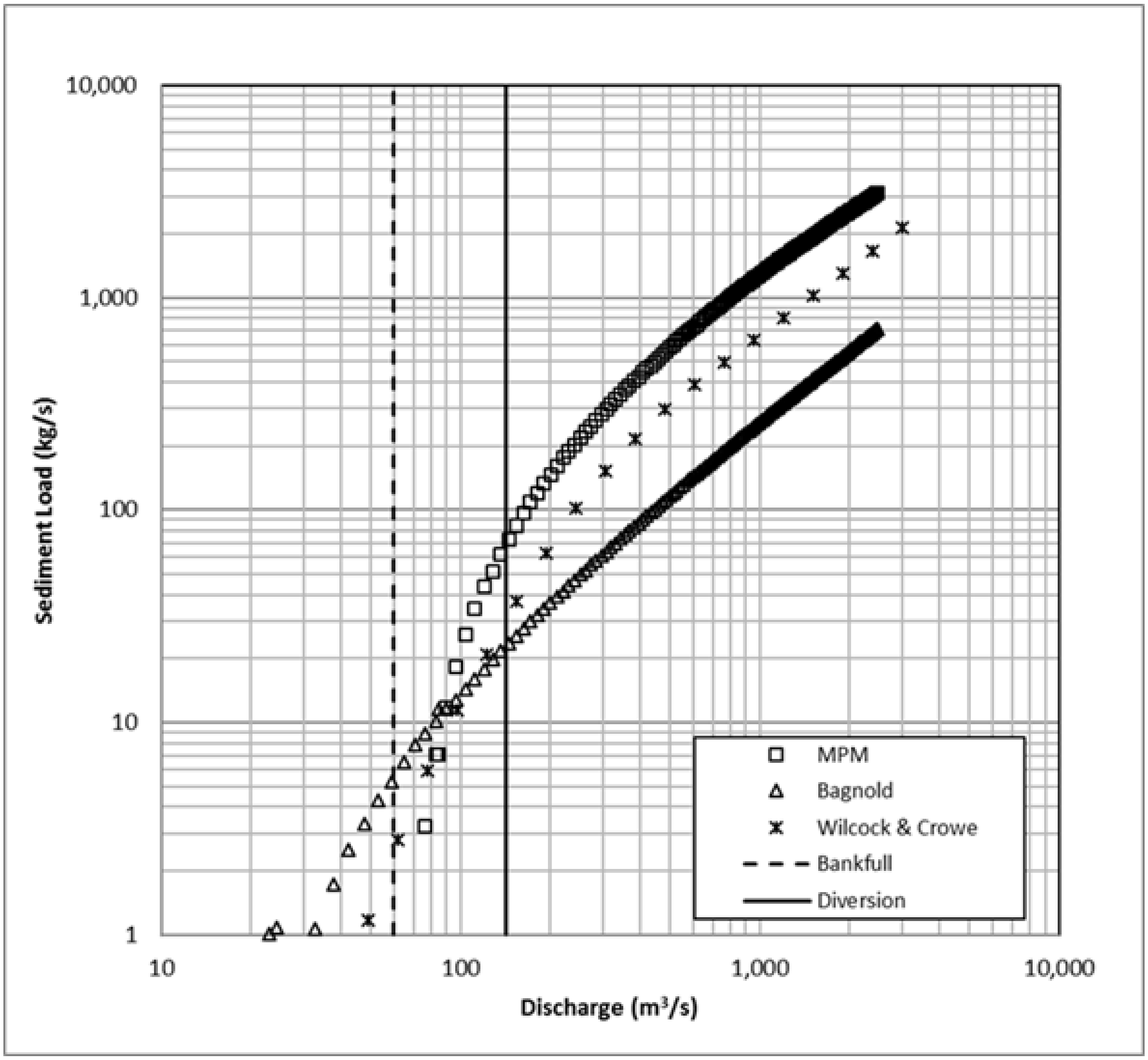

Figure IR527-3 Bedload Transport Rating Curves

The Meyer-Peter and Müller (MPM) method was selected for use in design because it represents the upper bound of transport rates. Further sensitivity analyses were then performed to assess the potential effects of grain size distribution and variability on transport rates and 2D model performance. Figure IR527-4 shows the change in transport rate based on the median particle diameter ranging from 20 mm to 30 mm. From the figure, a finer sediment distribution will result in higher sediment transport rates and a larger sediment distribution will result in lower sediment transport rates.

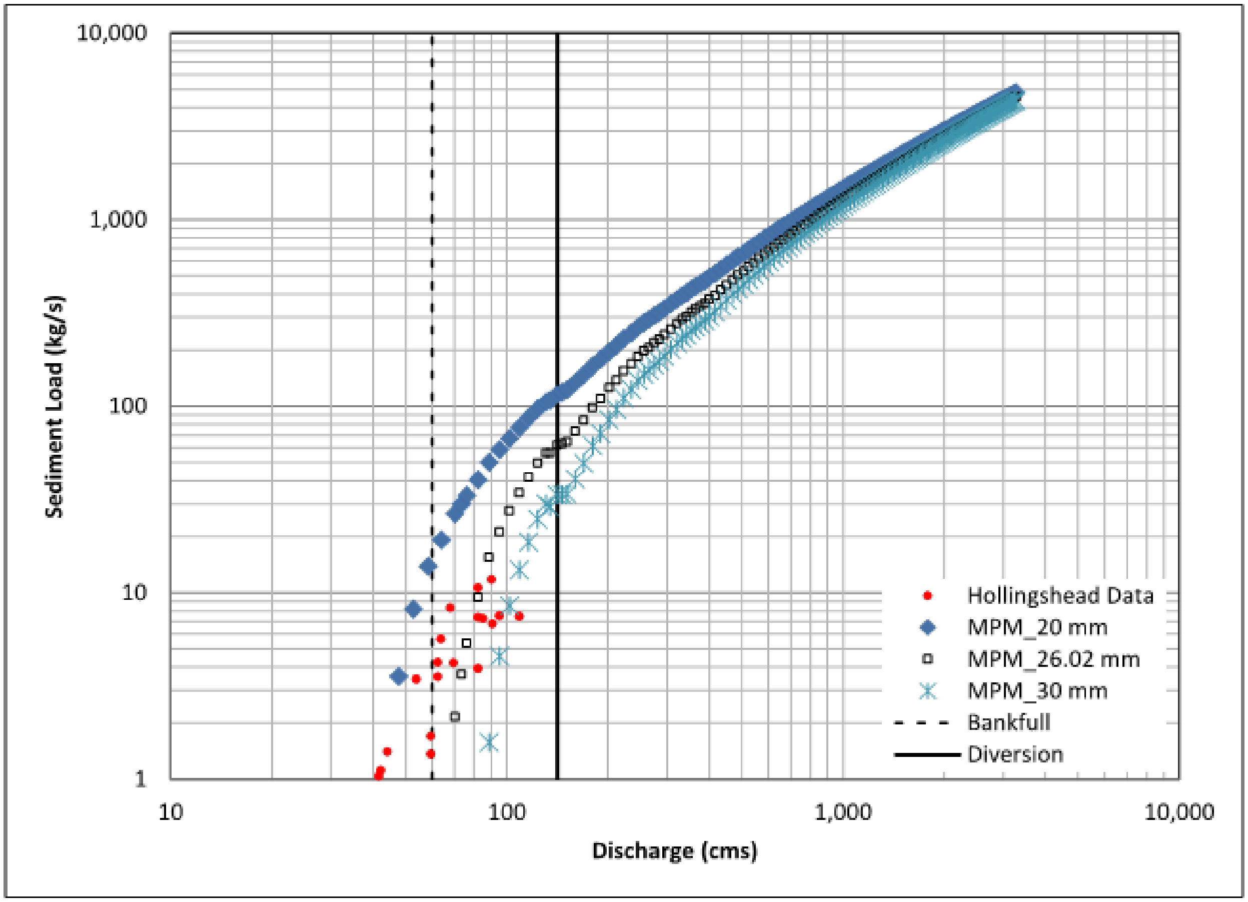

Figure IR527-4 MPM Bedload Transport Rating Curves for Varying Particle Size

The 2D model evaluated the potential effects of higher sediment transport rates (finer particle size) on the operations of the diversion system. The lower transport rates (larger particle size) would result in beneficial performance of the system because nearly all the bedload would be deposited in the river upstream of the diversion system (this was used for the diversion numerical model simulation).

Modelling indicates that sediment with a D50 of 20 mm is transported more readily into the diversion channel. This results in a faster decay of the diversion capacity than the D50 = 26 mm simulation. However, after two days of sediment loading at 760 m^3/s, the diversion capacity (503 m^3/s) remains greater than the minimum required diversion rate (480 m^3/s), without adjusting the gate heights on the service spillway.

Considering the conservative transport rate and performance under the various size distributions, the diversion structure is expected to meet the flood control performance requirements for floods up to the 2013 flood.

REFERENCES

Bagnold R.A. 1980. An Empirical Correlation of Bedload Transport Rates in Flumes and Natural Rivers. Royal Society of London Proceedings, A372: 453-473.

Meyer-Peter and Muller. 1948. Formulas for Bed-Load Transport. International Association of Hydraulic Research, 2nd Congressional Proceeding, Stockholm, pp. 39-64.

Rosgen, D.L. 2006. Watershed Assessment of River Stability and Sediment Supply (WARSSS). Fort Collins, CO: Wildland Hydrology Books.

Wilcock, P., and J. Crowe. 2003. Surface-Based Transport Model of Mixed-Size Sediment. Journal of Hydraulic Engineering. February 2003. Pp 121-128.

Question 528

Volume 1, Section 5.1.2, Table 5-4, Page 5.4
Preliminary Design Report (draft) Section 10.1.2, Page 151
Preliminary Design Report (draft) Section 10.1.2, Figure 39, Page 152

Alberta Transportation states that *adequate freeboard has been provided in the design to account for inflows from the probable maximum flood and potential wave run-up* (for the off-stream dam).

Section 10 of the draft Preliminary Engineering Report states that *Inflow from the Diversion Channel is based on an assumed failure of the Diversion Inlet gates during a PMF on the Elbow River.*

a. **Provide details of the freeboard calculation including routing and wind-wave analysis.**

b. **The starting reservoir elevation (on Figure 39) is said to account for sedimentation and local runoff. Does it also account for stored water remaining after passage of the antecedent rainfall event? Explain why or why not. Provide a complete figure showing inflow and outflow hydrographs and reservoir levels throughout the antecedent event and the PMF.**

c. **What return period event was assumed for estimation of the local runoff into the reservoir during passage of the PMF hydrograph? What was the rationale for that selection?**

d. **What is the reason for the horizontal portion of the inflow hydrograph in the Preliminary Design Report between hour 18 and hour 26? If it is to account for some duration of proper operation of the diversion structure gates, what is the rationale for the selection of that duration?**

e. Why is the reservoir inflow shown on Figure 39 consistently slightly less than the diversion channel inflow even when the emergency spillway is not operating?

Response 528

a. Freeboard criteria for the reservoir and dam were determined based on the CDA (2007). Freeboard is based on:

- hydrologic routing of probable maximum flood (PMF)
- evaluation of wind generated wave height
- setup and runup with consideration given to embankment settlement
- land-slide generated waves
- hydrologic uncertainty
- spillway and low-level outlet works malfunction

Flood routing of the PMF relies on several numerical models. Operation of the diversion structure and the resultant inflow hydrograph to the diversion channel was simulated using the United States Army Corps of Engineers (USACE) HEC-ResSim, version 3.1, software package. Routing of the inflow hydrograph through the diversion channel, reservoir and emergency spillway used the USACE HEC-RAS version 5.03, software package.

The HEC-ResSim model applies a series of operating rules and hydraulic rating curves to calculate the flow split that would advance downstream on Elbow River and the flow to be diverted to the diversion channel. The following rules were applied:

- Diversion of flow to the channel begins when Elbow River flow exceeds 160 m^3/s.
- Flow in excess of the 160 m^3/s would be diverted to the diversion channel until the target diversion rate of 480 m^3/s is achieved.
- Service spillway gates on the river channel will lower to maintain a steady diversion rate of 480 m^3/s into the reservoir and flows downstream in Elbow River would exceed 160 m^3/s.
- When the service spillway gates are completely open, flow into the diversion channel may exceed 480 m^3/s as the flow in the river increases.

Figure IR528-1 presents the calculated inflow hydrograph.

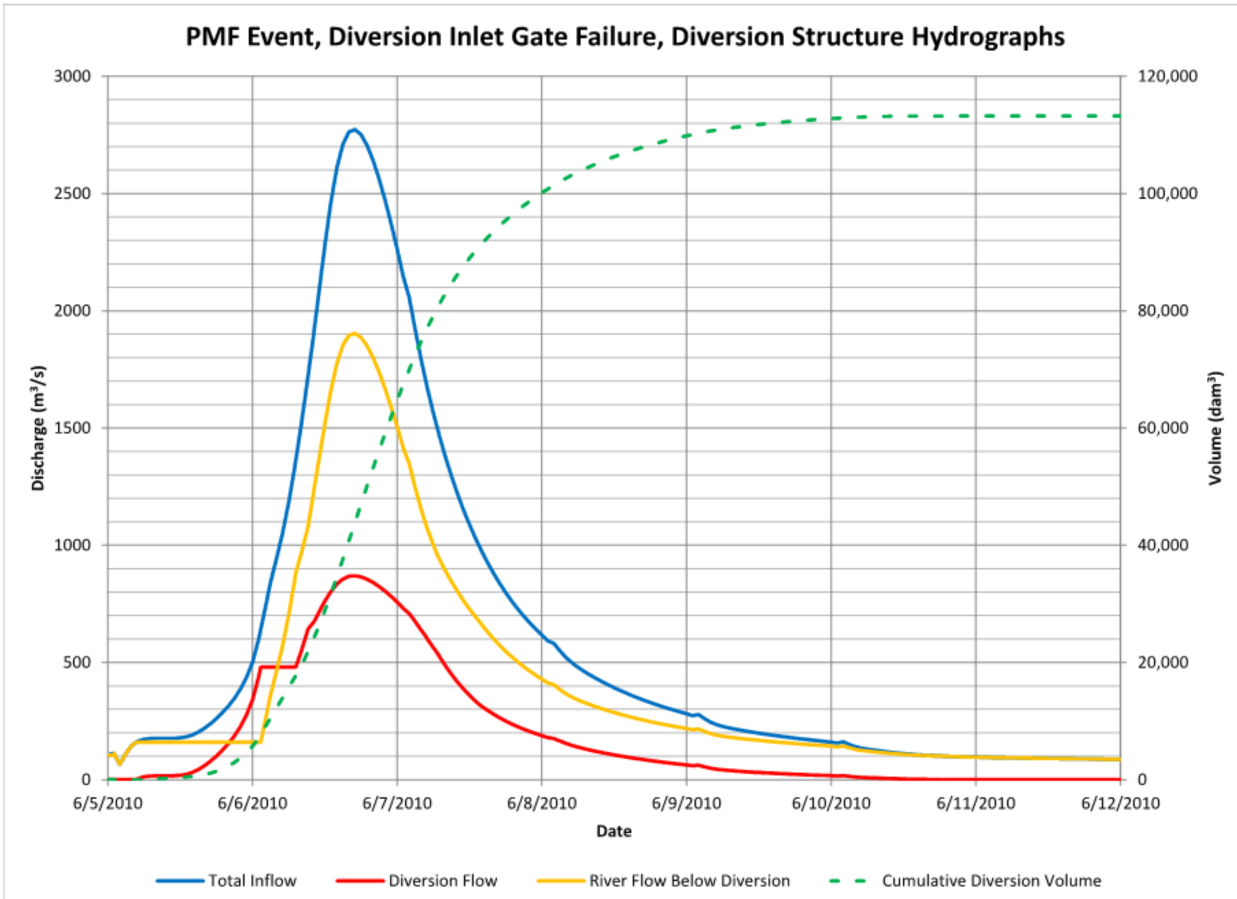

Figure IR528-1 Calculated Diversion Structure Hydrographs

The HEC-RAS model simulates the unsteady flow hydraulics of the diversion channel and the elevations of the reservoir for the calculated inflow hydrograph from the HEC-ResSim model. The emergency spillway is modelled as a lateral weir within the channel. The reservoir is modeled as a storage area at the end of the channel and provided with a stage-storage relationship. The results of the HEC-RAS model simulation are presented in Figure IR528-2.

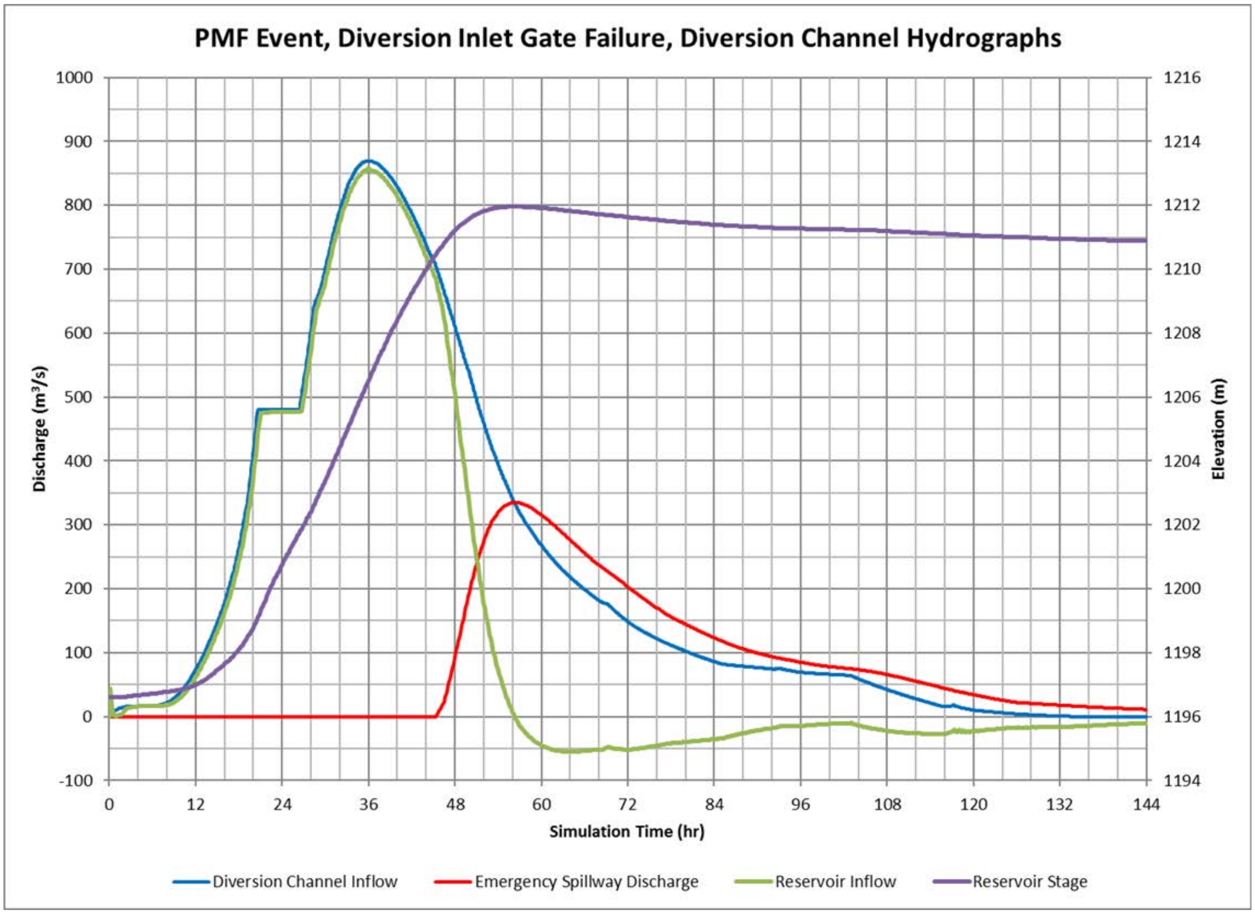

Figure IR528-2 Modelling Output for the Diversion Structure Hydrographs

Based on the simulation results, the maximum reservoir water level is calculated as El. 1,212 m.

Two wind generated wave scenarios were evaluated. The normal freeboard assumes the off-stream reservoir is at its maximum normal elevation and the freeboard prevents overtopping by 95 percent of the waves caused by the most critical wind with a frequency of 1:1,000 years. The minimum freeboard assumes the off-stream reservoir is at its maximum elevation during passage of the PMF and the freeboard should prevent overtopping by 95 percent of the waves caused by the most critical wind with a frequency of 1:2 years.

Calculations for wind and wave run-up used methods outlined in United States Bureau of Reclamation (USBR) Technical Memorandum No. 2 (1981). Maximum normal pool elevation in the reservoir is assumed to be the full service level (El. 1,210.75 m). Table IR528-1 presents the results of the freeboard calculations.

Table IR528-1 Normal and Minimum Freeboard Calculations Summary

	Normal Freeboard	Minimum Freeboard
Wind Velocity Return Interval (AEP)	1:1,000 year	1:2 year
Design Wind Velocity (m/s)	29.0	24.5
Fetch Length (km)	4.80	4.80
Calculated Wave Runup (m)	2.12	1.42
Calculated Wave Setup (m)	0.13	0.04
Total Freeboard Required Above Pool Elevation (m)	2.25	1.46
Pool Elevation (m)	1,210.75	1,212.00
Required Crest Height (m)	1,213.00	1,213.46

b. No, the antecedent event was applied to the hydrologic model in order to pre-wet the conditions for infiltration and produce a more conservative runoff volume. As an off-stream reservoir with the analyzed event being a complete failure of the diversion inlet gates to close, it was determined that routing both the antecedent event and the PMF through the reservoir was not an appropriate design basis. The antecedent event was not routed through the model; therefore, there is no such figure to be provided.

c. The local runoff volume (540 dam^3) represents the local runoff for a 1:100 year 6-hour rainfall event. Due to timing effects for PMF, it is assumed local rainfall associated with the PMP would be collected and pass through the low-level outlet works prior to operation of the gates for diversion. The local runoff (1:100 year) is assumed as a conservative estimate during or after the diversion of the Elbow River PMF.

d. The referenced period does reflect normal operation of the diversion structure. The period of normal operation begins when the flow in the river exceeds 160 m^3/s. Diversion rates increase from 0 m^3/s to 480 m^3/s as river flows increase from 160 m^3/s to 640 m^3/s. As river flows continue to increase above 640 m^3/s, the service spillway gates will continue operation by lowering and maintaining a constant diversion rate of 480 m^3/s. This continues until the service spillway gates are fully open (lowered) and influence of the gates no longer affects the diversion rate.

e. The slightly offset and lower hydrograph of the off-stream reservoir inflow, in comparison to the diversion inflow, represents the effects of lag time (distance from diversion inlet to the off-stream reservoir) and storage calculated in the HEC-RAS unsteady hydraulic model. The total volume of flow diverted to the diversion channel in the model is retained in the simulation.

Dam Safety
May 2019