REPORT

November 2011

HIGHFIELD DAM

Dam Classification and Hydro Technical Study

Submitted to:
Agriculture and Agri - Food Canada (AAFC)
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Golder Associates

November 10, 2011
Agriculture and Agri Food Canada (AAFC)
Room 408, 1800 Hamilton Street
Regina, Saskatchewan
S4P 4L2

Attention: Mr. Garth Haack, Project Manager

Re: Final Report - Highfield Dam Services Contract No. 4: Dam Classification and Hydro Technical Study for the Highfield Dam

Dear Mr. Haack:

We are pleased to submit to AAFC three hard copies of our final report on the Dam Classification and Hydro-technical Study for the Highfield Dam. The final report includes a DVD containing one digital copy of the report, the FLDWAV and HEC-RAS model data files and the relevant digital flood inundation maps.

We thank you for commissioning Golder Associates Ltd. to undertake this interesting assignment. Please contact the undersigned should you require any clarification regarding this report submission.

Yours truly,

GOLDER ASSOCIATES LTD.

Hua Zhang, Ph.D., P.Eng.
Project Manager and Associate
Senior Hydrodynamic Modeling Specialist

Anil Beersing, Ph.D., P.Eng.
Principal, Senior Hydrologist

Acknowledgements

Golder Associates Ltd. (Golder) acknowledges the contributions of the following staff of Agriculture and Agri Food Canada (AAFC) to this study entitled “Highfield Dam – Services Contract No. 4 – Dam Classification and Hydro Technical Study”:

- Mr. Garth Haack, AAFC’s project manager for this study, provided overall direction to the study, participated in the project kick-off meeting and field reconnaissance, supplied the available and relevant information, coordinated the inputs and participation from AAFC, and provided review comments on the study report.
- Mr. Glenn McLaughlin from AAFC’s Swift Current Office participated in the project kick-off meeting, and provided technical advice and review comments on the study report.

The contributions of the following staff from Golder to the study are also acknowledged:

- Dr. Hua Zhang, Golder’s project manager, was responsible for regular communications with AAFC, undertaking the field work, scheduling conference calls with AAFC, overseeing the dam breach modeling analysis, the flood inundation mapping and preparing the study report.
- Dr. Anil Beersing, Golder’s senior advisor and reviewer for the study, provided technical guidance and senior input to the study, provided quality control and assurance for the study, and reviewed the study report. He was also Golder’s project sponsor responsible for direct communications with AAFC, obtaining feedback on the project progress, providing senior advice to the project team and ensuring that AAFC’s expectations were met.
- Mr. Jie Chen conducted the field inspection and surveys and detailed hydrodynamic modeling analysis, including model setup and testing and preparation of inundation mapping, and assisted Dr. Zhang in preparing the study report.
- Mr. Mark Chiarandini was a member of the field survey crew who conducted the field inspection and surveys for this study.

Executive Summary

Golder Associates Ltd. (Golder) was retained by Agriculture and Agri Food Canada (AAFC) to conduct a study entitled "Highfield Dam – Services Contract No. 4 – Dam Classification and Hydro Technical Study" near the City of Swift Current in Saskatchewan. The study included hydraulic modeling and flood inundation mapping of Highfield Dam breach floods using recently acquired LiDAR survey data and contour information. AAFC plans to use the results of this study to provide inputs for the final design of the dam and spillway, to enhance the flood warning communication section in the Operation and Maintenance Manual and Emergency Preparedness Plans (EPP), and to confirm the classification of the dam.

The Highfield Dam is situated on Rush Lake Creek, approximately 28 km east of the City of Swift Current in Saskatchewan. The Highfield Dam is a zoned earthen dam located in Section 36-T15-R11-W3M. It was originally constructed in 1942. The dam is 8.2 m high and has a crest length of 1,040 m at the existing top-of-dam elevation of 724.8 m. The reservoir has a surface area of 5.2 km^2 at its Full Supply Level (FSL) of 723.0 m. The reservoir behind the dam has a storage capacity of 15,130 dam^3 at FSL and a storage capacity of 25,750 dam^3 at the existing top-of-dam elevation. AAFC is planning to raise dam crest elevation to 725.7 m by about 0.9 m. Accordingly, the surface area and storage capacity will be increased to 7.4 km^2 and 32,060 dam^3 at the planed top-of-dam elevation of 725.7 m, respectively.

The main man-made structures along the floodway include five major highway and CP railway bridge/culvert crossings, and 10 local road bridge/culvert crossings. There is one small community (i.e., Village of Rush Lake) and about 18 other residences and buildings situated within the potential dam breach flood inundation area.

The study reach for the flood flow routing extends from the Highfield Dam to a downstream study boundary located 45.7 km from the dam. Two hydrodynamic models (i.e., HEC-RAS and FLDWAV models) were used to conduct simulations of steady state hydraulic conditions and potential breaches of the Highfield Dam. A total of six stage-discharge rating curves and three dam failure scenarios were modeled. Model sensitivity analysis was conducted for quantifying modeling uncertainty.

The modeling results for the dam breach floods were used to prepare two sets of flood inundation maps at a scale of 1:20,000. One set shows the flood inundation limits on a base map with contours, roads and other mapping features. The other set shows the same flood inundation information but with ortho air photos as background. The flood inundation maps are believed to provide a sufficient definition of the areal extents of flooding due to the dam breach floods.

The results of this study support the following key conclusions:

- Based on the dam classification criteria provided by AAFC and the consequences assessment carried out for this study, the Dam Class of Highfield Dam is recommended to be in the Significant Consequence category.
- Based on the recommended Significant Consequence classification of Highfield Dam, the Inflow Design Flood (IDF) is expected to be between the 100-year and 1,000-year flood.
- The estimated loss of life, including both the temporary and permanent PAR, as a result of a hypothetical overtopping or fair-weather failure of Highfield Dam is expected to be zero.

- In the event of a piping failure or overtopping failure of the Highfield Dam, the Village of Rush Lake would not be flooded. In addition, all other residences, except one, and buildings in the study area would not be flooded. One house, downstream of the CPR bridge crossing, would likely be flooded during an overtopping failure of the Highfield Dam.
- In the event of an overtopping failure or a piping failure of the Highfield Dam, sections of the Highway 1 embankments (both east and west bounds) approximately 3 km west of the Village of Rush Lake would be overtopped and likely be damaged, the Highway 1 culvert road embankment would not be overtopped, and the 1st CPR bridge embankment would not be overtopped. However, the CRP bridge crossing would likely be damaged.
- In the event of a piping failure of the Highfield Dam, the 2nd CPR bridge embankment located approximately 5 km east of the Village of Rush Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, this 2nd CPR bridge embankment would be overtopped and likely be damaged.
- In the event of a piping failure of the Highfield Dam, the embankment of a low section of Highway 1 located approximately 6 km east of the Village of Rush Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, this low highway embankment would be overtopped and likely be damaged.
- In the event of an overtopping failure or a piping failure of the Highfield Dam, most downstream local road bridge/culvert crossings would be overtopped and likely be damaged.
- The Highway 1 east bound bridge and west bound bridge can safely pass a 200-year flood and a 50-year flood at the Highfield Dam, respectively. The Highway 1 embankment can safely pass a flood event between the 100-year and 200-year floods without causing a catastrophic failure of the highway embankment. The Highway 1 culvert crossing can safely pass a flood less than the 20-year flood event without causing a catastrophic failure of the structure.
- The 1st CPR bridge can safely pass a flood between the 200-year flood and the 500-year flood. The CPR embankment can safely pass the 1,000-year flood event. The 2nd CPR bridge can safely pass a flood less than the 10-year flood event at the Highfield Dam. The 2nd CPR embankment can only safely pass a 10-year flood event.

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1.0 INTRODUCTION

1.1 Background and Study Objective

Agriculture and Agri Food Canada (AAFC) is endeavouring to align the management of its dams in accordance with the Dam Safety Guidelines prepared by Canadian Dam Association (CDA 2007). These Dam Safety Guidelines have established the industry standard of care for dams in general. A requirement stated in the guidelines is that a formal incremental consequence classification has to be conducted for the dam. Such dam classification is used as a measure for setting or establishing the standard of care for the dam. Currently, AAFC is undertaking feasibility level engineering assessments toward an overall rehabilitation of the Highfield Dam. The Highfield Dam is owned, operated and maintained by AAFC. It is notionally classified as a "HIGH" Consequence Dam. However, AAFC has decided that that a formal, rigorous and detailed incremental consequence assessment (ICC) should be conducted for the dam prior to the design of a planned modification of the dam's spillway and embankment.

Golder Associates Ltd. (Golder) was retained by AAFC to conduct a study entitled "*Highfield Dam – Services Contract No. 4 – Dam Classification and Hydro Technical Study*" in accordance with the scope of work prepared by AAFC on June 20, 2011. AAFC required that the study be conducted in accordance with the 2007 CDA Dam Safety Guidelines. The study involves hydraulic modeling and inundation mapping analysis of the Highfield Dam using recently acquired LiDAR survey data and contour information. AAFC plans to use the results of this study to provide inputs for the final design of the dam and spillway, to enhance the flood warning communication section in the Operation and Maintenance Manual and Emergency Preparedness Plans (EPP), and to confirm the classification of the dam.

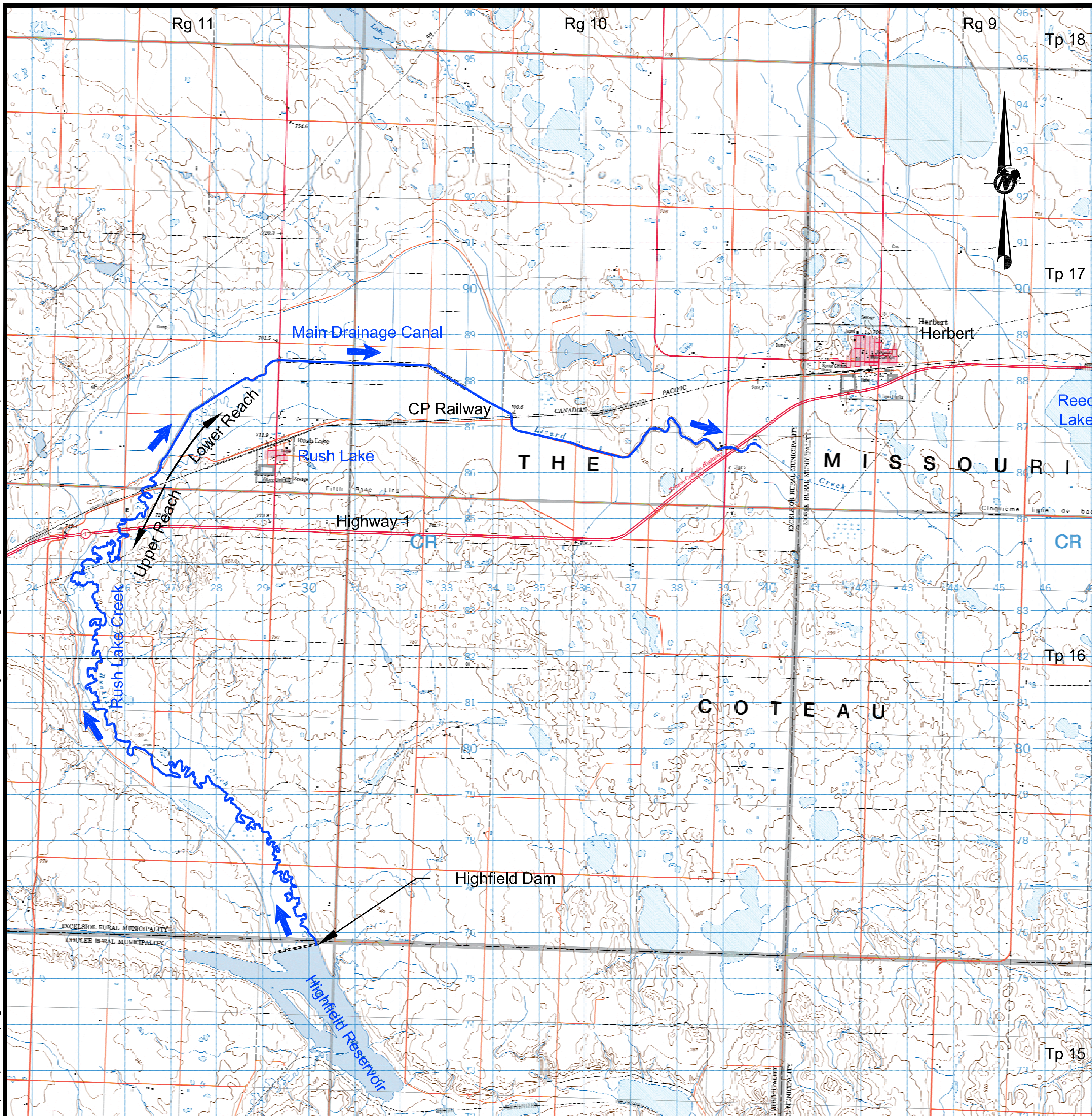
1.2 Study Area

Figure 1 shows the study area including locations of the Highfield Dam and the floodway study reach for potential breaches of the dam. The Highfield Dam is situated on Rush Lake Creek, approximately 28 km east of the City of Swift Current in Saskatchewan. The downstream water users include the Herbert and Rush Lake Irrigation Projects.

The Highfield Dam is a zoned earthen dam located in Section 36-T15-R11-W3M. It was originally constructed in 1942. The dam is about 8 m high and has a crest length of 1,040 m at the existing top-of-dam elevation of 724.8 m. The reservoir has a surface area of 5.2 km^2 at its Full Supply Level (FSL) of 723.0 m. The reservoir behind the dam has a storage capacity of 15,130 dam^3 at FSL and a storage capacity of 25,750 dam^3 at the existing top-of-dam elevation. AAFC is planning to raise the dam crest elevation by about 0.9 m to 725.7 m. Accordingly, the surface area and storage capacity will be increased to 7.4 km^2 and 32,060 dam^3, respectively, at the planned top-of-dam elevation of 725.7 m.

The discharge facilities at the Highfield Dam include one 20 m wide earth cut spillway and two low level outlet structures. The spillway is located on the west abutment of the dam. The existing spillway has a capacity of 58 m^3/s with the reservoir water level at the existing top-of-dam elevation. One irrigation low level outlet is located near the west abutment of the dam and the other one is located near the east abutment.

The extent of the inundation study for the Highfield Dam includes the potential floodway from the dam site to a location approximately 45.7 km (channel/canal length) downstream of the dam along the Rush Lake Creek reach as shown on Figure 1. The downstream study boundary terminates just downstream of the Highway 1 (TransCanada Highway) crossing of the main drainage canal from the Rush Lake Irrigation Project. The main structures along the potential dam breach floodway include five major highway and railway crossings and 10 local road crossings. There is one small community (Village of Rush Lake) along the dam breach floodway, in addition to some residences and developments located on the potential dam breach floodplains.



LEGEND

- STUDY REACH
- HIGHWAY
- RAILWAY
- WATERCOURSE
- FLOOD FLOW DIRECTION

2.5 0 2.5

SCALE KILOMETRES

REFERENCE

TOPOGRAPHIC MAP 72J/06(© 1999) OBTAINED FROM MapTown. HER MAJESTY THE QUEEN IN RIGHT OF CANADA. DEPARTMENT OF NATURAL RESOURCES. ALL RIGHTS RESERVED. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD83 COORDINATE SYSTEM: UTM ZONE 13

PROJECT

DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE

LOCATION OF THE STUDY AREA

Golder Associates
Calgary, Alberta

PROJECT	11132600459000		FILE No. 11132600459000FG005	
DESIGN	JC	09/03/11	SCALE AS SHOWN	REV. 0
CADD	YW	09/03/11	**FIGURE: 1**	
CHECK	JC	04/11/11		
REVIEW	HZ	04/11/11		

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2.0 MAIN PHYSICAL FEATURES AND MAN-MADE STRUCTURES

2.1 General

The physical setting and main man-made structures in the study area that are relevant to the steady state hydraulic modeling and dam breach modeling analyses are described in the following sections. These features and structures include the Highfield Reservoir, the Highfield Dam, the low level outlet structures, the spillway structure, the local road bridges/culvert crossings, the major highway and railway bridge/culvert crossings, and the creek channels/canal and floodplains along the study reach.

Sources of information for a description of the physical features and the man-made structures were obtained from AAFC and included:

- Past study and design reports, and design and as-built drawings;
- 1:50,000 scale, 10 m contour topographic maps;
- LiDAR data collected in 2009;
- available aerial or orthorectified imagery collected in 2009;
- Site information collected during a field reconnaissance on August 2 and 3, 2011; and
- Field surveys of 10 local road bridge/culvert crossings, three Highway 1 bridge/culvert crossings, and two CPR bridge crossings.

2.2 Highfield Reservoir

The FSL of the Highfield Reservoir is 723.0 m. The corresponding reservoir surface area and storage at the FSL are 5.2 km^2 and 15,130 dam^3, respectively. The reservoir elevation-storage-area curves shown on Figure 2 are based on AAFC-AESB's Drawing No. 208553 dated August 2011. Photographs of the Highfield Reservoir are presented on Figure A-1 in Appendix A.

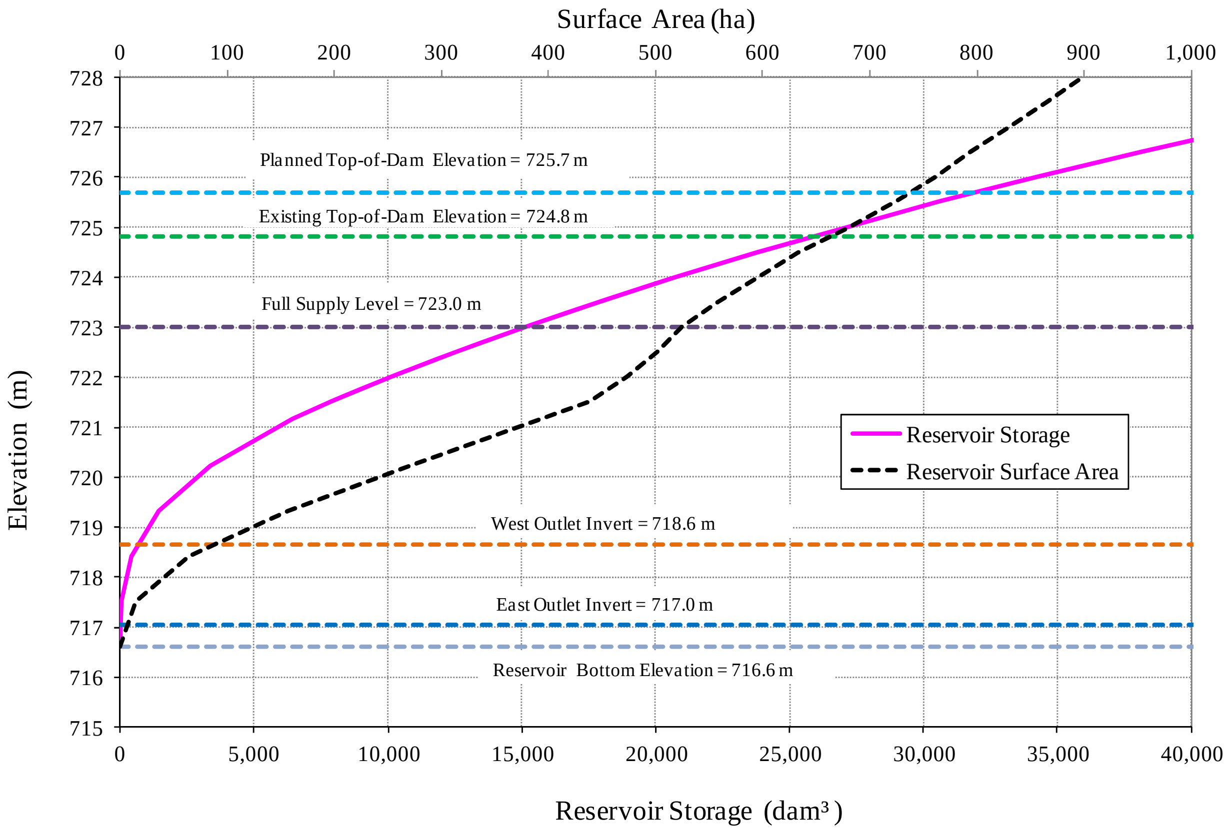

Figure 2: Highfield Reservoir Surface Area and Storage Curves

2.3 Highfield Dam

The Highfield Dam is an earth-fill dam located in Section 36-T15-R11-W3M. The upstream face of the dam is protected from wave erosion with riprap. The crest length of the main dam is 1.0 km.

2.3.1 Existing Highfield Dam

The existing top-of-dam elevation is 724.8 m. The height of the dam above the downstream toe of the dam is 8.2 m.

Table 1 summarizes the main engineering parameters of the dam. Figure 3 shows a plot of the outflow rating curve for the existing Highfield Dam Earth Spillway based on AAFC's Drawing No. 116036A dated April 1992. Figure A-1 in Appendix A presents photographs of the dam.

Table 1: Main Engineering Parameters of the Existing Highfield Dam

Parameter	Value
Existing Top-of-Dam Elevation	724.8 m
Reservoir Full Supply Level (FSL)	723.0 m
Reservoir Storage at FSL	15,130 dam^3
Dam Crest Length	1.0 km
Dam Height to Downstream Dam Toe	8.2 m
Average Dam Slopes	3.0 (H):1(V)
Dam Top Width (including Riprap)	5 m

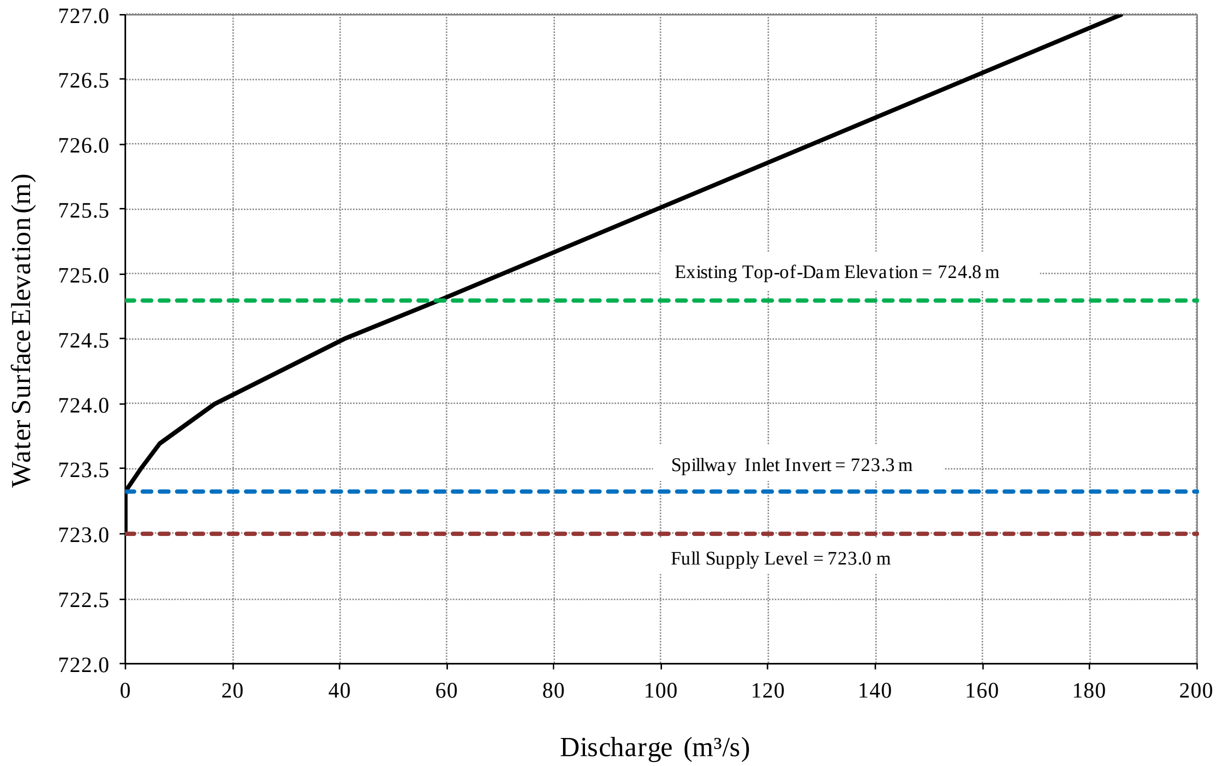

Figure 3: Outflow Rating Curve for the Existing Highfield Dam Earth Spillway

2.3.2 Anticipated Highfield Dam

The future top-of-dam elevation will be raised to 725.7 m. The height of the dam above the downstream toe of the dam will be 9.1 m.

Table 2 summarizes the main engineering parameters of the planned Highfield Dam. Figure 4 shows a plot of an outflow rating curve for the proposed Highfield Dam Spillway based on a draft spillway pre-design report by NHC for AAFC dated March 2010, (See Reference. Currently, it is under review by AAFC).

Table 2: Main Engineering Parameters of the Planned Highfield Dam

Parameter	Value
Planned Future Top-of-Dam Elevation	725.7 m
Reservoir Full Supply Level (FSL)	723.0 m
Reservoir Storage at FSL	15,130 dam^3
Dam Crest Length	1.0 km
Dam Height to Downstream Dam Toe	9.1 m
Average Dam Slopes	3~6 (H):1(V)
Dam Top Width (including Riprap)	5.5 m (to be confirmed by AAFC)

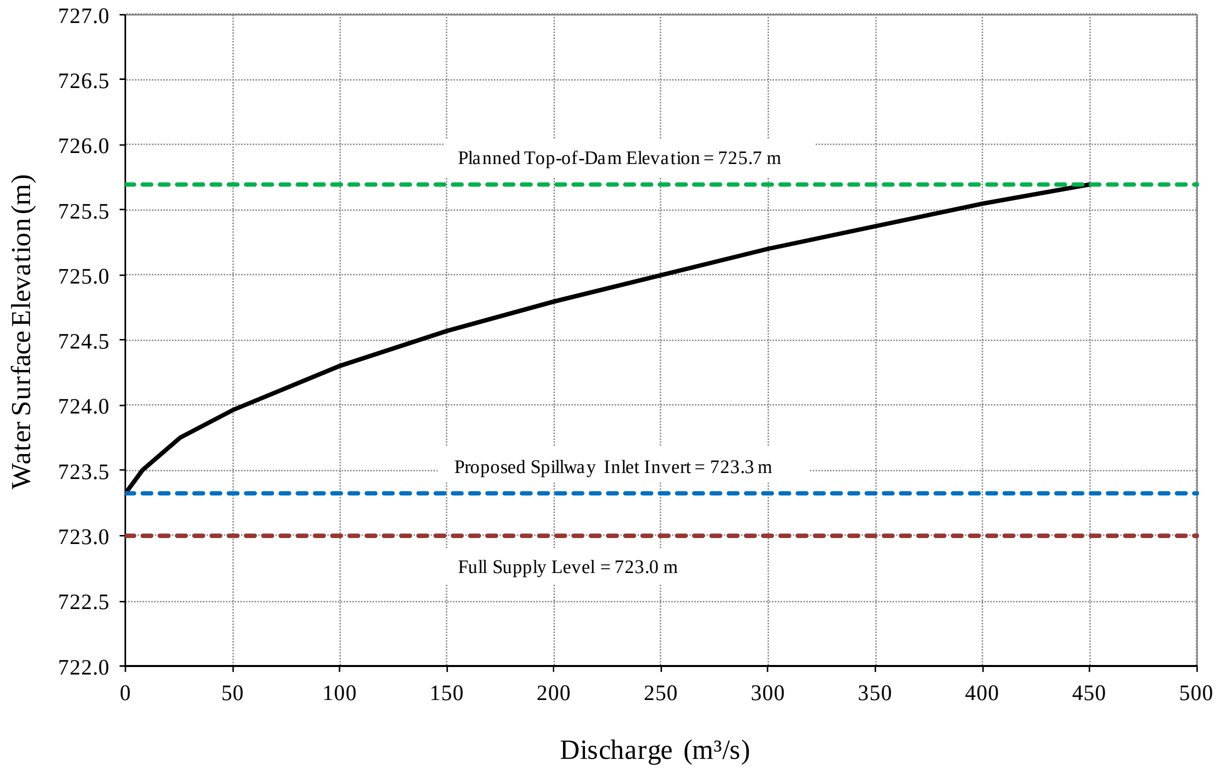

Figure 4: Outflow Rating Curve for the Proposed Highfield Dam Spillway

2.4 Rush Lake Creek

The floodway for a potential dam failure flood of the Highfield Dam extends from the dam site to a location approximately 45.7 km downstream of the dam along the Rush Lake Creek, where the study reach terminates. The distance used in this study includes channel and canal lengths along Rush Lake Creek. The study reach is divided into two sub-reaches for simulating dam breach floods from the Highfield Dam.

The upper Rush Lake Creek study reach extends from the Highfield Dam to the 1st CPR bridge crossing approximately 28.5 km downstream of the dam site. The total length of this upper study reach is approximately 28.5 km. Based on the LiDAR survey information and channel cross section surveys, the upper Rush Lake Creek channel has bottom widths of approximately 5 m to 10 m, bankfull widths of approximately 12 m to 20 m, and bankfull depths of approximately 1.5 m to 3.0 m. Based on the available topographic information, the average valley and creek channel bed slopes along the upper study reach are estimated to be 0.13% and 0.045%, respectively. The average creek channel sinuosity is estimated at 2.8.

The lower Rush Lake Creek study reach extends from the 1st CPR bridge crossing to the downstream study boundary. The total length of this lower study reach is approximately 17.2 km. Based on the LiDAR survey information and surveys of the Main Drainage Canal cross sections, the canal has bottom widths of approximately 6 m to 12 m, bankfull widths of approximately 15 m to 25 m, and bankfull depths of approximately 1.5 m to 4.0 m. Based on the available topographic information, the average valley slope along the lower study reach is estimated to be 0.038%.

Figure A-2 in Appendix A presents photographs of the Rush Lake Creek study reach and its floodplain characteristics. The channel bed/bank materials consist of sand, silt and clay. The vegetation cover on the creek banks and part of the floodplain consists mainly of grasses and scattered willows/trees along the upper Rush Lake Creek floodway. The floodplains are mainly farmland.

2.5 Bridge and Culvert Crossings

There are a total of 15 bridge/culvert crossings along the Rush Lake Creek study reach, including 10 local road bridge/culvert crossings, three Highway 1 bridge/culvert crossings, and two CPR bridge crossings. Figure A-3 in Appendix A presents photographs of these crossings. The bridge/culvert crossings were surveyed and information on total bridge span, bridge girdle height and number of piers, culvert diameter and length, etc., were recorded during the field inspections. Tables 2 and 3 summarize the relevant bridge and culvert information, respectively, based on the LiDAR survey information and the information collected during the field inspection.

Table 3: Relevant Bridge Information

Name of Bridge Crossing	Distance from the Highfield Dam (km)	Total Span(1) (m)	Top of Bridge Deck Elevation(1) (2) (m)	Bridge Low Chord Elevation(1) (m)	No. of Piers(1)
Highway 1 Bridge (East Bound)	26.3	24	711.2-711.4	710.7-711.0	2
Highway 1 Bridge (West Bound)	26.3	24	709.8	708.3	1
Local Bridge	28.4	23	707.3	706.5	0
1st CPR Bridge	28.5	20	710.7	708.2	0
Local Bridge	29.4	17	706.3	705.9	2
Local Bridge	33.1	11	703.2	702.4	1
Local Bridge	34.8	16	701.6	701.2	2
Local Bridge	36.4	15	700.4	699.8	2
2nd CPR Bridge	38.5	12	702.1	700.9	0

Notes: (1) Based on field survey (2) Based on LiDAR survey.

Table 4: Relevant Culvert Information

Name of Culvert Crossing	Distance from the Highfield Dam (km)	Culvert Diameter / Length(1) (m)	Culvert Upstream / Downstream Invert Elevation(1) (m)	Road Surface Elevation(1) (2) (m)	No. of Culverts (1)
Local Road Culvert	4.8	2.5 / 18	712.4 / 712.0	715.5	1
Local Road Culvert	6.5	2.7 / 17	711.8 / 711.5	715.0	1
Local Road Culvert	18.9	2.2 / 13	707.8 / 707.6	710.5	1
Local Road Culvert	40.2	4.5 / 14	696.0 / 695.5	700.3	1
Local Road Culvert (two culverts)	44.5	1.5 / 34 (small)	696.5 / 696.2	703.0	1
		2.0 / 24(large)	695.6 / 695.3		1
Highway 1 Culvert	44.7	3.2 / 97	694.7 / 694.5	702.1	1

Notes: (1) Based on field survey (2) Based on LiDAR survey.

2.6 Houses and Buildings on the Floodplains

There is one community (Rush Lake) and some houses and buildings on the floodplains along the Rush Lake Creek study reach. There are 18 houses and buildings located outside of Rusk Lake. Some of these houses and buildings would be affected by potential dam breach floods. Figure A-4 of Appendix A presents the photographs of the community, houses and buildings and their estimated coordinates. A field survey of these houses and buildings on the floodplains was conducted by using a GPS survey unit.

3.0 MODELING ANALYSIS

3.1 Steady State Hydraulic Modeling

3.1.1 Modeling Approach

A steady-state hydraulic modeling analysis is required to develop: 1) two tail water rating curves to be used by AAFC for the final designs of East Low Level Outlet and the new spillway structure; and 2) five stage-discharge rating curves at the Highway 1 bridge/culvert crossings and CPR bridge crossings to estimate flows and associated return periods that might lead to catastrophic failures of the highway and CPR embankments. Steady-state flow refers to the condition where the fluid properties at a point in a hydraulic system, such as flow depth and velocity, are not changing over time. During a steady-state hydraulic model simulation, it is assumed that the flow in the channel is constant, that is, there is an unlimited supply of water into the channel(s) being modeled.

The rating curves have been developed to a maximum discharge of 361 m^3/s, which is the 1,000-year flood event. The steady state hydraulic modeling analysis was conducted using the HEC-RAS model developed by the U.S. Corps of Engineers (Version 4.1, dated January 2010).

3.1.2 Description of the HEC-RAS Model

The HEC-RAS (Version 4.1, dated January 2010) model was used to route the flood flows. HEC-RAS is a hydraulic model that can be used to perform one-dimensional calculations for natural and constructed channels. The model was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The software has a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, and graphics and reporting facilities. The HEC-RAS model was developed for calculating water surface profiles for steady and unsteady events by solving the energy equation between cross-sections. HEC-RAS can be used for modeling mixed flow regime that includes subcritical, supercritical, hydraulic jump and drawdown in the unsteady flow module.

3.1.3 Development of Rating Curves

In accordance to the RFP and discussion with AAFC, the following seven stage-discharge rating curves were developed based on steady state hydraulic modeling analyses:

- Two tail water rating curve downstream of the Highfield Dam;
- Two rating curves at the two Highway 1 bridge crossings;
- One rating curve at the Highway 1 culvert crossing; and
- Two rating curves at the two CPR bridge crossings.

The modeling analyses included nine flood peak discharges (i.e., 1.2 m^3/s, 7.6 m^3/s, 19 m^3/s, 39 m^3/s, 80 m^3/s, 123 m^3/s, 180 m^3/s, 274 m^3/s and 361 m^3/s). The corresponding flood events are the 2-year, 5-year, 10-year, 20-year, 50-year, 100-year, 200-year, 500-year and 1,000-year flood events at the Highfield Dam, respectively.

The cross-section dimensions and profiles of the creek and canal and the associated floodplains used in the HEC-RAS model were extracted from recently surveyed channel cross sections surveyed during the field investigation and recently acquired LiDAR survey data set. The cross-sections of the creek and canal were surveyed at these key locations in order to develop accurate stage-discharge rating curves in this study.

The approach used for representing the creek channel and floodplain is believed to provide a sufficient level of accuracy for the flood routing analyses. The channel/canal top width and floodplain width are the two key geometric parameters that affect the flood routing because of the large discharges associated with potential dam breach floods. The width estimates used in the model are sufficiently accurate for flood modeling and the flood inundation mapping work.

Table 8 provides the estimated range of Manning's n values for the creek channel and floodplains considering their bed/bank materials and meandering characteristics, and the floodplain's vegetative cover (tree, grass and farmland), man-made structures and overland form roughness.

The rating curves developed for this study were based on best estimates of Manning's n values (see Table 8) because observed in-field flow data was not available for hydraulic model calibration.

3.1.4 Modeling Results

Figures B-1 to B-7 in Appendix B graphically present the rating curves. Table 5 summarizes the hydraulic capacity of Highway 1 bridges/culvert and CPR bridges, the predicted flows and estimated flood return periods, which could lead to catastrophic failures of the highway and railway embankments. In summary:

- The Highway 1 east bound bridge and west bound bridge can safely pass the 200-year flood and 50-year flood, respectively. The Highway 1 embankment can safely pass a flood event between the 100-year and 200-year floods without causing a catastrophic failure of the highway embankment. The Highway 1 culvert crossing can safely pass a flood less than the 20-year flood event without causing a catastrophic failure of the structure.
- The 1st CPR bridge can safely pass an event between the 200-year flood and the 500-year flood. The CPR embankment can safely pass the 1,000-year flood event. The 2nd CPR bridge can safely pass a flood less than the 10-year flood event. The 2nd CPR embankment can safely pass a 10-year flood event.

Table 5: Predicted Steady-State Flood Flows and Estimated Flood Return Periods at Key Road Crossing Locations

Name of Crossing	Bridge Low Chord		Bridge Deck		Road/Embankment Elevation	
	Predicted Discharge (m^3/s)	Estimated Return Period*	Predicted Discharge (m^3/s)	Estimated Flood Return period	Predicted Discharge (m^3/s)	Estimated Flood Return Period*
Highway 1 Bridge (East Bound)	190	~ 200-year flood	280	~ 500-year flood	160	Between 100- and 200-year floods
Highway 1 Bridge (West Bound)	80	~ 50-year flood	150	Between 100- and 200-year floods	160	Between 100- and 200-year floods
CPR 1st Bridge	210	Between 200- and 500-year floods	>361	> 1,000-year flood	>361	> 1,000-year flood
CPR 2nd Bridge	17	~ 10-year flood	28	Between 10- and 20-year floods	18	~ 10-year flood
Highway 1 Culvert	-		-		39	~ 20-year flood

*Note: Estimated rainfall generated flood events at the Highfield Dam site.

In June 2010, a significant rainfall generated flood event near Maple Creek, Saskatchewan led to the overtopping and subsequent failure of the Highway 1 section immediately downstream of the AAFC's Junction Dam. AAFC was concerned that a similar catastrophic failure could occur at the Highway 1 crossing below Highfield Dam either during one extreme flood flow passage or as a possible consequence of a Highfield Dam failure. In the original RFP, AAFC requested the assessment of the possibility of such a catastrophic failure.

The locations and geographic settings of Highway 1 crossings appear significantly different from those at the Highway 1 crossings at the Maple Creek. Although the Highfield Dam is also located upstream of Highway 1 and CP railway infrastructure in this study, a significant flood event (i.e., >1,000-year flood at the Highfield Dam or a dam failure) is less likely to cause a catastrophic failure of either the highway embankments or CPR embankment based on the above hydraulic analysis, the differences in highway topography and our project experience and judgement. The reasons for this opinion are as follows:

- Relative lower highway embankments (east bound and west bound) along the highway sections downstream of Highfield Dam;
- Highway embankments situated on relatively flat floodplains that tend to attenuate flood velocity and therefore the erosion potential of the flood wave;
- Potential backwater effects from the 1st CPR embankment during extreme flooding conditions, which also tend to attenuate flood velocity and therefore the erosion potential of the flood wave;

- Maximum flood velocity at Highway 1 embankments estimated to be 1 m/s during extreme flooding conditions based on HEC-RAS modeling, which is not considered to be fast enough for catastrophic erosion of the embankments;
- In the event of an overtopping failure or piping failure of the Highfield Dam, the 1st CPR bridge embankment would not be overtopped, see Section 3.4; and
- The CPR embankment has a long flat grade. It would serve as a very wide broad crested weir if overtopped, and would not generate high flow velocity and depth for catastrophic erosion of the embankment.

3.2 Dam Breach Flood Modeling

3.2.1 Modeling Approach

A dam breach analysis is required to predict outflows from a potential dam breach and to assign an appropriate dam classification based on an incremental consequence assessment as recommended in the 2007 CDA Guidelines. The modeling analysis involved the following:

- A hydraulic modeling analysis to predict the dam breach flood hydrographs at the dam site and downstream routing of the dam breach floods, including flood levels, discharges and flow velocities; and
- A sensitivity analysis of the key model parameters to define the level of uncertainty associated with the modeling results and to evaluate the effects of such uncertainties on the accuracy of the dam breach flood modeling.

The dam breach modeling analysis was conducted using the FLDWAV model developed by the U.S. National Weather Service (U.S. NWS 2000). The downstream dam breach flood routing analysis was conducted using the HEC-RAS model.

3.2.2 Model Selection and Description

3.2.2.1 Model Selection

Two computer models were selected for dam breach modeling and routing the dam breach floods along the floodway. The main considerations for the model selection are described below.

- FLDWAV Model: This one-dimensional model was selected for predicting the dam breach flood outflows at the potential dam breach openings. This model is widely used for dam breach flood inundation studies. The model offers the commonly-used, empirically-based formulations for characterizing the dam breach process and for predicting the outflows from a dam breach. FLDWAV model has an improved numerical scheme that makes it easier or better in simulating dam breach process than the HEC-RAS model. The latter model consists of a DAMBRK module, which is an older version of FLDWAV.
- HEC-RAS Model: This one-dimensional model was selected for routing the dam breach flood along the floodways. The model was selected because it can readily accept the available cross-sectional data for the study area. The HEC-GeoRAS module of the model is used to prepare cross-sectional data based on the digital elevation model (DEM) generated from the LiDAR survey data set. HEC-RAS is a commonly-used model for flood modeling analysis in North America. It can be used for both steady-state and unsteady-state flood profile computations.

3.2.2.2 Description of the FLDWAV Model

The FLDWAV model combines the capabilities of DWOPER and DAMBRK models developed for the U.S. NWS and supplies additional modeling features.

FLDWAV is a generalized flood routing model for unsteady or dynamic flow simulation. The governing equations contained in the model are the complete one-dimensional Saint-Venant equations for unsteady flows. Internal boundary equations included in the model, representing the rapidly varied (broad-crested weir) flows through structures (e.g., dams, bridges and embankments), enable users to specify time-dependent breach. In addition, the model allows the appropriate external boundary equations at the upstream and downstream ends of the routing reach to be specified.

The system of equations in the FLDWAV model is solved by an iterative, nonlinear, weighted four-point implicit finite-difference method. The flows that can be simulated may be either subcritical or supercritical, or a combination of each varying in space and time from one to the other. Fluid properties that can be simulated may obey either the principles of Newtonian or non-Newtonian flows. The hydrograph to be routed may be user-specified as input time series, or it can be developed by the model via user-specified breach parameters, including size, shape and time of development.

The model is designed to account for the following effects on the downstream propagation of a flood:

- Downstream dams that have significant flood storage and may be breached by the flood;
- Bridges and embankments that restrict the flood flows locally;
- Tributary inflows;
- River sinuosity;
- Levees located along the flood route; and
- Tidal effects.

The standard output from the FLDWAV model includes high water profiles along the river/valley, flood arrival times, times to peak flood levels and discharges, and discharge and stage (water-surface elevation) hydrographs at user-selected locations. The model input and output may be in English or metric (SI) units.

3.2.3 Dam Breach Scenarios and Model Parameters

3.2.3.1 Overtopping Failure

Analysis of one “flood induced” overtopping event is required in the 2007 CDA Guidelines for assessing dam ICC. Potential dam overtopping failure is a failure mode considered in this modeling analysis since it would result in severe downstream flooding.

Two overtopping failure scenarios were identified for this study. Both the 500-year and 1,000-year inflow hydrographs were modeled for potential dam overtopping failures. Table 6 presents the dam breach model parameter values selected for simulating these two potential overtopping failure scenarios. The parameter values were selected based on recommendations by the U.S. Federal Energy Regulatory Commission (FERC, 1994) and empirical formulations by Fread (2001), and considering the site-specific conditions of the dam.

Table 6: Selected Dam Breach Model Parameter Values for Overtopping Failure Simulation

Dam Breach Model Parameter	Selected Value
Average Width of Breach (BR)	54.6 m
Time to Failure (TFH)	0.5 hour
Horizontal Component of Side Slope of Breach (Z)	1.0
Elevation of Water When Dam Failure Commences (HFDD)	725.8 m

Dam breach modeling can be categorized as parametric or physically-based. The parametric breach models utilize key parameters, i.e., average breach width (BR) and time to failure (TFH) to represent the breach formation in earth dams. Physically-based dam breach models use principles of hydraulics, sediment erosion, and soil stability to construct time-stepping numerical solutions of the breach formation process.

Earth dams do not tend to completely fail, nor do they fail instantaneously. Average breach width (BR) is one of the key parameters that need to be determined with care in the parametric breach models. The fully formed breach openings in earth dams tend to have an average width (BR) in the range of 0.5HD to 8HD (HD is the height of the dam) as reported by Johnson and Illes (1976), Singh and Snorrason (1982) and Fread (2001). Both average breach width (BR) and time to failure (TFH) were predicted and reported by Froehlich (1987, 1995) based on statistical analyses of historical dam failures. Fread (2001) established the empirical relationships among the key breach parameters (i.e., BR and TFH), the reservoir volume (V_r) and the dam height (HD) based on statistical analysis of 63 historical breaches of dams ranging from 5 m to 87 m, with six dam heights greater 30 m.

The rationale for the selection of the modeling parameters is provided below.

- The average breach width (BR) is the most important dam breach model parameter. For this study, BR was selected to be six times the maximum height of the dam (HD) for the Highfield Dam, which is greater than the upper bound of a normal range (i.e., BR=5HD) recommended for earthfill dams by FERC. Fread (2001) indicated a wider range for BR (i.e., 0.5HD ≤BR≤ 8HD). Fread (2001) suggested the following empirical relationship for estimating BR:

$$BR = 9.5\, k_0\, (V_r\, H)^{0.25} \qquad (1)$$

where: BR – average breach width (ft)

k_0 – a coefficient (=1.0 for overtopping failure, and =0.7 for piping failure)

V_r – water volume (acre-ft)

H – height of water over the breach bottom (ft)

For the overtopping failure simulation, this relationship suggests BR=9.4HD for the Highfield Dam. A selection of BR=6HD is considered reasonable for the modeling the Highfield Dam for the following reasons:

- Age of the dam: Highfield Dam is a relatively old dam. It was originally constructed in 1942. There are no original design, construction and performance records available for the dam.
- Construction of the dam: Highfield Dam is not as well an engineered structure as other newly designed and constructed dams based on our communications with AAFC and our review of available reports.

- Wide downstream floodway: It would have very little hydraulic constraints during a potential dam overtopping failure because there is a 1 km wide and flat floodplain downstream of the Highfield Dam.
- Large reservoir storage volume: The potential breach width (BR) could be greater than 6HD as calculated based mainly on the reservoir volume in Equation (1).

- Time to failure (TFH) was assumed to be 0.5 hour. The value selected for this parameter is within the recommended range (0.1 hour ≤ TFH ≤ 1.0 hour) for earthfill dams by FERC. The following empirical relationship suggested by Fread (2001) provides an estimate of TFH≈3.1 hours for the Highfield Dam.

$$TFH = 0.3\ V_r^{0.53}/H^{0.9} \qquad (2)$$

  The predicted downstream flood peak levels are not expected to be very sensitive to this parameter. Therefore, a conservative estimate of TFH=0.5 hour was used in the modeling analysis.

- The horizontal component of the side slope of the breach (Z) was selected to be one, which is equal to the upper bound of the range (1/4 ≤ Z ≤ 1) recommended for earthfill dams by FERC. The predicted downstream flood peak levels are not expected to be sensitive to this parameter. Therefore, a conservative estimate for Z was used in the modeling analysis.

- The water level in the reservoir when dam overtopping failure commences (HFDD) during either the 500-year or 1,000-year flood event is assumed to be 725.8 m, which is 0.1 m higher than the planned top-of-dam elevation.

3.2.3.2 Piping Failure

A piping failure scenario was modeled to produce the predicted dam breach flood information for the flood inundation mapping. The piping failure modeling was conducted to quantify expected flooding conditions and to assess total flood damages associated with a typical fair weather failure. For a typical fair weather failure, it is reasonable to assume that the reservoir operates at the FSL.

Table 7 presents the dam breach model parameter values selected for simulating a potential piping failure of the Highfield Dam. These parameter values are based on recommendations made by the U.S. Federal Energy Regulatory Commission (FERC 1994) and empirical formulations developed by Fread (2001), and considered site-specific characteristics of the dam.

Table 7: Selected Dam Breach Model Parameter Values for Piping Failure Simulation

Dam Breach Model Parameter	Selected Value
Average Width of Breach (BR)	31.9 m
Time to Failure (TFH)	1.0 hour
Horizontal Component of Side Slope of Breach (Z)	1.0
Elevation of Water When Dam Failure Commences (HFDD)	723.0 m

The rationale for the selected model parameter values is provided below.

- For piping failure modeling, BR was selected to be 5 times the water depth at FSL (H) in the Highfield Reservoir, which is 3.5 times the maximum height of the dam (HD) for the Highfield Dam. This ratio falls in the middle of the range (1HD ≤ BR ≤ 5HD) recommended for earthfill dams by FERC. In comparison with BR=6HD for the overtopping failure modeling, this is a 42% reduction. The empirical equation (1) suggests BR≈5.0 HD for the Highfield Dam.

- Time to failure (TFH) was selected to be 1.0 hour, which is the upper bound of the recommended range (0.1 hour ≤ TFH ≤ 1.0 hour) for earthfill dams by FERC. The empirical equation (2) suggests TFH = 2.8 hours for the Highfield Dam. The predicted downstream flood peak levels are not expected to be sensitive to this parameter. A less conservative estimate of TFH was made for the piping failure modeling because breaches associated with piping failures generally take longer time to develop compared to those for overtopping failures.
- The horizontal component of the side slope of the breach (Z) was selected to be 1.0, which is equal to the upper bound of the range (1/4 ≤ Z ≤ 1) recommended for earthfill dams by FERC. The predicted downstream flooding is not expected to be sensitive to this parameter.
- The reservoir water level when dam failure commences (HFDD) for piping failure was assumed to be equal to its FSL of 723.0 m.

3.2.4 Representation of the Creek Channel/Canal and Floodplains

The floodway along the Rush Lake Creek study reach was divided into two sub-reaches (i.e., upper and lower reaches), which are represented in the model as follows:

- The total length of the upper study reach is approximately 28.5 km. A total of 24 creek channel and floodplain cross sections are used in the model to represent the upper study reach.
- The total length of the lower study reach is approximately 17.2 km. A total of 21 channel/canal and floodplain cross sections are used in the model to represent the lower study reach.

Flow conveyance in the creek channel/canal and floodplains was modeled. Creek channel/canal and floodplain cross-sections used in the HEC-RAS model are based mainly on recently surveyed channel cross sections and acquired LiDAR survey data set. The locations of these cross sections are shown on the flood inundation maps.

The approach used for representing the creek channel and floodplain (based mainly on the LiDAR survey data information) is believed to provide a sufficient level of accuracy for the flood routing analyses. The channel/canal top width and floodplain width are the two key geometric parameters that affect the flood routing because of the large discharges associated with potential dam breach floods. The width estimates used in the model are sufficiently accurate for flood modeling and the flood inundation mapping work.

Table 8 provides the estimated range of Manning's n values for the creek channel and floodplains considering their bed/bank materials and meandering characteristics, and the floodplain's vegetative cover (tree, grass and farmland), man-made structures and overland form roughness.

Table 8: Estimated Manning's n Values for the Channel/Canal and Floodplains

Maximum		Best Estimate		Minimum	
Channel/Canal	Floodplain	Channel/Canal	Floodplain	Channel/Canal	Floodplain
0.050	0.080	0.040	0.065	0.030	0.050

3.2.5 Bridge and Culvert Road Crossing Modeling

There are four bridge and three culvert crossings along the upper study reach and five bridge and three culvert crossings along the lower study reach as described in Section 2.5. Bridges, culverts and the approaching road embankments can restrict flows on the creek floodplains. The major bridge decks and major road crossings (i.e., three Highway 1 bridge/culvert crossings and two CPR bridge crossings) are represented in the model using the site-specific data. It is assumed in the modeling that they would not be washed away if overtopped. This assumption would result in marginal higher water level predictions upstream of these major crossings.

3.2.6 Hydraulic Conditions for Dam Breach Flood Modeling

The initial hydraulic conditions required in the model for routing the dam breach floods are not expected to have noticeable effects on the resulting floods, particularly the resulting maximum flood levels. However, a reasonable approximation of the initial hydraulic conditions in the study reach is required to initiate the numerical computation and to provide a reasonable representation of the likely hydraulic conditions to be expected during the potential dam breach floods. The considerations, assumptions and approximations made in this study for specifying the initial hydraulic conditions along the study reach are presented below.

Initial Conditions for Modeling the Overtopping Failure Floods

The 500-year flood event or 1,000-year flood event could trigger an overtopping dam breach flood from the Highfield Dam. It is anticipated that the creek downstream of the dam would also experience wet conditions if such an event would occur upstream of the dam. It is recognized that the Highfield Reservoir would attenuate the upstream inflow and delay occurrence of the peak outflow from the reservoir. Therefore, it would be overly conservative to assume that peak reservoir outflow would coincide with a peak flood event downstream of the dam. In this study, a more reasonable assumption was made that a wet hydrologic condition would occur along the study reach. For modeling the dam overtopping failure event, the initial flow is assumed to be 19 m^3/s, which is the peak discharge for the 10-year flood event at the Highfield Dam.

Initial Conditions for Modeling the Piping Failure Flood

For piping failure modeling, it is reasonable to assume that fair weather hydrologic conditions would occur along the Rush Lake Creek study reach. For modeling the piping failure event, the initial flow is assumed to be 1.2 m^3/s, which is the peak discharge for the 2-year flood event at the Highfield Dam.

Hydraulic Boundary Condition

The hydraulic boundary condition assumed for the most downstream cross section is normal flow condition. For modeling purposes, this most downstream cross section is located downstream of the Highway 1 Culvert Crossing.

3.2.7 Modeling Results

Three model runs were made to generate flow, water depth and water velocity results for the simulated dam breach floods, including two for overtopping failure scenarios and one for the piping failure scenario. These model runs are summarized in Table 9.

Table 9: Final Model Runs for Analyzing the Highfield Dam Breach Floods

Dam Breach Scenario	Model Run No.	Inflow Flood Event	Purpose of the Model Run
Overtopping Failure	1	500-Year Flood Event	For predicting the maximum overtopping failure flood peak levels and discharges under the 500-year rainfall generated flood event
Overtopping Failure	2	1,000-Year Rainfall Flood Event	For predicting the maximum overtopping failure flood peak levels and discharges under the 1,000-year rainfall generated flood event
Piping Failure	3	Fair Weather Condition	For predicting the maximum piping failure flood peak levels and discharges associated with fair weather conditions

Figures B-8 to B-16 in Appendix B graphically present the following main modeling results for the above overtopping and piping failure floods:

- Predicted maximum flood levels, used for flood inundation mapping;
- Predicted flood peak discharges;
- Predicted maximum flood flow depths above assumed thalwegs;
- Predicted maximum flood flow depths above bankfull;
- Predicted times to maximum flood levels;
- Predicted maximum flood channel/canal flow velocities;
- Predicted maximum flood floodplain flow velocities;
- Predicted flood stage hydrographs at selected locations; and
- Predicted flood discharge hydrographs at selected locations.

Table B-1 in Appendix B compares the predicted maximum flood levels and predicted flood peak discharges of the three scenarios. The comparisons show the following:

- The maximum flood levels along the study reach associated with the 1,000-year flood overtopping failure are, on average, approximately 3.2 m and range from 0.1 m to 5.6 m above bankfull. The flood peak discharges associated with the 1,000-year flood overtopping failure are, on average, about 1,100 m^3/s.
- For overtopping failure during the 1,000-year flood event, the flood peak levels along the study reach are approximately 0.1 m higher, on average, than the overtopping failure flood associated with the 500-year flood event. The flood peak discharges along the study reach associated with the 1,000-year flood overtopping failure flood are approximately 30 m^3/s higher, on average, than those associated with the 500-year flood event.
- The maximum flood levels along the study reach associated with the piping failure are, on average, approximately 1.1 m and range from 2.1 m below to 4.3 m above tops of the channel banks. The flood peak discharges associated with the piping failure are, on average, about 310 m^3/s.

- The maximum flood levels along the study reach associated with the piping failure are approximately 2.1 m lower, on average, than those associated with the 1,000-year flood induced overtopping failure flood levels. The maximum flood levels associated with the piping failure flood are approximately 1.2 m ~ 1.6 m lower than the overtopping flood levels at the Highway 1 bridge crossings. The maximum flood levels associated with the piping failure flood are approximately 1.1 m and 3.0 m lower than the overtopping flood levels at the 1st and 2nd CPR bridge crossings, respectively. The maximum flood levels associated with the piping failure flood are approximately 3.2 m lower than the overtopping flood levels at the Highway culvert crossing. The predicted flood peak discharges along the study reach associated with the piping failure are, on average, 65% less than those associated with the overtopping failure flood.
- The maximum flood levels associated with the piping failure flood would arrive at a downstream location later than the overtopping failure floods by up to 21 hours.

The modeling results are discussed in Sections 4.2 and 4.3 in conjunction with the flood inundation maps to describe the predicted impacts and the areas affected by the potential dam breach floods.

3.3 Dam Breach Model Sensitivity Analysis

3.3.1 Purpose and Scope

A number of factors affect the accuracy and uncertainty associated with modeling dam breach floods. The main factors include the following:

- Uncertainty due to the empirical nature of the dam breach modeling formulation in the FLDWAV model and the uncertainty associated with selection of the breach model parameter values; and
- None of the known and recorded historical floods on Rush Lake Creek had the same magnitudes of flood peak discharges as the dam breach floods. It is not feasible to calibrate the hydraulic models (mainly the Manning's roughness coefficients) based on the available information.

A best estimate approach was adopted to address the modeling uncertainty and to generate best estimated modeling results. The approach is characterized by the following:

- Using best estimates of the dam breach model parameter values, which fall on the conservative side of the recommended ranges in the available literature and guidelines; and
- Using best estimates of the Manning's n values for the creek channel/canal and floodplains for predicting the maximum flood levels and the times to maximum flood levels.

Therefore, the modeling results are reasonable for predicting the potential downstream impacts of the dam breach floods. A modeling sensitivity analysis was conducted to evaluate the effects of the modeling approach. The sensitivity analysis provides an indication of the degree of uncertainty or the level of conservatism in the modeling results.

The piping failure scenario was selected for conducting the sensitivity analysis. The modeling sensitivity analysis involved three key model parameters: the average breach width, time to failure and Manning's n. The first two parameters are related to modeling dam breach, and the last parameter to the flood flow routing. In addition, one modeling sensitivity analysis of a proposed higher top-of-dam elevation (i.e., 726.7 m) was conducted for the 1,000-year flood induced overtopping failure scenario based on our discussions with AAFC. The results of the sensitivity analysis are presented in the following sections.

3.3.2 Sensitivity to Average Breach Width

The characteristics of the predicted dam breach floods are most sensitive to average breach width because this parameter controls the shape and peak of the outflow hydrograph through a dam breach. The model sensitivity analysis involves two assumed dam breach widths (BR=2HD and BR=5HD), in addition to the dam breach width (BR=3.5HD) used to predict the maximum flood levels. The results of these model runs, based on a best estimate set of the Manning's n values, are compared to quantify the model sensitivity to the assumed average breach width.

Figure B-16 in Appendix B graphically presents the differences between the predicted maximum flood levels due to piping failure of the dam. The results of the sensitivity analysis indicate the following:

- The uncertainty in the predicted maximum flood levels, based on the differences in the predicted maximum flood levels between BR=5HD and BR=3.5HD, range from 0.4 m at the dam site to 0.0 m at a location 45.7 km downstream of the dam.
- The uncertainty in the predicted maximum flood level estimates, based on the differences in the predicted maximum flood levels between BR=2HD and BR=3.5HD, range from -0.5 m at the dam site to 0.0 m at a location 45.7 km downstream of the dam.

3.3.3 Sensitivity to Time to Failure

The sensitivity analysis of model results to time to failure parameter used two additional times to failure (TFH=0.5 hour and TFH=2.0 hours), in addition to the assumed time to failure (TFH=1.0 hour) used to predict the maximum flood levels. The results of these model runs, based on a best estimate of the Manning's n values, are compared to quantify the model sensitivity.

Figure B-17 in Appendix B graphically presents the differences between the predicted maximum flood levels based on different time-to-failure assumptions. The results of the sensitivity analysis indicate the following:

- The uncertainty in the predicted maximum flood levels, based on the differences in the predicted maximum flood levels between TFH=0.5 hour and TFH=1.0 hour, is not significant.
- The uncertainty in the predicted maximum flood levels, based on the differences in the predicted maximum flood levels between TFH=2.0 hours and TFH=1.0 hour, is not significant.

These results of the sensitivity analyses indicate that the predicted differences in the maximum flood levels are negligible. However, it is noted that the time-to-failure assumption directly affects the estimates of times to maximum flood levels.

3.3.4 Sensitivity to Manning's n

The analysis of the sensitivity to Manning's n used two assumed sets of Manning's n values for the creek channel/canal and floodplains (one set corresponding to the maximum estimates of Manning's n values and the other set corresponding to the minimum estimates of Manning's n values), in addition to a set of best estimates of the Manning's n values. The results of these model runs are compared to quantify the model sensitivity.

Figure B-18 in Appendix B graphically presents the differences between the predicted maximum flood levels associated with different assumed Manning's n values. The results of the sensitivity analysis indicate the following:

- The uncertainty in the predicted maximum flood levels, range from 0.2 m (at the dam site) to 0.1 m (at 45.7 km downstream of the dam) with an average difference of 0.1 m (along the entire study reach), based on the differences in the predicted maximum flood levels between the maximum and best estimated sets of Manning's n values.
- The uncertainty in the predicted maximum flood levels, range from -0.2 m (at the dam site) to –0.1 m (at 45.7 km downstream of the dam) with an average difference of -0.1 m (along the entire study reach), based on the differences in the predicted maximum flood levels between the minimum and best estimates of Manning's n values.

The model sensitivity analysis conducted for the above three key dam breach model parameters shows that the predicted maximum flood levels are more sensitive to the assumed average breach width and less sensitive to the assumed Manning's n values, and not sensitive to the assumed time to failure. For the piping failure flood modeling, the predicted maximum flood levels along the study reach may be under-predicted by 0.4 m or over-predicted by 0.3 m, on average.

3.3.5 Sensitivity to Dam Crest Elevation

The analysis of the sensitivity to dam crest elevation used one proposed top-of-dam elevation for the Highfield Dam (i.e., a higher dam crest elevation of 726.7 m), in addition to the planned top-of-dam elevation of 725.7 m. The results of these model runs are compared to quantify the model sensitivity for the overtopping failure scenario.

Figure B-19 in Appendix B graphically presents the differences between the predicted maximum flood levels associated with the two different top-of-dam elevations. The results of the sensitivity analysis indicate that the uncertainty in the predicted maximum flood levels, range from 0.5 m (at the dam site) to 0.0 m (at 45.7 km downstream of the dam) with an average difference of 0.2 m (along the entire study reach), based on the differences in the predicted maximum flood levels between the proposed (726.7 m) and planned (725.7 m) top-of-dam elevations.

4.0 FLOOD INUNDATION MAPPING AND AFFECTED AREAS

4.1 Preparation of Dam Breach Flood Inundation Maps

The base maps used for preparing the dam breach flood inundation maps are mainly topographic maps with 1 m contour intervals, roads, watercourse, and range and township information. The flood inundation maps have been prepared at a scale of 1:20,000.

The dam breach flood inundation maps were prepared for two modeling scenarios (i.e., 1,000-year flood induced overtopping failure and piping failure). The predicted maximum flood levels for the overtopping failure scenario associated with the 500-year flood event **are not presented on the inundation maps because the average difference in the maximum flood levels is 0.1 m between the two overtopping failure scenarios**. Each flood inundation map includes the following information:

- The flooding extents delineated based on the predicted maximum dam breach flood levels at the cross sections shown on the maps, which are calculated using the HEC-RAS model. Mapping the maximum flood levels at locations between the cross sections is done using HEC-GeoRAS based on a linear interpolation of the computed maximum flood levels at the adjacent cross sections along the upper and lower study reaches. Mapping the maximum flood levels at locations between the cross sections is done based on a linear interpolation of the computed maximum flood levels at the adjacent cross sections along the middle study reach.

- Locations and labels of the cross sections used in the HEC-RAS model;
- Locations and names of the major structures or features that affect the resulting dam breach floods, including road and bridge crossings;
- A table showing the key flooding information, including the predicted maximum dam breach flood levels and times to flood peak levels; and
- A table showing the bridge, culvert and structure flooding information, including the predicted maximum dam breach flood levels, maximum flood depths, maximum flood velocity and times to maximum flood levels.

The following two sets of maps are presented in this report:

- The first is a set of dam breach flood inundation maps (Drawings 1 to 4) to show the flooding delineation on a base map with contours, roads and other mapping features for the two dam breach scenarios; and
- The second is a set of dam breach flood inundation mosaic maps (Drawings 1 to 4) to show the same flooding information as Set No. 1 but with ortho air photos as background for the two dam breach scenarios.

The values of the predicted maximum flood levels are rounded up to 0.1 m and the predicted flood travel times are rounded up to 0.1 hour. The mapping accuracy of the predicted maximum flood levels for the inundated area is generally ±0.5 m vertical distance because most of the delineation is made based on the 1 m LiDAR topographic contour information. Depending on the local floodplain overland slopes, this mapping error can lead to various degrees of horizontal accuracy. The delineation is believed to provide a sufficient definition of the areal extents of the floods. The cross sections with straight lines indicate their locations on inundation maps, which are generally different from those used in HEC-RAS models.

4.2 Residences and Areas Affected by the Dam Failure Floods

The Village of Rush Lake would not be affected under any of the dam failure flood scenarios analyzed in this study. In addition, 17 other residences and buildings on the floodplains of the Rush Lake Creek, which are situated outside of Rusk Lake, would likely not be affected. There is, however, only one house downstream of CPR embankment, which is situated on the floodplains of the Rush Lake Creek that would likely be affected by the overtopping failure flood. Table B-2 in Appendix B presents the key flooding information for the residences and areas downstream of the Highfield Dam for the two dam failure scenarios.

4.3 Main Structures and Areas Affected by the Dam Failure Floods

The key flooding information for the main structures downstream of the Highfield Dam is summarized in Table B-3 in Appendix B for the two dam failure scenarios.

- In the event of an overtopping failure or a piping failure of the Highfield Dam, sections of the Highway 1 embankments (both east and west bounds) approximately 3 km west of the Village of Rusk Lake would be overtopped, and the total lengths of the highway sections that would be flooded are 1.5 km and 0.6 km, respectively.

- In the event of an overtopping failure or piping failure of the Highfield Dam, the 1st CPR bridge embankment approximately 2.5 km west of the Village of Rush Lake would not be overtopped. However, the CRP bridge crossing would likely be damaged.
- In the event of a piping failure of the Highfield Dam, the 2nd CPR bridge embankment approximately 5 km east of the Village of Rusk Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, this 2nd CPR bridge embankment would likely be overtopped, and the total length of the flooded section is 4 km.
- In the event of a piping failure of the Highfield Dam, one low section of the Highway 1 embankment approximately 6 km east of the Village of Rusk Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, this low embankment would likely be overtopped, and the total length of the flooded section is 0.6 km.
- In the event of an overtopping failure or piping failure of the Highfield Dam, the Highway 1 culvert road embankment approximately 6.5 km west of the Village of Rusk Lake would not be overtopped.
- In the event of an overtopping failure or a piping failure of the Highfield Dam, several other downstream local road bridge/culvert crossings would be overtopped and likely be damaged.

Highfield Dam

In the event of a dam overtopping failure, the maximum flood level immediately downstream of the dam is predicted to be 720.3 m or approximately 2.8 m above the top of creek bank. The time to maximum flood level is estimated to be 0.9 hour after commencement of the dam breach. The maximum flood peak discharge is estimated to be 2,550 m^3/s.

In the event of a piping failure, the maximum flood level immediately downstream of the dam is predicted to be 718.7 m, which is 1.6 m lower than the maximum flood level associated with the overtopping failure. The flood arrival time is estimated to be 0.1 hour and the time to flood peak level is estimated to be 1.5 hours after commencement of the dam breach. The flood peak discharge is estimated to be 740 m^3/s.

Highway 1 Bridge Crossing (East Bound) – 26.3 km Downstream of the Highfield Dam

In the event of an overtopping failure, the Highway 1 bridge crossing (East Bound) would be overtopped and damaged. The maximum flood level at the road crossing is predicted to be 712.4 m and the corresponding maximum flow depth is approximately 1.1 m above the bridge road surface elevation (711.3 m). The flood arrival time is estimated to be 2.8 hours and the time to flood peak level is estimated to be 6.0 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 1,220 m^3/s and 1.4 m/s, respectively.

In the event of a piping failure, the maximum flood level at the road crossing is predicted to be 711.2 m and the corresponding maximum flow depth is approximately 0.1 m below the bridge road surface elevation (711.3 m), which is 1.2 m lower than the flood peak level associated with the overtopping failure. The flood arrival time is estimated to be 5.2 hours and the time to flood peak level is estimated to be 12.1 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 270 m^3/s and 1.2 m/s, respectively.

Highway 1 Bridge Crossing (West Bound) – 26.3 km Downstream of the Highfield Dam

In the event of an overtopping failure, the Highway 1 bridge crossing (West Bound) would be overtopped and damaged. The maximum flood level at the road crossing is predicted to be 712.2 m and the corresponding maximum flow depth is approximately 2.4 m above the bridge road surface elevation (709.8 m). The flood arrival time is estimated to be 2.8 hours and the time to flood peak level is estimated to be 6.1 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 1,220 m^3/s and 1.9 m/s, respectively.

In the event of a piping failure, the maximum flood level at the road crossing is predicted to be 710.6 m and the corresponding maximum flow depth is approximately 0.8 m above the bridge road surface elevation (709.8 m), which is 1.6 m lower than the flood peak level associated with the overtopping failure. The flood arrival time is estimated to be 5.2 hours and the time to flood peak level is estimated to be 12.1 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 270 m^3/s and 1.8 m/s, respectively.

1st CPR Bridge Crossing – 28.5 km Downstream of the Highfield Dam

In the event of an overtopping failure, the CPR bridge crossing would likely be damaged. The maximum flood level at the CPR crossing is predicted to be 710.5 m and the corresponding maximum flow depth is approximately 0.2 m below the bridge road surface elevation (710.7 m). The flood arrival time is estimated to be 3.2 hours and the time to flood peak level is estimated to be 6.2 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 1210 m^3/s and 6.0 m/s, respectively.

In the event of a piping failure, the maximum flood level at the road crossing is predicted to be 709.4 m and the corresponding maximum flow depth is approximately 1.3 m below the bridge road surface elevation (710.7 m), which is 1.1 m lower than the flood peak level associated with the overtopping failure. The flood arrival time is estimated to be 5.7 hours and the time to flood peak level is estimated to be 13.8 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 260 m^3/s and 5.4 m/s, respectively.

2nd CPR Bridge Crossing – 38.5 km Downstream of the Highfield Dam

In the event of an overtopping failure, the 2nd CPR bridge crossing would be overtopped and damaged. The maximum flood level at the CPR crossing is predicted to be 703.6 m and the corresponding maximum flow depth is approximately 1.5 m above the bridge road surface elevation (702.1 m). The flood arrival time is estimated to be 7.7 hours and the time to flood peak level is estimated to be greater than 63 hours after commencement of the dam breach since there is a significant flood storage area (approximately 15 km^2) downstream of the Village of Rush Lake. The maximum flood peak discharge and channel flow velocity are estimated to be 620 m^3/s and 0.1 m/s, respectively.

In the event of a piping failure, the maximum flood level at the CPR crossing is predicted to be 700.6 m and the corresponding maximum flow depth is approximately 1.5 m below the CPR embankment surface elevation (702.1 m), which is 3.0 m lower than the flood peak levels associated with the overtopping failure. The flood arrival time is estimated to be 7.7 hours and the time to flood peak level is estimated to be 37.8 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 68 m^3/s and 1.2m/s, respectively.

Highway 1 Culvert Crossing – 44.7 km Downstream of the Highfield Dam

In the event of an overtopping failure, this Highway 1 culvert crossing would not be overtopped. The maximum flood level at this culvert crossing is predicted to be 701.9 m and the corresponding maximum flow depth is approximately 0.2 m below the road surface elevation (702.1 m). The flood arrival time is estimated to be 9.7 hours and the time to flood peak level is estimated to be greater than 64 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 76 m^3/s and 0.3 m/s, respectively.

In the event of a piping failure, the maximum flood level at this culvert crossing is predicted to be 698.7 m and the corresponding maximum flow depth is approximately 3.4 m below the road surface elevation (702.1 m), which is 3.2 m lower than the flood peak level associated with the overtopping failure. The flood arrival time is estimated to be 9.3 hours and the time to flood peak level is estimated to be greater than 85 hours after commencement of the dam breach. The maximum flood peak discharge and channel flow velocity are estimated to be 14 m^3/s and 0.5 m/s, respectively.

Local Road Bridge/Culvert Crossings

In the event of an overtopping failure, all 10 local bridge and culvert crossings would be overtopped and likely be damaged. The maximum flood levels at the road crossings are predicted to be in the range of 0.4 m to 4.6 m above the bridge road surface elevations.

In the event of a piping failure, five local bridge and culvert crossings would be overtopped and likely be damaged. The maximum flood levels at these five road crossings are predicted to be in the range of 0.3 m to 2.1 m above the bridge road surface elevations.

Downstream Study Boundary – 45.7 km Downstream of the Highfield Dam

The downstream study boundary is located about 45.7 km downstream of the Highfield Dam. In the event of an overtopping failure, the maximum flood level at the downstream study boundary is predicted to be 699.0 m and the corresponding maximum flow depth is estimated to be 1.5 m above the top of channel banks. The flood arrival time is estimated to be 10.0 hours, and the time to flood peak level is estimated to be greater than 64 hours after commencement of the dam breach. The flood peak discharge is estimated to be 76 m^3/s.

In the event of a piping failure, the maximum flood level at the downstream study boundary is predicted to be 697.8 m, which is 1.2 m lower than the flood peak level associated with the overtopping failure. The maximum flow depth is estimated to be 0.3 m above the top of channel banks. The flood arrival time is estimated to be 9.6 hours and the time to maximum flood level is estimated to be greater than 85 hours after commencement of the dam breach. The flood peak discharge is estimated to be 14 m^3/s.

5.0 INCREMENTAL CONSEQUENCE CLASSIFICATION FOR THE HIGHFIELD DAM

5.1 Introduction

According to the Canadian Dam Association's 2007 Dam Safety Guidelines (CDA 2007 Guidelines), the standard of care and due diligence expected of a dam owner relates to the incremental losses due to a dam failure, that is, losses above and beyond those that would have occurred due to a natural event if the dam had not failed (CDA 2007). The incremental consequences of failure are defined as the total damage from an event with dam failure minus the damage that would have resulted from the same event had the dam not failed. The scope of work identified by AAFC includes a flood event scenario with and without overtopping failure, and a fair weather (piping) failure scenario. For a fair-weather failure scenario, the incremental consequences of a dam failure are the same as the total consequences.

According to the CDA 2007 Guidelines, the incremental consequence classification of a dam takes into consideration consequences that fall into three broad categories: (1) potential loss of life, (2) infrastructure and economic losses, and (3) losses of environmental and cultural values. The scope of this study specifies that consequence assessments be determined for (1) loss of life, (2) infrastructure and economics (third party damages and loss of water impacts) and (3) repair of dam breach. An assessment of environmental and cultural consequences is excluded from the scope of this study.

According to the RFP for this study, AAFC has not yet adopted a Dam Safety Management Policy; however, a draft is under review. AAFC has not adopted specific guidelines to classify its dams. In the interim, AAFC is considering a classification scheme similar to that used by the Saskatchewan Watershed Authority (SWA) under its Dam Safety Management Policy. This scheme is an adaptation of the criteria presented in the CDA 2007 Guidelines, with dollar values attached to infrastructure and economic losses. AAFC has requested that an adaptation of this scheme, shown in Table 10, shall be used in developing the consequence classification for the Highfield Dam.

Table 10: Proposed Consequence Classification Guideline for AAFC Dams

Dam Class	Population at Risk (PAR) [Note 1]	Incremental Losses*			
		Loss of Life [Note 2]	Environmental and Cultural Values	Third Party Damages	Estimated Restoration Costs Following Failure
Low	None	0	Minimum short term loss. No long term loss.	Low economic losses; area contains limited infrastructure or services. < $1.0 Million	<$1.0 Million
Significant	Temporary PAR only	Unspecified	No significant loss/deterioration of fish/wildlife habitat. Loss of marginal habitat only. Feasibility/practicality of restoration or compensation is high.	Losses to recreational facilities, seasonal workplaces, and lower use transportation routes. <$10 Million	<$10 Million
High	Permanent PAR present	10 or less	Significant loss/deterioration of important fish/wildlife habitat. Feasibility/practicality of restoration in kind is high.	High economic losses affecting infrastructure, public transportation and commercial facilities. <$25 Million	<$25 Million
Very High	Permanent PAR present	100 or less	Significant loss/deterioration of critical fish/wildlife habitat. Feasibility/practicality of restoration in kind is low.	Very high economic losses affecting important infrastructure or services. <$100 Million	<$250 Million
Extreme	Permanent PAR present	More than 100	Loss of critical fish/wildlife habitat. Restoration or compensation in kind is impossible.	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, or major storage of dangerous substances. >$100 Million	>$250 Million

* Table adapted from Saskatchewan Watershed Authority Dam Safety Management Policy which was modified from that included in the 2007 CDA Dam Safety Guidelines to include Column *No. 6*, as well as dollar limits in Column *No. 5*. Third party damages and post-failure restoration costs of Watershed Authority Works are expressed in 2009 dollars. Class to be determined by the highest potential consequence, whether loss of life, environmental, cultural or economic losses.

Note 1. Definitions for population at risk:

None – There is no identifiable population at risk; there is no possibility of loss of life other than by unforeseeable misadventure.

Temporary – People are only temporarily in the dam-breach inundation zone (e.g., downstream camp grounds/seasonal cottage use, passing through on transportation routes, or participating in recreational activities).

Permanent – The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three classes (High, Very High, and Extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist decision making if the appropriate analysis is carried out).

Populations at risk are defined as persons who would be directly exposed to flood waters within the dam failure inundation zone of they took no action to evacuate. Loss of life estimates are a function of flood warning time, flood severity, and the level of understanding of the flood severity by the waning issuers.

Note 2. Implications for loss of life:

Unspecified – The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, nature of activity and other conditions. The requirements could correspond to a higher class. However, the design flood requirements, for example, might not be higher if the temporary population is not likely to be present during the flood season.

The modeling extent of the inundation for the Highfield Dam includes the potential floodway from the dam site to a location approximately 45.7 km downstream of the dam along the Rush Lake Creek reach as shown on Figure 1. The entire study reach was divided into two reaches for simulating dam breach floods along the Rush Lake Creek floodway. The upper Rush Lake Creek study reach extends from the Highfield Dam to the 1st CPR bridge crossing located approximately 28.5 km downstream of the dam site. The lower Rush Lake Creek study reach extends from the 1st CPR bridge crossing to the downstream study boundary. The downstream study boundary terminates just downstream of the Highway 1 (TransCanada Highway) crossing of the main drainage canal from the Rush Lake Irrigation Project. The total length of this lower study reach is approximately 17.2 km.

The main structures along the potential dam breach floodway include five major highway and railway crossings and 11 local road crossings. There is one small community (Village of Rush Lake) along the dam breach floodway, in addition to some residences and developments located on the potential dam breach floodplains.

5.2 Loss of Life

5.2.1 General

The CDA 2007 Guidelines state that, in addition to economic and environmental losses, the consequences of a dam failure should be evaluated in terms of life safety. The population at risk (PAR) in an inundated area provides an indication of the number of people exposed to the hazard. The PAR is usually classified as "permanent" or "temporary". Note 1 below Table 9 provides definitions of permanent and temporary PAR.

5.2.2 Potential Loss of Life

The potential loss of life (LOL) would be a proportion of the PAR, depending on factors such as warning time, location, elevation, flood depth and flow velocity, and time of day or night. The CDA 2007 Guidelines state that the potential for loss of life would depend on many highly uncertain and variable factors, including depth of flow, flow velocity, time of day, advance warning, etc. The CDA 2007 Guidelines also recognize that consistent estimates of expected loss of life are very difficult to develop, with no simple, reliable, or universally applicable methodology available.

5.2.3 Empirical Methods for Estimating Loss of Life

There are a number of empirical approaches for estimating the potential loss of life (LOL) from a hypothetical dam failure. McClelland and Bowles (2002) provide a detailed description and discussion of methods developed by various agencies and researchers for estimating LOL. Empirical approaches for estimating LOL can result in a range of estimates depending on whether there is adequate warning time for downstream populations to evacuate, whether there is an existing evacuation plan, the level of emergency preparedness, the topography of the downstream areas (narrow and fast flowing waters or flat and slow flowing waters but of greater depth), etc. AAFC indicates in the RFP that it is interested in a range of LOL estimates and a best estimate of LOL, reflecting the uncertainty in such estimates. Three approaches (Graham 1999, Brown and Graham 1988, and DeKay and McClelland 1993) to estimate potential loss of life from McClelland and Bowles (2002) are described in Appendix E. Appendix E is a document on the incremental consequence assessment methodology prepared by Golder for the Saskatchewan Watershed Authority (SWA) and is reproduced for this report as it is applicable for this study. The approach by Graham (1999) is used to provide the range and best estimates of LOL for this study.

The general approach suggested by Graham (1999) is to divide the PAR into subpar, classify each subPAR according to a trichotomous division of flood severity (Low, Medium, High), a trichotomous division of (official) warning time (No warning, Some Warning, Adequate warning), and a dichotomous division of flood severity understanding (Vague, Precise). Graham (1999) provides an expected (mean) value for the proportional life loss (P) for each of the 15 possible categories of flood severity, warning time, and flood severity understanding. Details on the Graham (1999) approach are provided in Appendix E.

5.2.4 Estimated Loss of Life

5.2.4.1 Permanent Population at Risk and Estimated Loss of Life

Table B-2 in Appendix B summarizes the effects of the dam breach flood event on residences in the potential floodway. Only one permanent residence (I.D. 1) was identified as potentially affected by a dam breach flood as a result of an overtopping failure of the dam during the 1000-year flood event. This residence is located about 29.2 km from the dam. It is not affected during the piping failure scenario or during the 1000-year flood event without an overtopping failure of the dam. During the flood-induced overtopping failure, the flood arrival time is about 3.3 hrs from breach initiation. The maximum flood level occurs about 7.1 hrs after breach initiation. The permanent population at risk (PAR) was estimated to be nominally three (3) at Residence I.D. 1. Table 11 shows an application of Graham (1999) that suggests an expected loss of life from the permanent PAR of three is zero (0), with both the low and high estimates also zero. The expected loss of life during a 1,000-year flood event without an overtopping dam failure is also zero, hence, the incremental LOL is zero. The piping failure scenario results in a total LOL of zero as well.

Table 11: Estimation of Loss of Life from Permanent Population Based on Graham (1999)

		Peak Flood Discharge	Mean Annual Flood Discharge	Maximum Width of Flooding	Flood Severity Index	Warning Time	Flood Severity Understanding	Flood Severity	Expected Loss of Life (LOL) as a Fraction of Population at Risk (PAR)			Population at Risk (PAR)	Range of LOL		
Structure	**Location**	**Qdf (m^3/s)**	**Q2.33 (m^3/s)**	**Wdf (m)**	**DV (m^2/s)**	**WT (mins)**	**FSU**	**FS**	**Low**	**Mid**	**High**	**Permanent PAR**	**Low LOL**	**Mid LOL**	**High LOL**
Residence I.D. 1	Within floodplain and 29 km from dam	1144	1.2	2100	1	90	V	L	0.000	0.000	0.001	3	0.0	0.0	0.0

5.2.4.2 Temporary Population at Risk and Estimated Loss of Life

The loss of life assessment also considers temporary population at risk, such as people in recreation areas adjacent to streams and traffic over bridges or crossings, and the likelihood of their exposure to the flood waves. Details on the method of estimating the temporary PAR at road or bridge crossings are provided in Appendix E.

The temporary PAR below the dam is assumed to consist primarily of traffic on the various roads and highways in the path of the dam breach flood wave. Recreational use of the reservoir or Rush Lake Creek has been assumed to zero during an extreme flood event. Adequate warning time is assumed for trains moving along the CPR line, which crosses the floodway at about 29 and 39 km downstream of the dam. In addition, the joint probability of a train being on either of the two CPR bridges in the flood way at the time that they fail during a potential dam breach is very low. Thus, the probability of loss of life at the two CPR bridges is extremely low.

Traffic counts are published for selected years for a number of local roads and highways by Saskatchewan Highways and Transportation. The nominal and conservative average daily traffic (ADT) count values used in the estimation of temporary PAR are 6,000 for Highway 1 and 100 for local roads. West and east bound traffic on the bridges on Highway 1, located about 26.3 km from dam, has been lumped together as the modeling results in Table B-5 in Appendix B indicates that the west and east bound bridges would be affected by the flood waves during all three flood scenarios simulated.

Table 12 shows that the estimated total temporary PAR at the eleven (11) road crossings and two (2) Highway 1 crossings below the dam is about 79 when a night time dam breach flood situation is considered. Using the same flood considerations (severity, warning time, severity understanding) and the fatality rates as for a permanent PAR, Table 13 shows an application of Graham (1999) that suggests an expected loss of life from the temporary PAR of 79 is zero (0.1), with both the low and high estimates also zero. Since the piping failure scenario and the 1,000-year flood scenario without overtopping failure of the dam result in less severe flooding conditions at the road and highway crossings, the expected loss of life for these scenarios is also necessarily zero. The incremental LOL for the flood scenario (with and without dam failure) is zero.

5.2.4.3 Total Loss of Life Estimate

Based on the discussion above, the estimated loss of life (LOL), including both the temporary and permanent PAR, following a hypothetical piping or overtopping breach of the dam is zero.

Table 12: Estimation of Temporary Population at Risk from Traffic Counts

Structure Downstream of Dam	Road Classification	Average Daily Traffic (ADT)* ADT	Estimated Day Traffic over 1-hr period DAY-ADT-1 hr	Estimated Night Traffic over 1-hr period** NIGHT-ADT-1 hr	Flood Arrival Time FAT (hr)	Time Between Flood Arrival Time and Time to Peak Flood Water Level TPFWL (hr)	**Estimate of DAY TIME Temporary PAR** Temporary PAR Day	Duration of Flood Overtopping of Structure D (hr)	Estimated Night Traffic over D-hr (Flood Overtopping Period)*** NIGHT-ADT-D hr	**Estimate of NIGHT TIME Temporary PAR** Temporary PAR-Night	Maximum of DAY or NIGHT PAR	
1st Local Road Crossing	Access Road	100	10	2	0	1	**1.5**	>4	4	**1.6**	1.6	
2nd Local Road Crossing	Access Road	100	10	2	1	1	**1.5**	>4	4	**1.6**	1.6	
3rd Local Road Crossing	Access Road	100	10	2	1	3	**1.5**	>4	4	**0.8**	1.5	
4th Local Road Crossing	Access Road	100	10	2	2	4	**0.0**	>4	4	**0.8**	0.8	
Highway 1 (East & West Bound)	Highway 1	6000	600	180	3	3	**0.0**	>4	360	**72.0**	72.0	
5th Local Road Crossing	Access Road	100	10	2	3	3	**0.0**	>4	4	**0.8**	0.8	
6th Local Road Crossing	Access Road	100	10	2	3	4	**0.0**	>4	4	**0.8**	0.8	
7th Local Road Crossing	Access Road	100	10	2	6	58	**0.0**	>4	4	**0.0**	0.0	
8th Local Road Crossing	Access Road	100	10	2	7	57	**0.0**	>4	4	**0.0**	0.0	
9th Local Road Crossing	Access Road	100	10	2	7	57	**0.0**	>4	4	**0.0**	0.0	
10th Local Road Crossing	Access Road	100	10	2	8	56	**0.0**	>4	4	**0.0**	0.0	
11th Local Road Crossing	Access Road	100	10	2	10	54	**0.0**	>4	4	**0.0**	0.0	
Highway 1 Culvert Crossing	Highway 1	3000	300	90	10	55	**0.0**	>4	180	**0.0**	0.0	
											79	**TOTAL**

Table 13: Application of Graham (1999) for Estimating Loss of Life from Temporary Population at Risk below Highfield Dam during an Overtopping Failure

Structure	Location	Peak Flood Discharge Qdf (m^3/s)	Mean Annual Flood Discharge Q2.33 (m^3/s)	Maximum Width of Flooding Wdf (m)	Flood Severity Index DV (m^2/s)	Warning Time WT (mins)	Flood Severity Understanding FSU	Flood Severity FS	Expected Loss of Life as a Fraction of Population at Risk: Low	Mid	High	Population at Risk: Temporary PAR*	Range of LOL: Low LOL	Mid LOL	High LOL
1st Local Road Crossing	4.8 km from dam	2106	1.2	891	2	30	V	L	0.000	0.007	0.015	1.6	0.0	0.0	0.0
2nd Local Road Crossing	6.5 km from dam	1965	1.2	967	2	30	V	L	0.000	0.007	0.015	1.6	0.0	0.0	0.0
3rd Local Road Crossing	13.3 km from dam	1417	1.2	600	2	60	V	L	0.000	0.007	0.015	1.5	0.0	0.0	0.0
4th Local Road Crossing	18.9 km from dam	1334	1.2	772	2	90	V	L	0.000	0.000	0.001	0.8	0.0	0.0	0.0
Highway 1 (East & West Bound)	26.3 km from dam	1215	1.2	1568	1	90	V	L	0.000	0.000	0.001	72.0	0.0	0.0	0.0
5th Local Road Crossing	28.4 km from dam	1211	1.2	967	1	90	V	L	0.000	0.000	0.001	0.8	0.0	0.0	0.0
6th Local Road Crossing	29.4 km from dam	1211	1.2	2068	1	90	V	L	0.000	0.000	0.001	0.8	0.0	0.0	0.0
7th Local Road Crossing	33.1 km from dam	1144	1.2	2050	1	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
8th Local Road Crossing	34.8 km from dam	1042	1.2	3107	0	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
9th Local Road Crossing	36.4 km from dam	975	1.2	3875	0	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
10th Local Road Crossing	40.2 km from dam	789	1.2	2393	0	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
11th Local Road Crossing	44.5 km from dam	256	1.2	297	1	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
Highway 1 Culvert Crossing	44.7 km from dam	76	1.2	637	0	90	V	L	0.000	0.000	0.001	0.0	0.0	0.0	0.0
												Estimated LOL from Temporary PAR	0.0	0.1	0.1

* Temporary PAR estimated from Traffic Count and Rough Probabilities of Entering Flood Zone at Crossing - see Table 11

5.3 Infrastructure and Economic Loss

Infrastructure losses are based on approximate replacement or repair costs (present value) of such structures where damages (complete or partial) are expected. Economic losses due to the loss of water from the reservoir were assessed on an understanding of the current uses of water from the reservoir.

5.3.1 Infrastructure - Road and Highway Crossings, and Residences

Crossings along the Highfield Dam breach floodway study reach include the first Highway 1 crossing about 26.3 km from the dam, the second Highway 1 crossing about 44.7 crossing from the dam, two CPR crossings: the first one 29 km form the dam and the second one 39 km from the dam, and 11 local road crossings. Table 2 provides the structural characteristics of these crossings. It is also possible that the residence (I.D. 1) located about 29 km from the dam could be damaged during an overtopping failure of the dam.

5.3.1.1 Damage to Residences

Table 14 shows that during an overtopping failure scenario, the flood depth near residence I.D. 1 is expected to be about 1.2 m, which is assumed to result in significant damage to the residence estimated as 90% of the replacement cost of the house. Assuming that the residence is valued at about $200,000, the expected damage is about $180,000 assuming the residence is not covered by insurance for "acts of God". The residence is not expected to be affected during the 1,000-year flood without an overtopping dam failure nor during a piping failure scenario.

5.3.1.2 Infrastructure Loss

Table 15, Table 16 and Table 17 show the consequences of a failure of the Highfield Dam on downstream infrastructure for the three scenarios considered: 1,000-year flood event with overtopping failure of the dam, 1,000-year flood event without overtopping failure of the dam, and piping failure of the dam, respectively. The expected repair costs of the damaged crossings are expressed as a percentage of their replacement costs. The assessment of the possible damage to these infrastructures during the dam breach flood event was based on the methodology described in Appendix E and on approximate bridge and culvert replacement costs provided by the Saskatchewan Ministry of Highways and Infrastructure.

The incremental infrastructure repair cost during the 1,000-year flood event would be the difference between the costs estimated from Table 15 and Table 16. It is apparent from these two tables that the 1,000-year flood event even without an overtopping failure of the dam could potentially result in significant damage to the crossings. The incremental repair cost at these structures is estimated at about $2,800,000 as shown in Table 18. The cost estimates are only for the purposes of the consequence assessment and should not be used for capital expenditure planning.

For the piping failure scenario, the incremental repair costs would be considered as equal to the total repair costs estimated from Table 17. Table 19 shows that the repair costs are estimated to be about $6,500,000. The cost estimates are only for the purposes of the consequence assessment and should not be used for capital expenditure planning.

Table 14: Flood Characteristics near Residences and Damage Potential during an Overtopping Failure of Highfield Dam

Residence ID	Distance from Dam (km)	Flood Arrival Time After Dam Breach (hr)	Time to Maximum Flood Level After Dam Breach (hr)	Flood Peak Discharge (m^3/s)	Maximum Flood Level (m)	Ground Elevation (m)	Flood Depth above Ground Level (m)	Estimated Damage to House as Rough Percentage of Replacement Value[1]
Residence 1	29.2	3.3	7.1	1,140	707.1	705.9	1.2	90%

[1] Criteria used for estimating damage to houses:

<u>Flood Depth above Ground Level (m)</u>	<u>Estimated Damage as Rough Percentage of House Replacement Value</u>
Greater than 2 m	115%
Between 1 and 2 m	90%
Between 0.5 and 1 m	65%
Between 0.25 and 0.5 m	30%
Between -0.25 and 0.25 m	20%
Less than -0.25 m	0%

Table 15: Expected Damages to Road and Railway Crossings during an Overtopping Failure of Highfield Dam

Crossing	Distance from Dam (km)	Maximum Channel Flow Velocity (m/s)	Maximum Flood Level (m)	Bridge Deck/Road Surface Elevation (m)	Overtopping Depth (m)	Estimated Repair Cost as a Rough Percentage of Replacement Cost[1]
1st local road crossing - Culvert	4.8	1.5	718.4	715.5	2.9	120%
2nd local road crossing - Culvert	6.5	1.9	718.0	715.0	3.0	120%
3rd local road	13.3	1.4	716.8	710.6	6.1	120%
4th local crossing – Culvert	18.9	1.2	712.9	710.5	2.4	120%
Highway 1 Bridge (East Bound)	26.3	1.4	712.4	711.3	1.1	120%
Highway 1 Bridge (West Bound)	26.3	1.9	712.2	709.8	2.4	120%
5th local road crossing - Bridge	28.4	1.0	711.9	707.3	4.6	120%
1st CPR Bridge	28.5	6.0	710.5	710.7	-0.2	100%
6th local road crossing - Bridge	29.4	3.4	707.0	706.3	0.7	120%
7th local road crossing - Bridge	33.1	1.7	703.6	703.2	0.4	90%
8th local road crossing - Bridge	34.8	0.7	703.6	701.6	2.0	100%
9th local road crossing - Bridge	36.4	0.2	703.6	700.4	3.3	90%
2nd CPR Bridge	38.5	0.1	703.6	702.1	1.5	70%
10th local road crossing - Culvert	40.2	0.1	703.6	700.3	3.3	50%
11th local road crossing - Culvert	44.5	0.3	703.5	703.0	0.5	50%
Highway 1 Culvert	44.7	0.3	701.9	702.1	-0.2	30%

[1] Criteria used for estimating damage to crossings:

Flood Depth above Crossing Deck (m)	Maximum Channel Velocity (m/s)	Estimated Damage as Percentage of Crossing's Replacement Value
Greater than 0.5 m	Greater than 2 m/s	120%
Between 0 and 0.5 m	Greater than 2 m/s	100%
Between 0 and 0.5 m	Between 1 and 2 m/s	90%
Between -0.5 and 0 m	Greater than 1 m/s	70%
Between -0.5 and 0 m	Less than 1 m/s	30%
Between -1 and -0.5 m		20%
Less than -1 m		0%

Table 16: Expected Damages to Road and Railway Crossings during a 1000-year Flood Event without Overtopping of Highfield Dam

Crossing	Distance from Dam (km)	Maximum Channel Flow Velocity (m/s)	Maximum Flood Level (m)	Bridge Deck/Road Surface Elevation (m)	Overtopping Depth (m)	Estimated Repair Cost as a Rough Percentage of Replacement Cost[1]
1st local road crossing - Culvert	4.8	0.8	716.4	715.5	0.9	90%
2nd local road crossing - Culvert	6.5	1.1	715.8	715.0	0.8	90%
3rd local road	13.3	0.6	714.7	710.6	4.0	90%
4th local crossing – Culvert	18.9	1.3	711.5	710.5	1.0	90%
Highway 1 Bridge (East Bound)	26.3	1.9	711.3	711.3	0.0	90%
Highway 1 Bridge (West Bound)	26.3	1.0	710.7	709.8	0.9	90%
5th local road crossing - Bridge	28.4	5.5	709.7	707.3	2.4	120%
1st CPR Bridge	28.5	1.8	709.7	710.7	-1.0	50%
6th local road crossing - Bridge	29.4	1.9	705.8	706.3	-0.5	70%
7th local road crossing - Bridge	33.1	0.2	703.3	703.2	0.1	50%
8th local road crossing - Bridge	34.8	0.0	703.3	701.6	1.7	30%
9th local road crossing - Bridge	36.4	0.0	703.3	700.4	3.0	30%
2nd CPR Bridge	38.5	0.0	703.3	702.1	1.2	30%
10th local road crossing - Culvert	40.2	0.2	703.3	700.3	3.0	50%
11th local road crossing - Culvert	44.5	0.3	703.3	703.0	0.3	50%
Highway 1 Culvert	44.7	0.8	701.6	702.1	-0.5	30%

[1] Criteria used for estimating damage to crossings:

Flood Depth above Crossing Deck (m)	Maximum Channel Velocity (m/s)	Estimated Damage as Percentage of Crossing's Replacement Value
Greater than 0.5 m	Greater than 2 m/s	120%
Between 0 and 0.5 m	Greater than 2 m/s	100%
Between 0 and 0.5 m	Between 1 and 2 m/s	90%
Between -0.5 and 0 m	Greater than 1 m/s	70%
Between -0.5 and 0 m	Less than 1 m/s	30%
Between -1 and -0.5 m		20%
Less than -1 m		0%

Table 17: Expected Damages to Road and Railway Crossings during a Piping Failure of Highfield Dam

Crossing	Distance from Dam (km)	Maximum Channel Flow Velocity (m/s)	Maximum Flood Level (m)	Bridge Deck/Road Surface Elevation (m)	Overtopping Depth (m)	Estimated Repair Cost as a Rough Percentage of Replacement Cost[1]
1st local road crossing - Culvert	4.8	1.0	716.8	715.5	1.3	120%
2nd local road crossing - Culvert	6.5	1.8	716.3	715.0	1.3	120%
3rd local road	13.3	1.0	715.0	710.6	4.3	120%
4th local crossing – Culvert	18.9	0.7	711.4	710.5	0.9	120%
Highway 1 Bridge (East Bound)	26.3	1.2	711.2	711.3	-0.1	80%
Highway 1 Bridge (West Bound)	26.3	1.8	710.6	709.8	0.9	120%
5th local road crossing - Bridge	28.4	0.9	709.4	707.3	2.1	120%
1st CPR Bridge	28.5	5.4	709.4	710.7	-1.3	50%
6th local road crossing - Bridge	29.4	1.8	705.7	706.3	-0.6	50%
7th local road crossing - Bridge	33.1	1.8	703.0	703.2	-0.2	70%
8th local road crossing - Bridge	34.8	0.9	700.9	701.6	-0.7	50%
9th local road crossing - Bridge	36.4	1.0	700.7	700.4	0.3	90%
2nd CPR Bridge	38.5	1.2	700.6	702.1	-1.5	0%
10th local road crossing - Culvert	40.2	0.6	700.2	700.3	-0.1	50%
11th local road crossing - Culvert	44.5	0.5	699.7	703.0	-3.3	0%
Highway 1 Culvert	44.7	0.5	698.7	702.1	-3.4	0%

[1] Criteria used for estimating damage to crossings:

Flood Depth above Crossing Deck (m)	Maximum Channel Velocity (m/s)	Estimated Damage as Percentage of Crossing's Replacement Value
Greater than 0.5 m	Greater than 2 m/s	120%
Between 0 and 0.5 m	Greater than 2 m/s	100%
Between 0 and 0.5 m	Between 1 and 2 m/s	90%
Between -0.5 and 0 m	Greater than 1 m/s	70%
Between -0.5 and 0 m	Less than 1 m/s	30%
Between -1 and -0.5 m		20%
Less than -1 m		0%

Table 18: Incremental Repair Cost Following 1,000-year Flood Event

Structure	Replacement Cost	INCREMENTAL Estimated Repair Cost as a Percentage of Replacement Cost (%)	Estimated Incremental Repair Cost
1st local road crossing - Culvert	$125,000	30%	$37,500
2nd local road crossing - Culvert	$125,000	30%	$37,500
3rd local road crossing	$50,000	30%	$15,000
4th local crossing – Culvert	$125,000	30%	$37,500
Highway 1 Bridge (East Bound)	$1,300,000	30%	$390,000
Highway 1 Bridge (West Bound)	$1,800,000	30%	$540,000
5th local road crossing - Bridge	$750,000	0%	$0
1st CPR Bridge	$500,000	50%	$250,000
6th local road crossing - Bridge	$625,000	50%	$312,500
7th local road crossing	$550,000	40%	$220,000
8th local road crossing	$625,000	70%	$437,500
9th local road crossing	$625,000	60%	$375,000
2nd CPR Bridge	$300,000	40%	$120,000
10th local road crossing - Culvert	$125,000	0%	$0
11th local road crossing - Culvert	$125,000	0%	$0
Highway 1 Culvert	$500,000	0%	$0
		INCREMENTAL REPAIR COST	**$2,800,000**

Table 19: Total Repair Cost Following Piping Failure

Structure	Replacement Cost	Estimated Repair Cost as a Percentage of Replacement Cost (%)	Estimated Repair Cost
1st local road crossing - Culvert	$125,000	120%	$150,000
2nd local road crossing - Culvert	$125,000	120%	$150,000
3rd local road crossing	$50,000	120%	$60,000
4th local crossing – Culvert	$125,000	120%	$150,000
Highway 1 Bridge (East Bound)	$1,300,000	80%	$1,040,000
Highway 1 Bridge (West Bound)	$1,800,000	120%	$2,160,000
5th local road crossing - Bridge	$750,000	120%	$900,000
1st CPR Bridge	$500,000	50%	$250,000
6th local road crossing - Bridge	$625,000	50%	$312,500
7th local road crossing	$550,000	70%	$385,000
8th local road crossing	$625,000	50%	$312,500
9th local road crossing	$625,000	90%	$562,500
2nd CPR Bridge	$300,000	0%	$0
10th local road crossing - Culvert	$125,000	50%	$62,500
11th local road crossing - Culvert	$125,000	0%	$0
Highway 1 Culvert	$500,000	0%	$0
		TOTAL REPAIR COST	**$6,500,000**

5.3.2 Loss of Recreational Benefits

Recreational users of Highfield Reservoir would include visitors, campers and fishermen. The recreational benefits from Highfield Reservoir foregone, if the latter were to be drained because of a breach of the dam, can be achieved from alternate sites. The economic cost of losing the recreational benefits of Highfield Reservoir is therefore assumed to be minimal from an "available alternative" analysis point of view.

5.3.3 Water Use Loss

The downstream users of water from Highfield Reservoir include the Herbert and Rusk Lake Irrigation Projects. The loss of water supplies from Highfield Reservoir following a dam failure could have economic costs in terms of lost agricultural production. The economic damages from the loss of water to all downstream irrigated areas have been assumed to be nominally $1,000,000 following discussions with AAFC. This cost estimate is for the purposes of the consequence assessment only.

5.3.4 Infrastructure and Economic Losses

The incremental infrastructure and economic losses during the 1,000-year flood would be the sum of the damage to residence I.D. 1 ($180,000), incremental repair costs for crossings ($2,800,000) and water use benefit losses ($1,000,000), which is equal to about $4,000,000.

The total infrastructure and economic losses during a piping failure scenario would be the sum of the total repair costs for crossings ($6,500,000) and water use benefit losses ($1,000,000), which is equal to about $7,500,000.

It is apparent that the combined infrastructure and economic losses of about $7,500,000 from a piping failure event will be the governing scenario.

5.4 Dam Repair Cost

The Highfield Dam is a zoned earthen dam. The dam is about 8 m high and has a crest length of 1,040 m at the existing top-of-dam elevation of 724.8 m. The dam's top width is about 5 m and the average dam slopes are 3H:1V. The reservoir has a surface area of 5.2 km^2 at its Full Supply Level (FSL) of 723.0 m. The reservoir behind the dam has a storage capacity of 15,130 dam^3 at FSL and a storage capacity of 25,750 dam^3 at the existing top-of-dam elevation. The discharge facilities at the Highfield Dam include one 20 m wide earth cut spillway and two low level outlet structures. The spillway is located on the west abutment of the dam. One irrigation low level outlet located near the west abutment of the dam and the other one located near the east abutment.

The cost to repair the dam was calculated from the volume of earthfill material required to back-fill the breach and a cost per cubic metre of material. The volume of earth material in the dam that would be washed away during a dam breach is estimated to be about 15,000 m^3 based on approximate dimensions of the earth-fill dam and breach dimensions as given in Table 4 (average breach width of 55 m). Assuming that it costs about $25 per cubic metre of earth material to fill and grade the breach to bring the dam back to its original dimensions and factoring in other potential costs result in a dam repair cost of about $375,000. This cost estimate is only for the purposes of the consequence assessment and should not be used for capital expenditure planning. The costs for clean-up of the existing structure after breaching, clean-up of immediate downstream reaches, new power lines, regulatory approvals, etc. were approximated as about 25% of the repair cost. The total cost of repairing a breach of the dam could be about $500,000. This cost does not include any repairs or replacements of the spillway and/or low level outlets if these structures were to fail during the breach.

5.5 Summary of Total Consequences

The estimated loss of life, including both the temporary and permanent PAR, as a result of a hypothetical overtopping or fair-weather failure of Highfield Dam is expected to be zero.

The infrastructure and economic losses are estimated to be about $7,500,000 following a piping failure of the Highfield dam.

The cost of repairing Highfield Dam, in case it is breached, is estimated at about $500,000.

5.6 Consequence Classification of Highfield Dam and Inflow Design Flood

AAFC has requested that the criteria shown in Table 10 shall be used in developing the consequence classification for the Highfield Dam. Based on these criteria and the summary of the consequences of a failure the dam as given in Section 5.5, the Dam Class of Highfield Dam is recommended to be in the Significant Consequence category.

According to the CDA 2007 Guidelines, the appropriate inflow design flood (IDF) for a dam should be based on the Consequence Classification of the dam. Table 20 shows the criteria to select an IDF for a dam in terms of the consequences related to the incremental loss of life, economic and infrastructure losses, and environmental and cultural losses following a failure of the dam. Based on the recommended Significant Consequence classification of Highfield Dam, the IDF is expected to be between the 100-year and 1,000-year flood.

Table 20: Classification of Dams and IDF Selection according to the CDA 2007 Guidelines

Dam Class	Population at Risk	Incremental Loss of Life	Incremental Infrastructure and Economic Losses	Incremental Environmental and Cultural Losses	Inflow Design Flood (IDF) – Return Period or Peak Flow
Low	None	0	Low economic losses; area contains limited infrastructure or services	Minimal Short-term loss No long-tern loss	100-year Event
Significant	Temporary only	Unspecified	Recreational facilities and seasonal workplaces	Loss of marginal fish and wildlife habitat only. Compensation in kind highly possible	Between 100-year and 1,000-year Flood Events
High	Permanent	10 or fewer	High economic losses affecting infrastructure, public, transportation and commercial facilities	Significant loss of important fish and wildlife habitat. Compensation in kind highly possible.	1/3 between the 1,000-year Flood and PMF Events
Very High	Permanent	100 or fewer	Very high economic losses affecting important infrastructure or services	Significant loss of critical fish and wildlife habitat. Compensation in kind possible but impractical	2/3 between the 1,000-year Flood and PMF Events
Extreme	Permanent	More than 100	Extreme losses affecting critical infrastructure or services	Major loss of critical fish and wildlife habitat. Compensation in kind impossible.	PMF

6.0 CONCLUSIONS

The results of this study support the following conclusions:

- The Highway 1 east bound bridge and west bound bridge can safely pass the 200-year flood and the 50-year flood, respectively. The Highway 1 embankment can safely pass a flood event between the 100-year and 200-year floods without causing a catastrophic failure of the highway embankment. The Highway 1 culvert crossing can safely pass a flood less than the 20-year flood event without causing a catastrophic failure of the structure.
- The 1st CPR bridge can safely pass a flood between the 200-year flood and the 500-year flood. The CPR embankment can safely pass the 1,000-year flood event. The 2nd CPR bridge can safely pass a flood less than the 10-year flood event. The 2nd CPR embankment can safely pass a 10-year flood event.

- A model sensitivity analysis was conducted to assess the sensitivity of the simulation results to three key dam breach model parameters and a proposed dam crest elevation. The predicted maximum flood levels are more sensitive to the assumed average breach width and the proposed dam crest elevation, less sensitive to the assumed Manning's n values, and not sensitive to the assumed time to failure. For the piping failure flood modeling, the predicted maximum flood levels along the study reach may be under-predicted by 0.4 m or over-predicted by 0.3 m, on average. The uncertainty in the predicted maximum flood levels, range from 0.5 m (at the dam site) to 0.0 m (at 45.7 km downstream of the dam) with an average difference of 0.2 m along the entire study reach, based on the differences in the predicted maximum flood levels between the proposed (726.7 m) and planned (725.7 m) top-of-dam elevations.

- The maximum flood levels along the study reach associated with the piping failure are approximately 2.1 m lower, on average, than those associated with the overtopping failure flood levels. The maximum flood levels associated with the piping failure flood are approximately 1.2 m ~ 1.6 m lower than the overtopping flood levels at the Highway 1 bridge crossings. The maximum flood levels associated with the piping failure flood are approximately 1.1 m and 3.0 m lower than the overtopping flood levels at the 1st and 2nd CPR bridge crossings, respectively. The maximum flood levels associated with the piping failure flood are approximately 3.2 m lower than the overtopping flood levels at the Highway culvert crossing. The predicted flood peak discharges along the study reach associated with the piping failure are, on average, 65% less than those associated with the overtopping failure flood. The maximum flood levels associated with the piping failure flood would arrive at a downstream location later than the overtopping failure floods by up to 21 hours.

- In the event of a piping failure or overtopping failure of the Highfield Dam, the Village of Rush Lake would not be flooded. In addition, all other residences, except one, and buildings in the study area would not be flooded. One house, downstream of the CPR bridge crossing, would likely be flooded during an overtopping failure of the Highfield Dam.

- In the event of an overtopping failure or a piping failure of the Highfield Dam, sections of the Highway 1 embankments (both east and west bounds) approximately 3 km west of the Village of Rusk Lake would be overtopped and likely be damaged, the Highway 1 culvert road embankment would not be overtopped, and the 1st CPR bridge embankment would not be overtopped. However, the CRP bridge crossing would likely be damaged.

- In the event of a piping failure of the Highfield Dam, the 2nd CPR bridge embankment approximately 5 km east of the Village of Rush Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, the 2nd CPR bridge embankment would be overtopped and likely be damaged.

- In the event of a piping failure of the Highfield Dam, the embankment of a low section of Highway 1 located approximately 6 km east of the Village of Rush Lake would not be overtopped. In the event of an overtopping failure of the Highfield Dam, this low highway embankment would be overtopped and likely be damaged.

- In the event of an overtopping failure or a piping failure of the Highfield Dam, most of 10 other downstream local road bridge/culvert crossings would be overtopped and likely be damaged.

- The study reach terminates at about 45.7 km downstream of the Highfield Dam. In the event of an overtopping failure of the Highfield Dam, the flood peak discharges at the downstream study boundary and further downstream would be less than 76 m^3/s, which is less than the 1:50-year flood peak discharge of 80 m^3/s at the Highfield Dam site. The flood arrival time is estimated to be 10.0 hours, and the time to flood peak level is estimated to be greater than 64 hours after commencement of the dam breach.

- In the event of a piping failure of the Highfield Dam, the flood peak discharges at the downstream study boundary and further downstream would be less than 14 m^3/s, which is less than the 1:10-year flood peak discharge of 19 m^3/s at the Highfield Dam site. The flood arrival time is estimated to be 9.6 hours, and the time to flood peak level is estimated to be greater than 85 hours after commencement of the dam breach.

- The estimated loss of life, including both the temporary and permanent PAR, as a result of a hypothetical overtopping or fair-weather failure of Highfield Dam is expected to be zero.

- The incremental infrastructure and economic losses during the 1,000-year flood would be the sum of the damage to residence I.D. 1 ($180,000), incremental repair costs for crossings ($2,800,000) and water use benefit losses ($1,000,000), which is equal to about $4,000,000.

  The total infrastructure and economic losses during a piping failure scenario would be the sum of the total repair costs for crossings ($6,500,000) and water use benefit losses ($1,000,000), which is equal to about $7,500,000.

  The combined infrastructure and economic losses of about $7,500,000 from a piping failure event will be the governing scenario.

- The cost of repairing Highfield Dam, in case it is breached, is estimated at about $500,000.

- Based on these dam classification criteria provided by AAFC and the consequences assessment carried out for this study, the Dam Class of Highfield Dam is recommended to be in the Significant Consequence category.

- According to the CDA 2007 Guidelines, the appropriate inflow design flood (IDF) for a dam should be based on the Consequence Classification of the dam. Based on the recommended Significant Consequence classification of Highfield Dam, the IDF is expected to be between the 100-year and 1,000-year flood.

Report Signature Page

This report presents the methodology and results of the Highfield Dam flood inundation study. Please direct any questions or clarification regarding the contents of this report to the following study team members who prepared this report.

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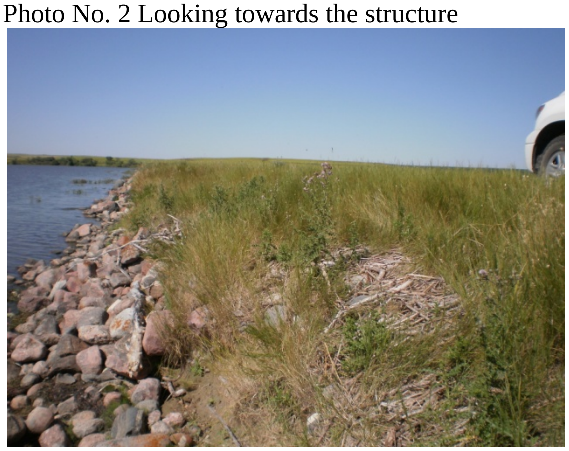
APPENDIX A

Photographs Taken during the Field Reconnaissance from August 2 to 3, 2011

Figure A-1 Photographs of the Highfield Dam and Dam Structures

Highfield Dam –Dam Upstream and Reservoir
Photo No. 1 Looking east from west end of the dam

Highfield Dam - Dam Upstream Erosion at the West Low Level Outlet
Photo No. 2 Looking towards the structure



Highfield Dam - Dam Upstream Erosion and Debris
Photo No. 3 Looking west along the dam upstream slope

Highfield Dam - Dam Downstream Slope
Photo No. 4 Looking east along the dam

Highfield Dam – Earth Spillway
Photo No. 5 Looking downstream of the spillway

Highfield Dam – West Low Level Inlet Structure
Photo No. 6 Looking west from east

Highfield Dam – West Low Level Outlet Structure
Photo No. 7 Looking downstream from the dam

Highfield Dam – East Low Level Inlet Structure
Photo No. 8 Looking towards the structure

Highfield Dam – East Low Level Outlet Structure
Photo No. 9 Looking upstream towards the structure

Figure A-2 Photographs of the Rush Lake Creek Channel and Floodplains

Rush Lake Creek, 0.1 km of Downstream of the Highfield Dam
Photo No. 1 Looking downstream from the dam

Rusk Lake Creek, 4.8 km of Downstream of the Highfield Dam
Photo No. 4 Looking downstream from a local road culvert crossing

Rusk Lake Creek, 0.1 km of Downstream of the Highfield Dam
Photo No. 2 Looking at the right bank from the left bank

Rusk Lake Creek, 6.5 km of Downstream of the Highfield Dam
Photo No. 5 Looking upstream from a local road culvert crossing

Rusk Lake Creek, 4.8 km of Downstream of the Highfield Dam
Photo No. 3 Looking upstream from a local road culvert crossing

Rusk Lake Creek, 6.5 km of Downstream of the Highfield Dam
Photo No. 6 Looking downstream from a local road culvert crossing

Rusk Lake Creek and its Floodplain in the Land Designated under Wildlife Protection Act, 8.4 km Downstream of the Highfield Dam
Photo No. 7 Looking upstream

Rusk Lake Creek, 13.3 km of Downstream of the Highfield Dam
Photo No. 10 Looking upstream from a local road crossing

Rusk Lake Creek in the Land Designated under Wildlife Protection Act, 8.4 km Downstream of the Highfield Dam
Photo No. 8 Looking at the left bank from the right bank

Rusk Lake Creek, 13.3 km of Downstream of the Highfield Dam
Photo No. 11 Looking downstream from a local road crossing

Rusk Lake Creek and its Floodplain in the Land Designated under Wildlife Protection Act, 8.4 km Downstream of the Highfield Dam
Photo No. 9 Looking downstream

Rusk Lake Creek, 18.9 km of Downstream of the Highfield Dam
Photo No. 12 Looking upstream from a local road culvert crossing


Rusk Lake Creek, 18.9 km of Downstream of the Highfield Dam
Photo No. 13 Looking downstream from a local road culvert crossing


Rusk Lake Creek, 26.3 km of Downstream of the Highfield Dam
Photo No. 14 Looking upstream from Highway 1 Bridge Crossing (East Bound)


Rusk Lake Creek Floodplain, 26.3 km of Downstream of the Highfield Dam
Photo No. 15 Looking east along Highway 1


Rusk Lake Creek, 26.3 km of Downstream of the Highfield Dam
Photo No. 16 Looking downstream from Highway 1 Bridge Crossing (West Bound)


Rusk Lake Creek, 28.4 km of Downstream of the Highfield Dam
Photo No. 17 Looking upstream from a local road bridge crossing


Rusk Lake Creek, 28.5 km of Downstream of the Highfield Dam
Photo No. 18 Looking downstream from the CPR bridge crossing

Main Drainage Canal, 29.4 km of Downstream of the Highfield Dam
Photo No. 19 Looking upstream from a local road bridge crossing

Main Drainage Canal, 33.1 km of Downstream of the Highfield Dam
Photo No. 22 Looking downstream from a local road bridge crossing

Main Drainage Canal, 29.4 km of Downstream of the Highfield Dam
Photo No. 20 Looking downstream from a local road bridge crossing

Main Drainage Canal, 34.8 km of Downstream of the Highfield Dam
Photo No. 23 Looking upstream from a local road bridge crossing

Main Drainage Canal, 33.1 km of Downstream of the Highfield Dam
Photo No. 21 Looking upstream from a local road bridge crossing

Main Drainage Canal, 34.8 km of Downstream of the Highfield Dam
Photo No. 24 Looking downstream from a local road bridge crossing

Main Drainage Canal, 38.5 km of Downstream of the Highfield Dam
Photo No. 25 Looking upstream from the 2nd CPR Bridge Crossing

Main Drainage Canal, 44.5 km of Downstream of the Highfield Dam
Photo No. 28 Looking downstream from a local road culvert crossing

Main Drainage Canal, 38.5 km of Downstream of the Highfield Dam
Photo No. 26 Looking downstream from the 2nd CPR Bridge Crossing

Main Drainage Canal, 44.7 km of Downstream of the Highfield Dam
Photo No. 29 Looking upstream from Highway 1 Culvert Crossing

Main Drainage Canal, 44.5 km of Downstream of the Highfield Dam
Photo No. 27 Looking upstream from a local road culvert crossing

Main Drainage Canal, 44.7 km of Downstream of the Highfield Dam
Photo No. 30 Looking upstream from the Highway 1 Culvert Crossing

Figure A-3 Photographs of the Rush Lake Creek Culvert and Bridge Crossings


A Local Road Culvert Crossing – 4.8 km Downstream of the Highfield Dam
Photo No. 1 Upstream end of the culvert crossing


A Local Road Culvert Crossing – 6.5 km Downstream of the Highfield Dam
Photo No. 2 Upstream end of the culvert crossing

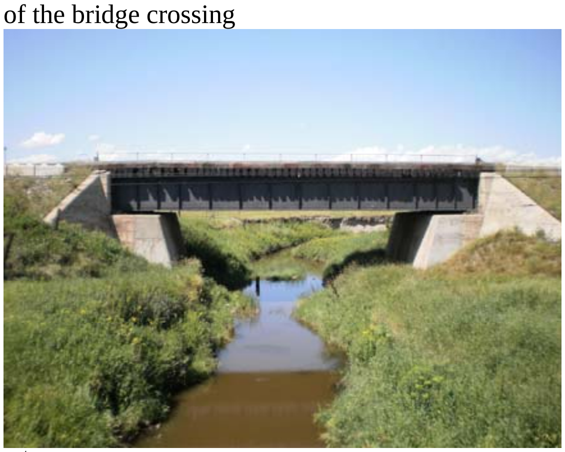

A Local Road Crossing – 13.3 km Downstream of the Highfield Dam
Photo No. 3 Looking across the local road


A Local Road Culvert Crossing – 18.9 km Downstream of the Highfield Dam
Photo No. 4 Upstream end of the culvert crossing


Highway 1 Bridge Crossing (East Bound), 26.3 km Downstream of the Highfield Dam
Photo No. 5 Looking upstream towards the bridge


Highway 1 Bridge Crossing (West Bound), 26.3 km Downstream of the Highfield Dam
Photo No. 6 Looking downstream from the Highway 1 Bridge crossing (East Bound)

A Local Road Bridge Crossing – 28.4 km Downstream of the Highfield Dam
Photo No. 7 Looking downstream from upstream of the bridge crossing



1st CPR Bridge Crossing, 28.5 km Downstream of the Highfield Dam
Photo No. 8 Looking downstream from a local road bridge

A Local Road Bridge Crossing – 29.4 km Downstream of the Highfield Dam
Photo No. 9 Looking upstream from downstream of the bridge crossing

A Local Road Bridge Crossing – 33.1 km Downstream of the Highfield Dam
Photo No. 10 Looking downstream from upstream of the bridge crossing

A Local Road Bridge Crossing – 34.8 km Downstream of the Highfield Dam
Photo No. 11 Looking downstream from upstream of the bridge crossing

A Local Road Bridge Crossing – 36.4 km Downstream of the Highfield Dam
Photo No. 12 Looking downstream from upstream of the bridge crossing

2nd CPR Bridge Crossing, 38.5 km of Downstream of the Highfield Dam
Photo No. 13 Looking downstream from the upstream of the bridge crossing

A Local Road Culvert Crossing – 40.2 km Downstream of the Highfield Dam
Photo No. 14 Upstream end of the culvert crossing

A Local Road Culvert Crossing – 44.5 km Downstream of the Highfield Dam
Photo No. 15 Downstream end of the culvert crossing

Highway 1 Culvert Crossing, 44.7 km Downstream of the Highfield Dam
Photo No. 16 Upstream end of the culvert crossing

Highway 1 Culvert Crossing, 44.7 km of Downstream of the Highfield Dam
Photo No. 17 Downstream end of the culvert crossing

Figure A-4 Photographs of the Residences on the Rush Lake Creek Floodplain

House No. 1
(GPS Readings: 325973 E, 5586566 N)

House No. 4
(GPS Readings: 328259 E, 5586370 N)

House No. 2
(GPS Readings: 327763 E, 5586151 N)

House No. 5
(GPS Readings: 328556 E, 5586471 N)

House No. 3
(GPS Readings: 327819 E, 5586487 N)

House No. 6
(GPS Readings: 328709 E, 5586532 N)

House No. 7
(GPS Readings: 328861 E, 5586590 N)

House No. 8
(GPS Readings: 328992 E, 5586670 N)

House No. 9
(GPS Readings: 329222 E, 5586747 N)

House No. 10
(GPS Readings: 329342 E, 5587226 N)

House No. 11
(GPS Readings: 329357 E, 5587157 N)

House No. 12
(GPS Readings: 329363 E, 5587046 N)

House No. 13
(GPS Readings: 329299 E, 5586794 N)

House No. 16
(GPS Readings: 330114 E, 5587009 N)

House No. 14
(GPS Readings: 329743 E, 5587157 N)

House No. 17
(GPS Readings: 330208 E, 5587075 N)

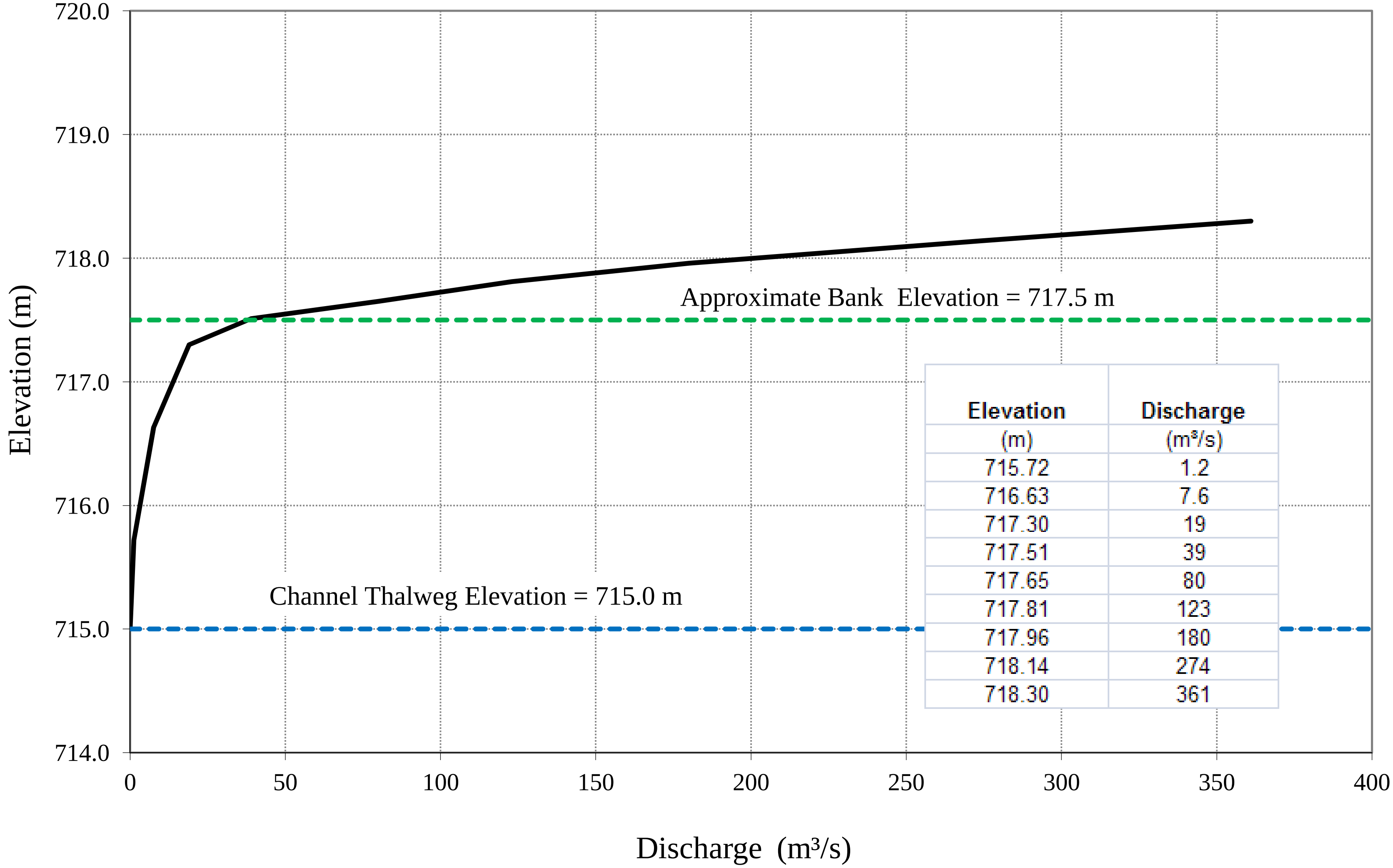
House No. 15
(GPS Readings: 329876 E, 5586959 N)

House No. 18
(GPS Readings: 338929 E, 5586575 N)

APPENDIX B

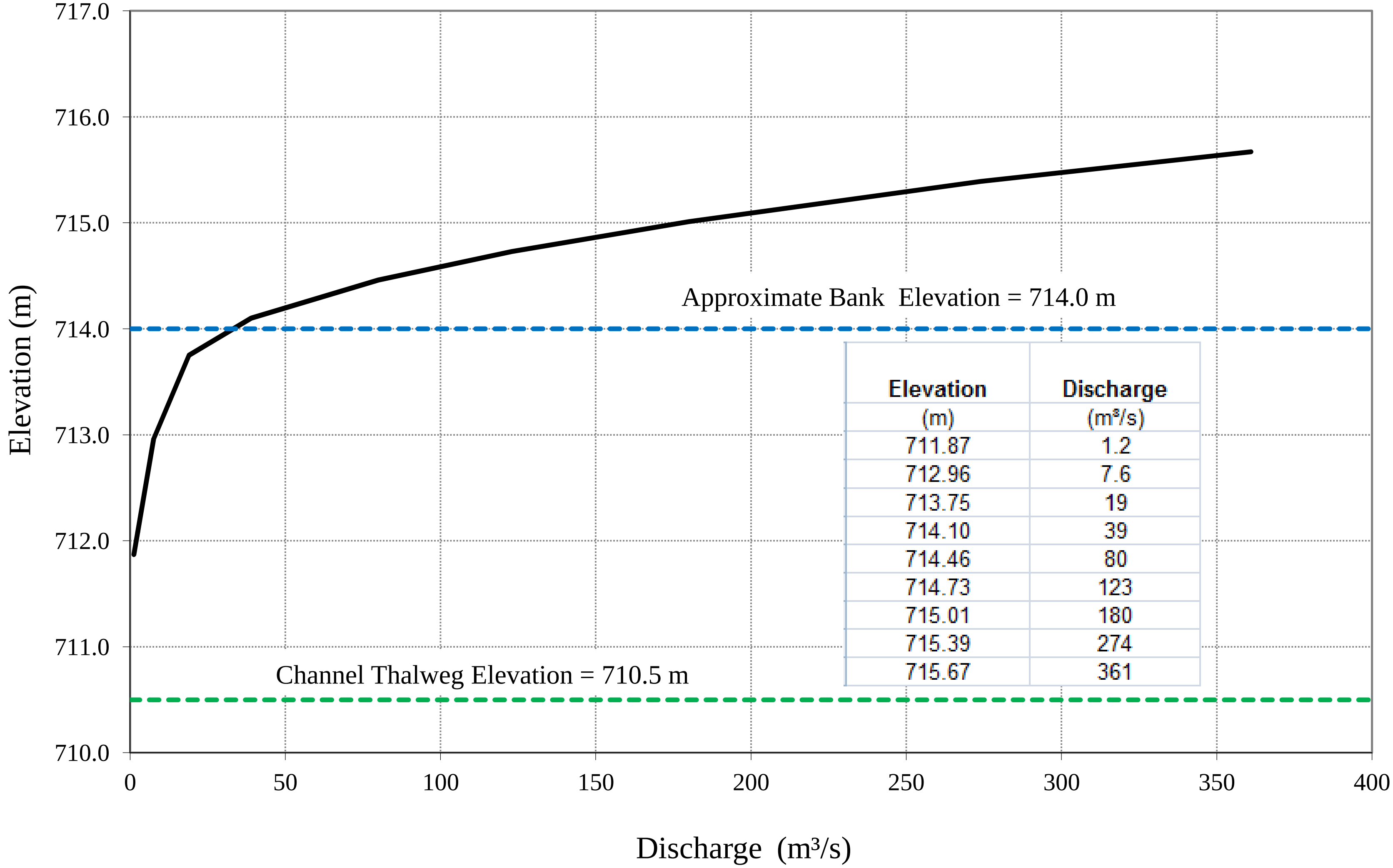
Modeling Results

Figure B-1: Tail Water Stage-Discharge Rating Curve at the Dam Site



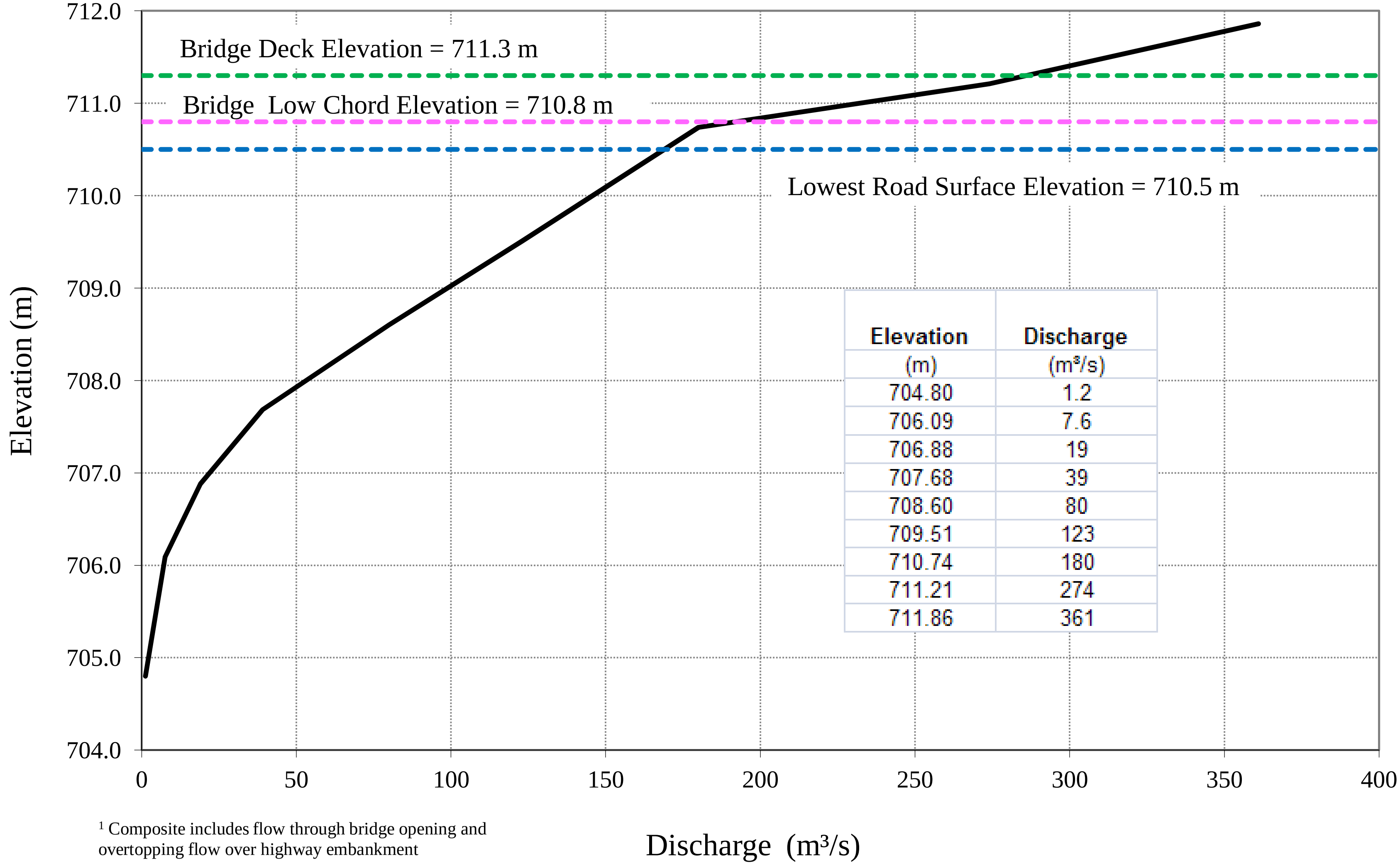
Elevation (m)	Discharge (m^3/s)
715.72	1.2
716.63	7.6
717.30	19
717.51	39
717.65	80
717.81	123
717.96	180
718.14	274
718.30	361

Figure B-2: Stage-Discharge Rating Curve at the Land Designated under Wildlife Protection Act



Elevation (m)	Discharge (m^3/s)
711.87	1.2
712.96	7.6
713.75	19
714.10	39
714.46	80
714.73	123
715.01	180
715.39	274
715.67	361

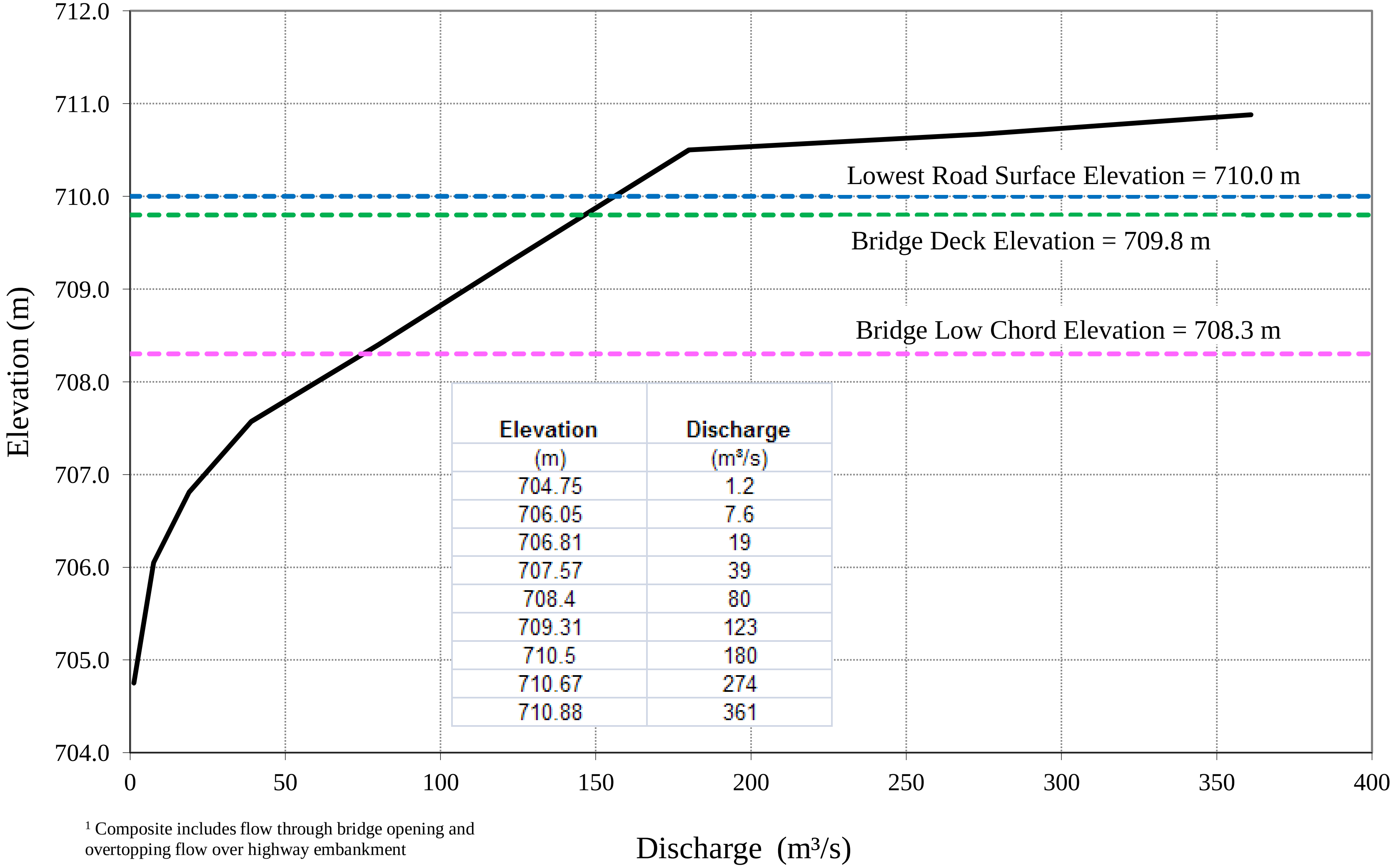
Figure B-3: Composite[1] Stage-Discharge Rating Curve at Highway 1 East Bound



Elevation (m)	Discharge (m^3/s)
704.80	1.2
706.09	7.6
706.88	19
707.68	39
708.60	80
709.51	123
710.74	180
711.21	274
711.86	361

[1] Composite includes flow through bridge opening and overtopping flow over highway embankment

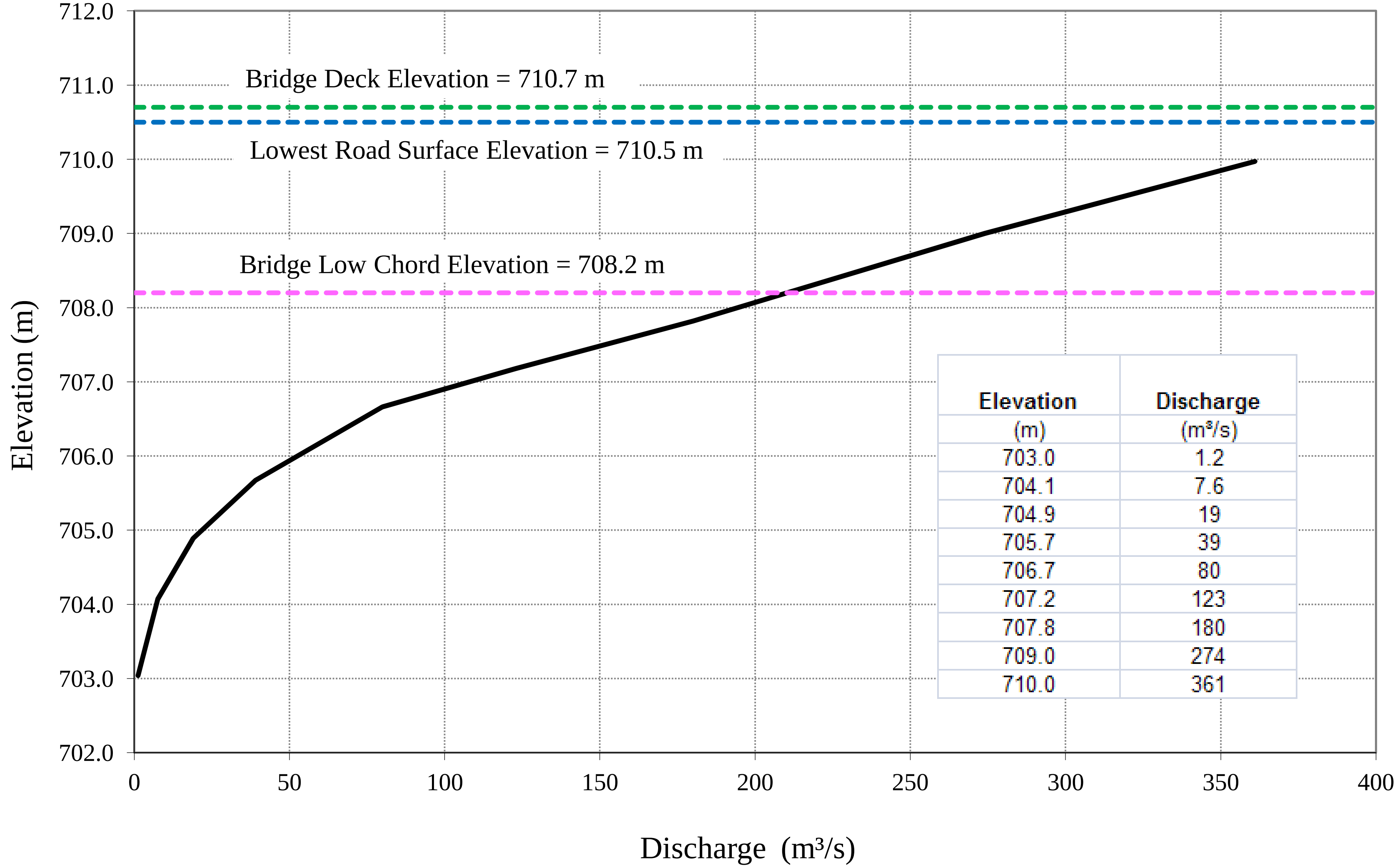
Figure B-4: Composite[1] Stage-Discharge Rating Curve at Highway 1 West Bound



Elevation (m)	Discharge (m^3/s)
704.75	1.2
706.05	7.6
706.81	19
707.57	39
708.4	80
709.31	123
710.5	180
710.67	274
710.88	361

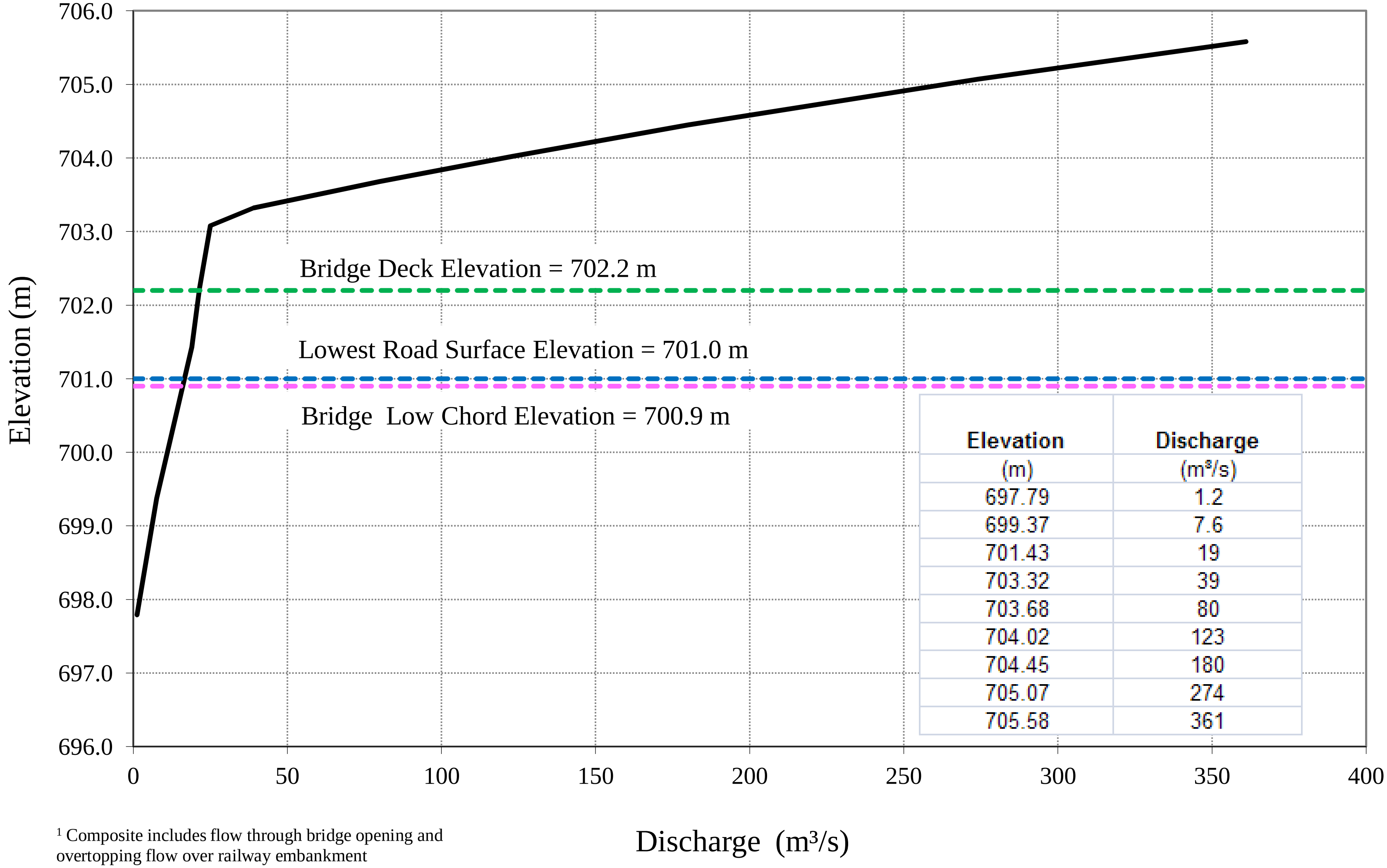
[1] Composite includes flow through bridge opening and overtopping flow over highway embankment

Figure B-5: Stage-Discharge Rating Curve at the 1st CPR Bridge



Elevation (m)	Discharge (m^3/s)
703.0	1.2
704.1	7.6
704.9	19
705.7	39
706.7	80
707.2	123
707.8	180
709.0	274
710.0	361

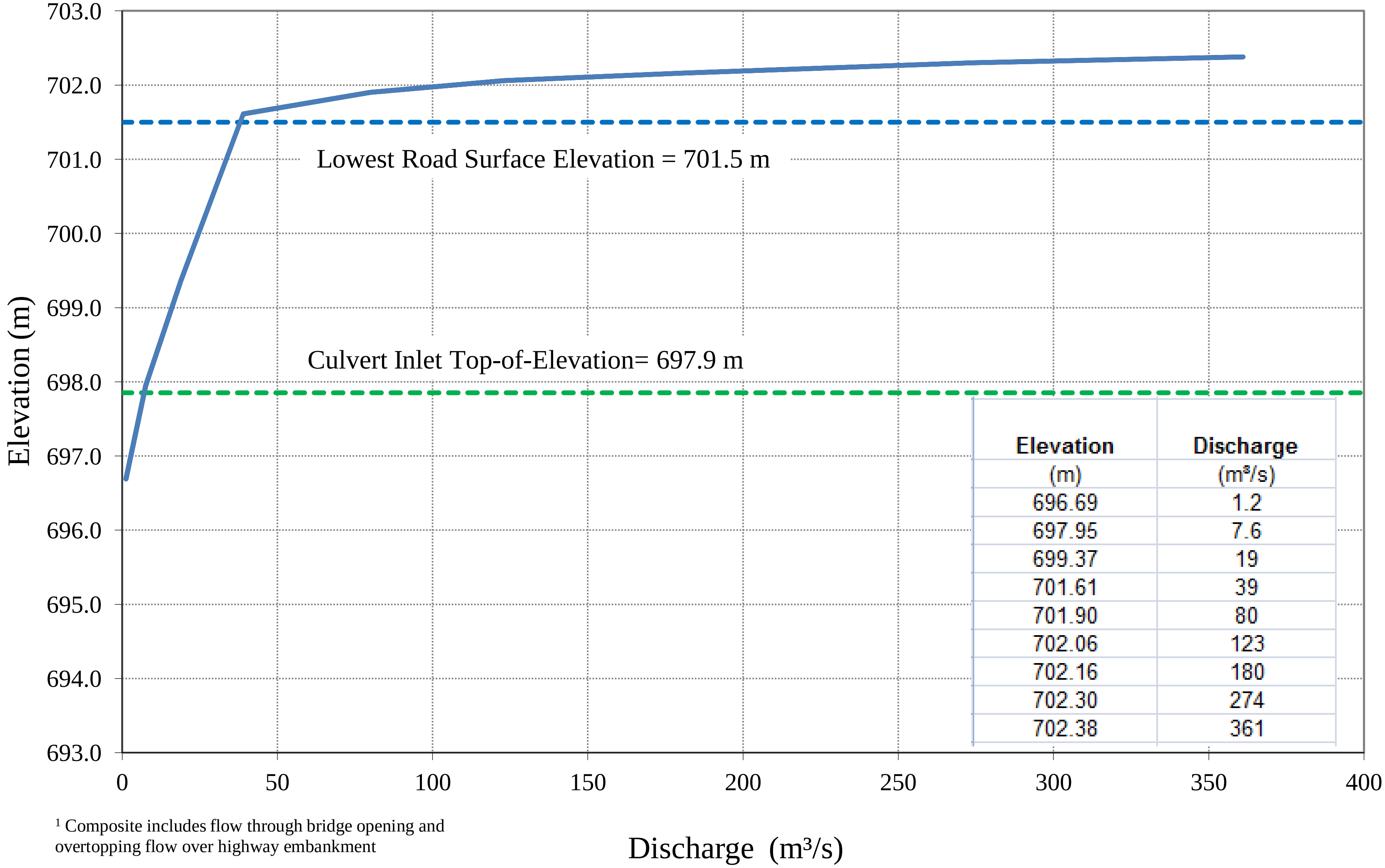
Figure B-6: Composite[1] Stage-Discharge Rating Curve at the 2nd CPR Bridge



Elevation (m)	Discharge (m^3/s)
697.79	1.2
699.37	7.6
701.43	19
703.32	39
703.68	80
704.02	123
704.45	180
705.07	274
705.58	361

[1] Composite includes flow through bridge opening and overtopping flow over railway embankment

Figure B-7: Composite[1] Stage-Discharge Rating Curve at Highway 1 Culvert



Elevation (m)	Discharge (m^3/s)
696.69	1.2
697.95	7.6
699.37	19
701.61	39
701.90	80
702.06	123
702.16	180
702.30	274
702.38	361

[1] Composite includes flow through bridge opening and overtopping flow over highway embankment

Figure B-8: Predicted Flood Peak Discharges Associated with the Dam Breach Floods

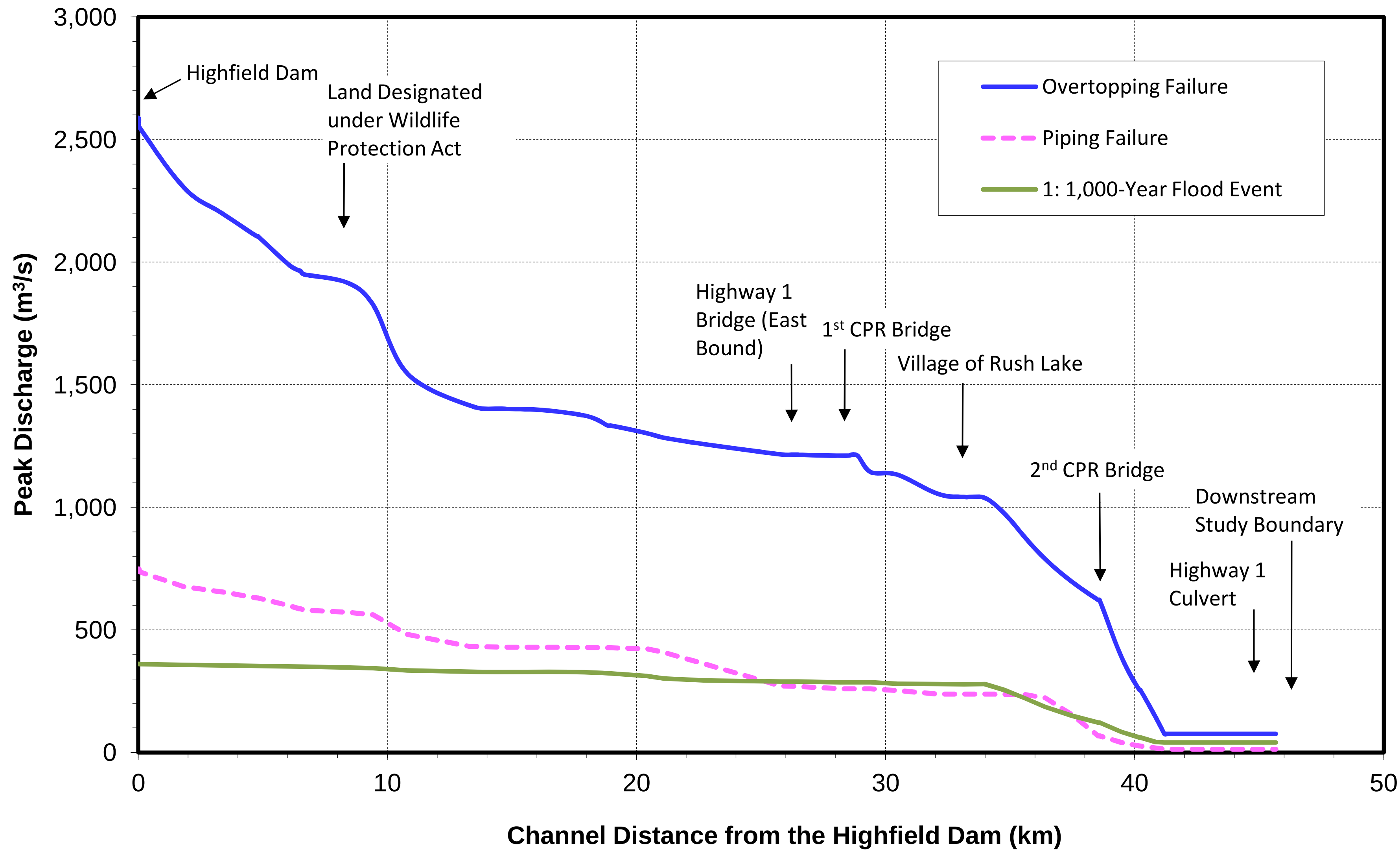


Figure B-9: Predicted Maximum Flood Levels Associated with the Dam Breach Floods

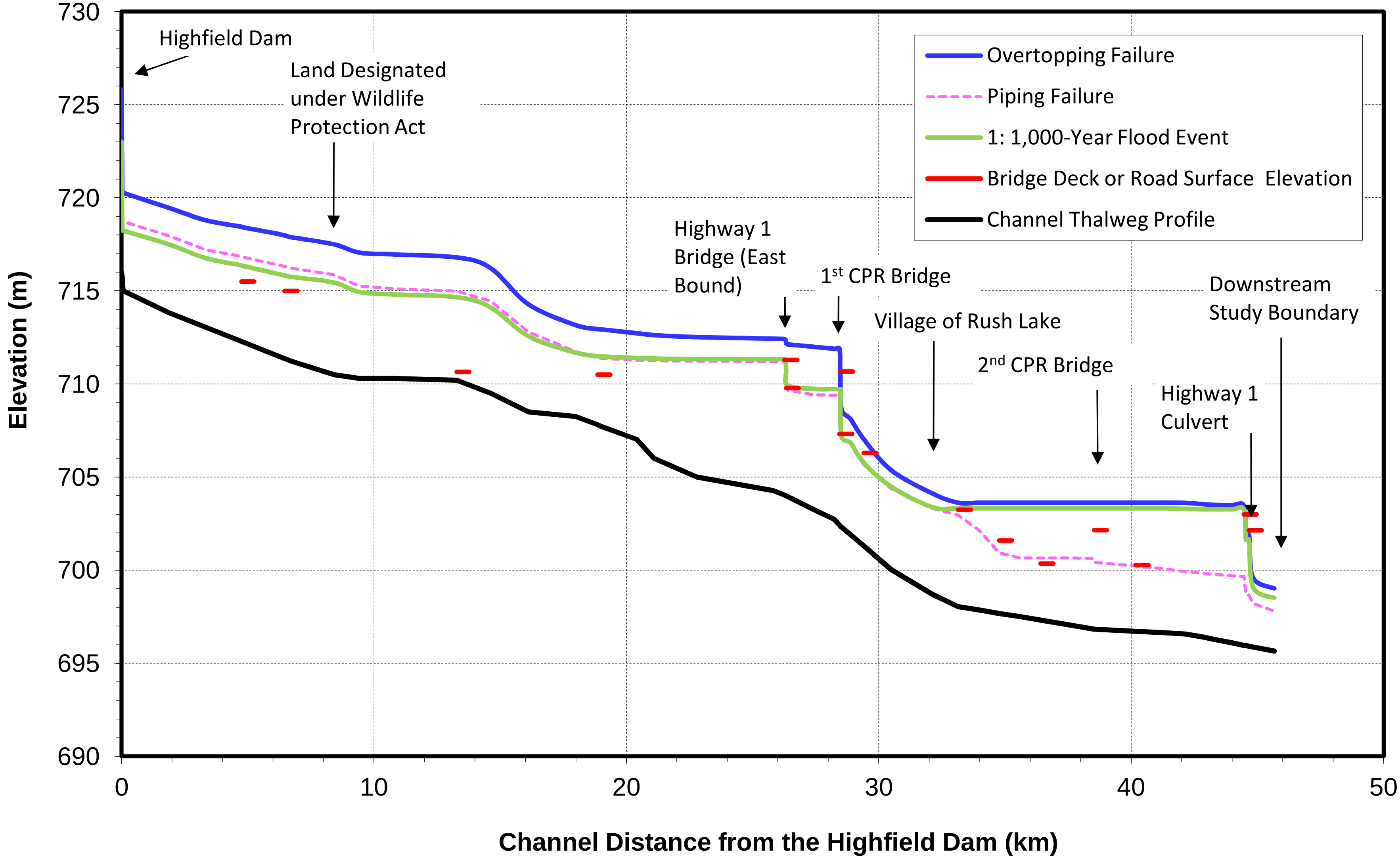


Figure B-10: Predicted Maximum Flood Depths Associated with the Dam Breach Floods

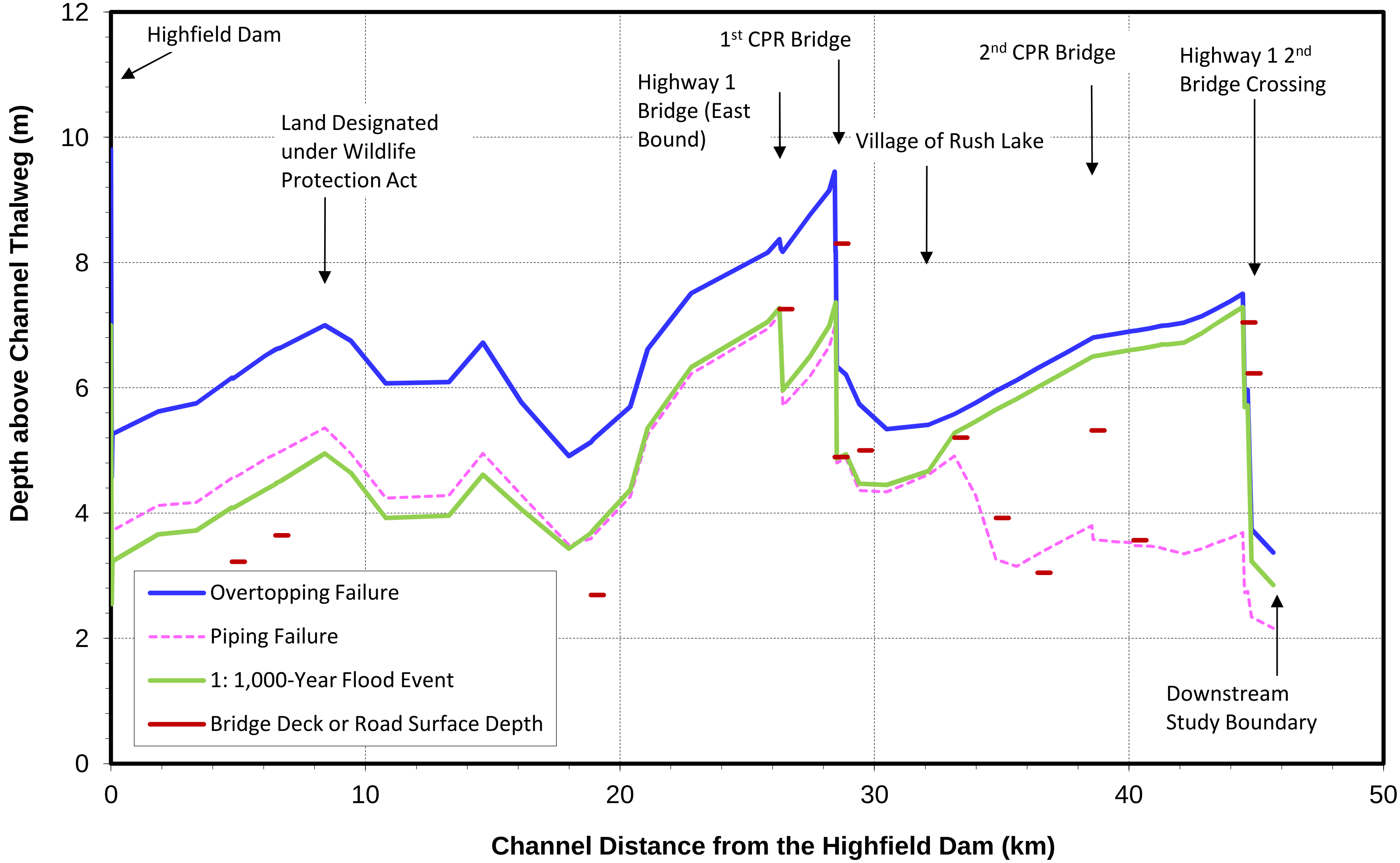


Figure B-11: Predicted Maximum Flood Depths Associated with the Dam Breach Floods

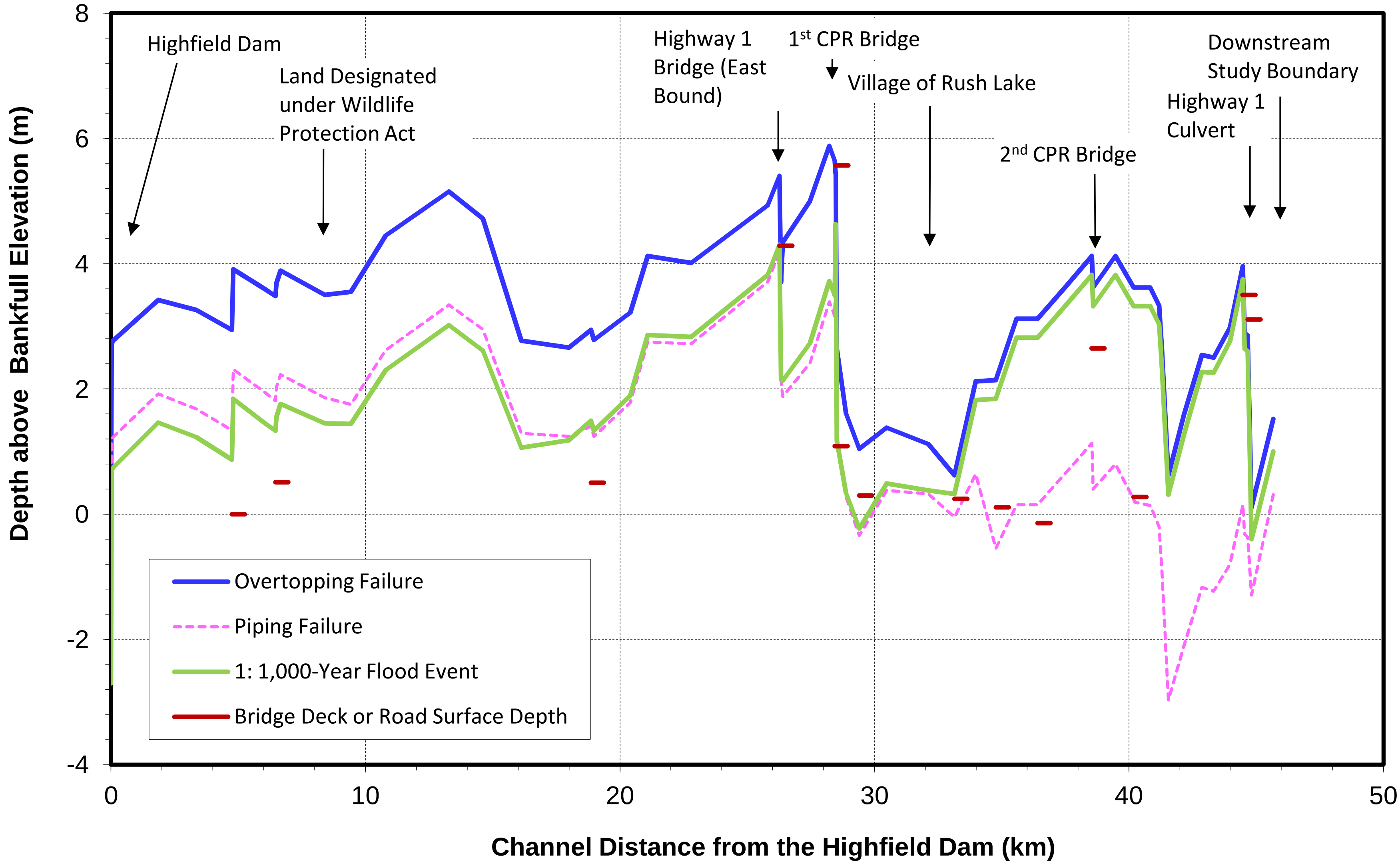


Figure B-12: Predicted Times to Maximum Flood Levels Associated with the Dam Breach Floods

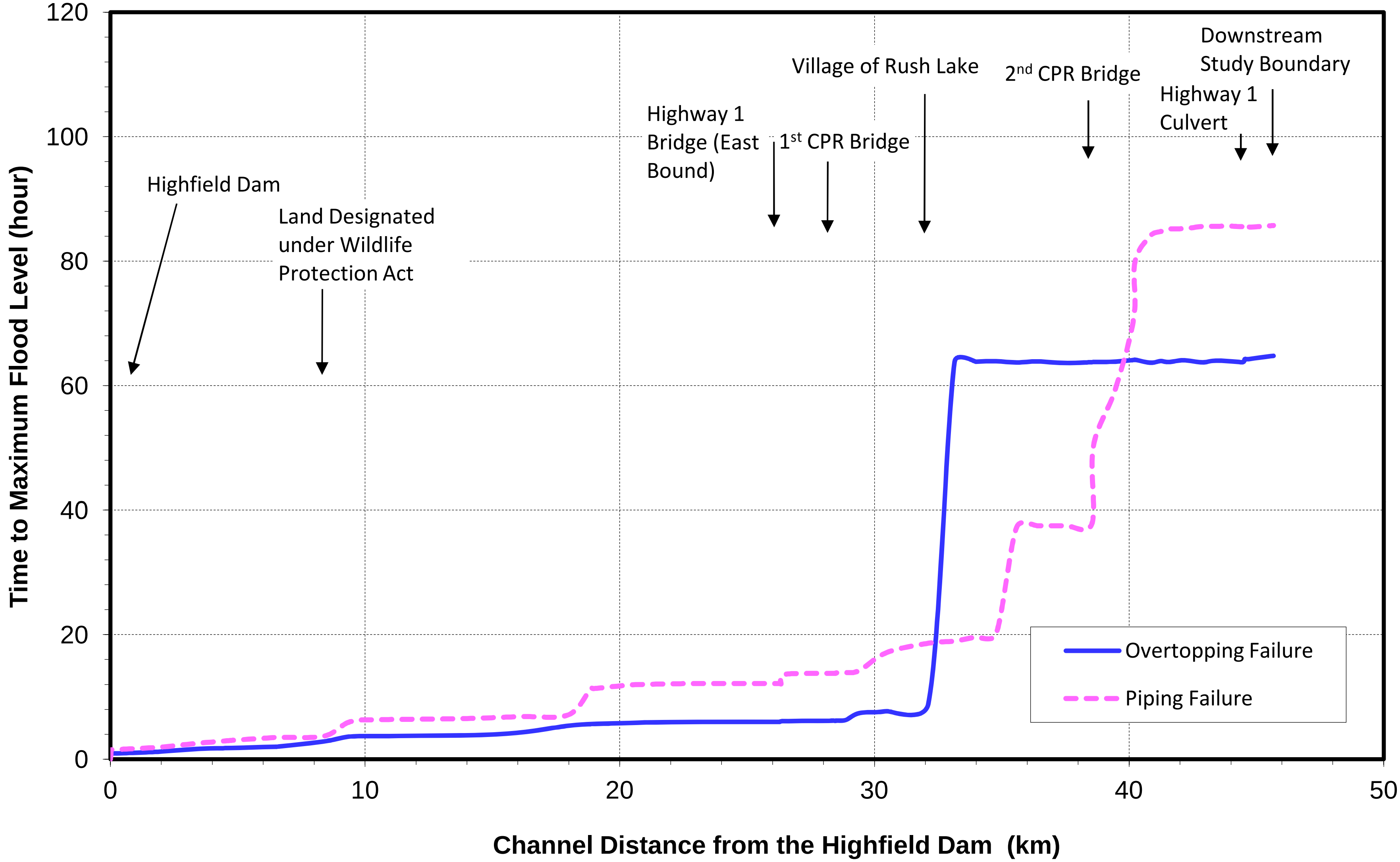


Figure B-13: Predicted Maximum Channel Flow Velocities Associated with the Dam Breach Floods

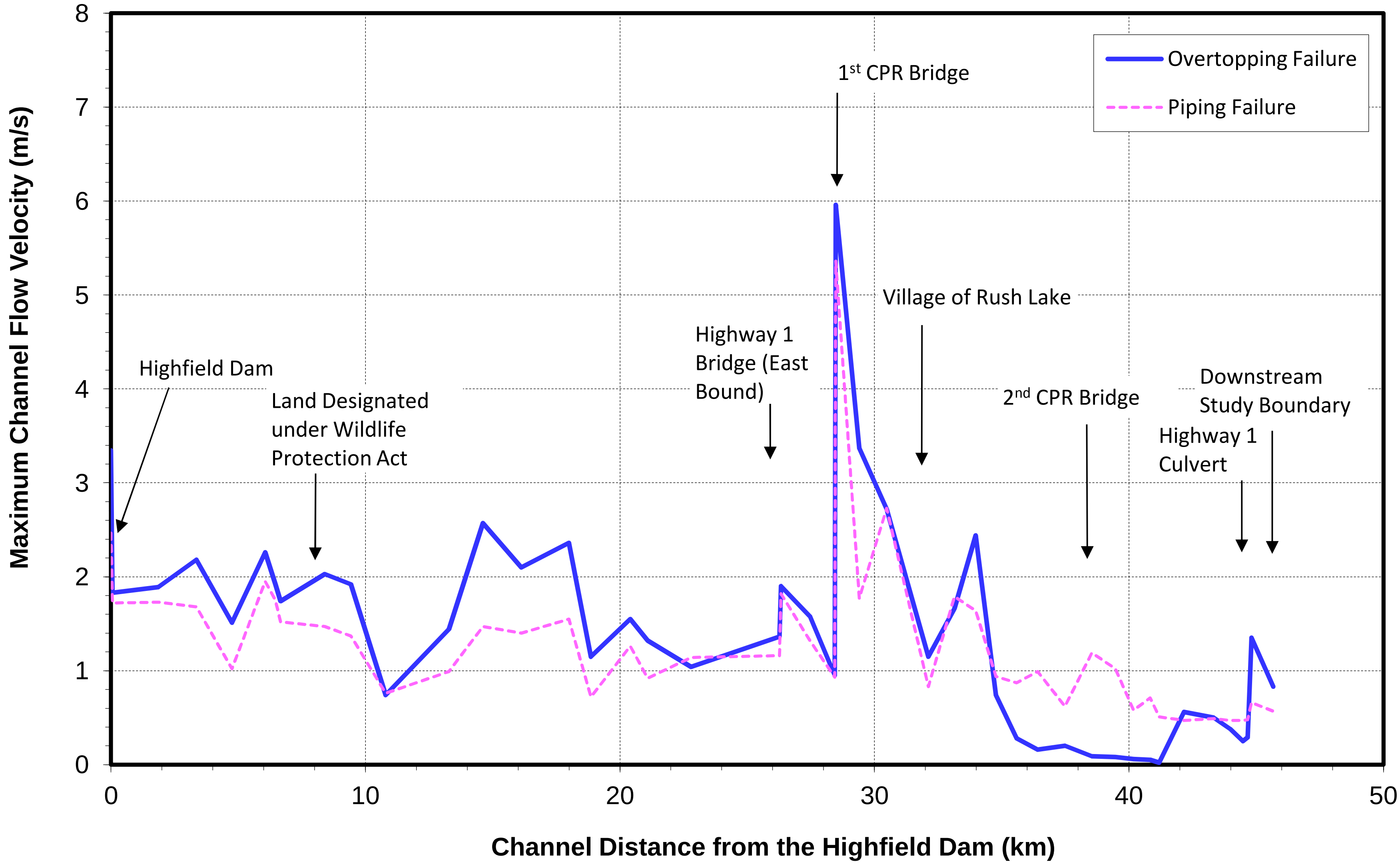


Figure B-14: Predicted Maximum Floodplain Flow Velocities Associated with the Dam Breach Floods

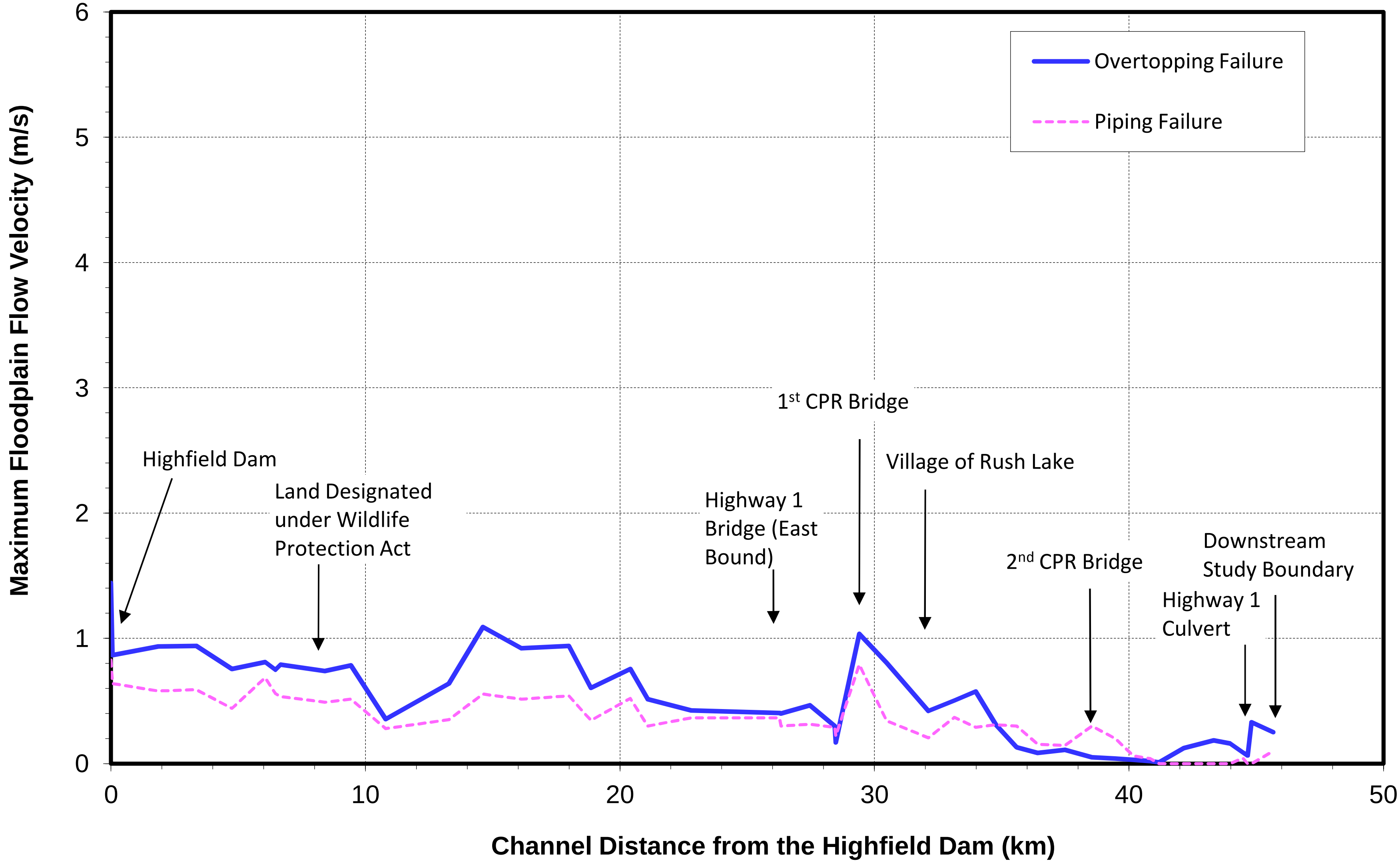


Figure B-15: Predicted Flood Discharge Hydrographs Associated with the Highfield Dam Piping Failure Flood

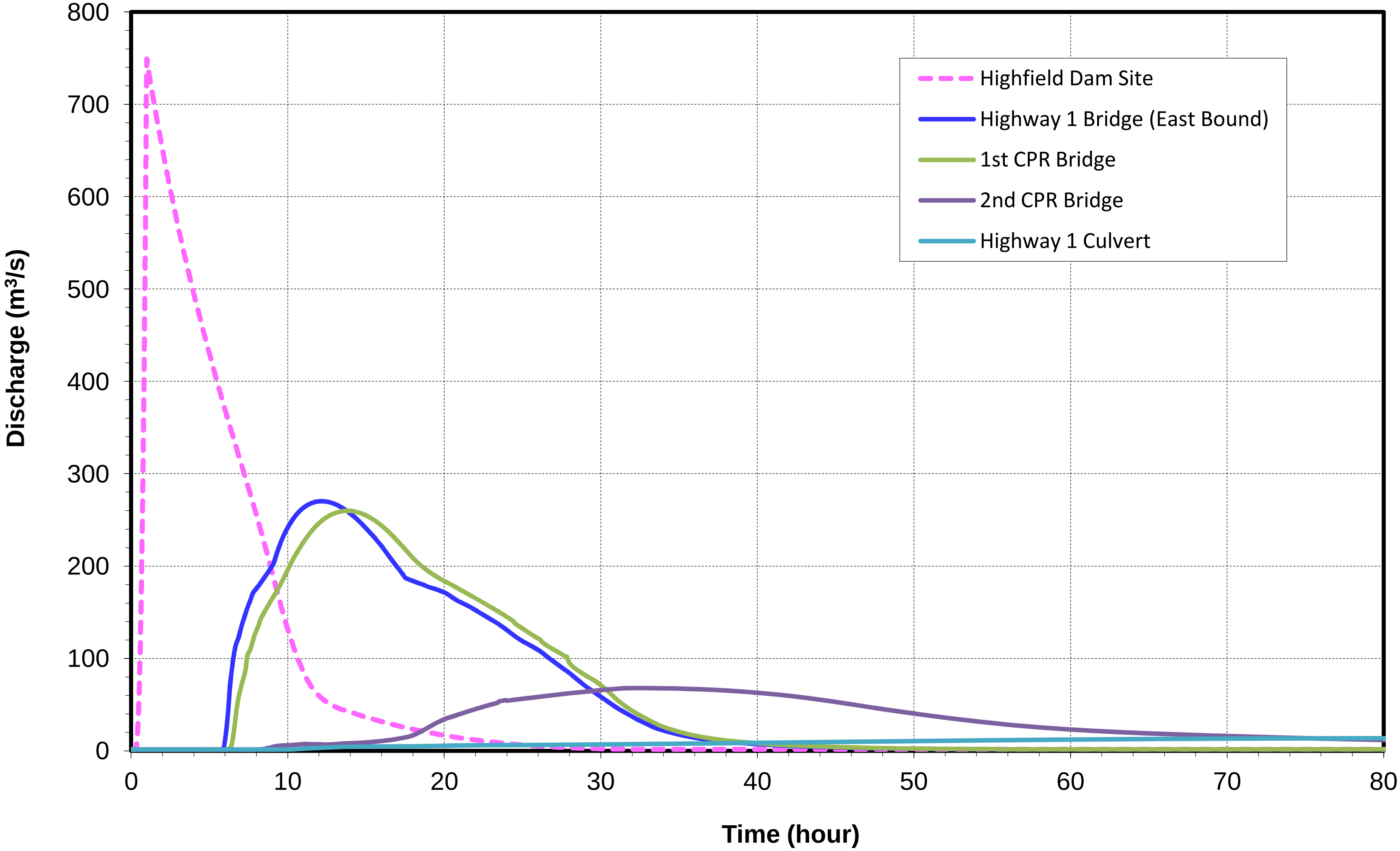


Figure B-16: Predicted Flood Stage Hydrographs Associated with the Highfield Dam Piping Failure Flood

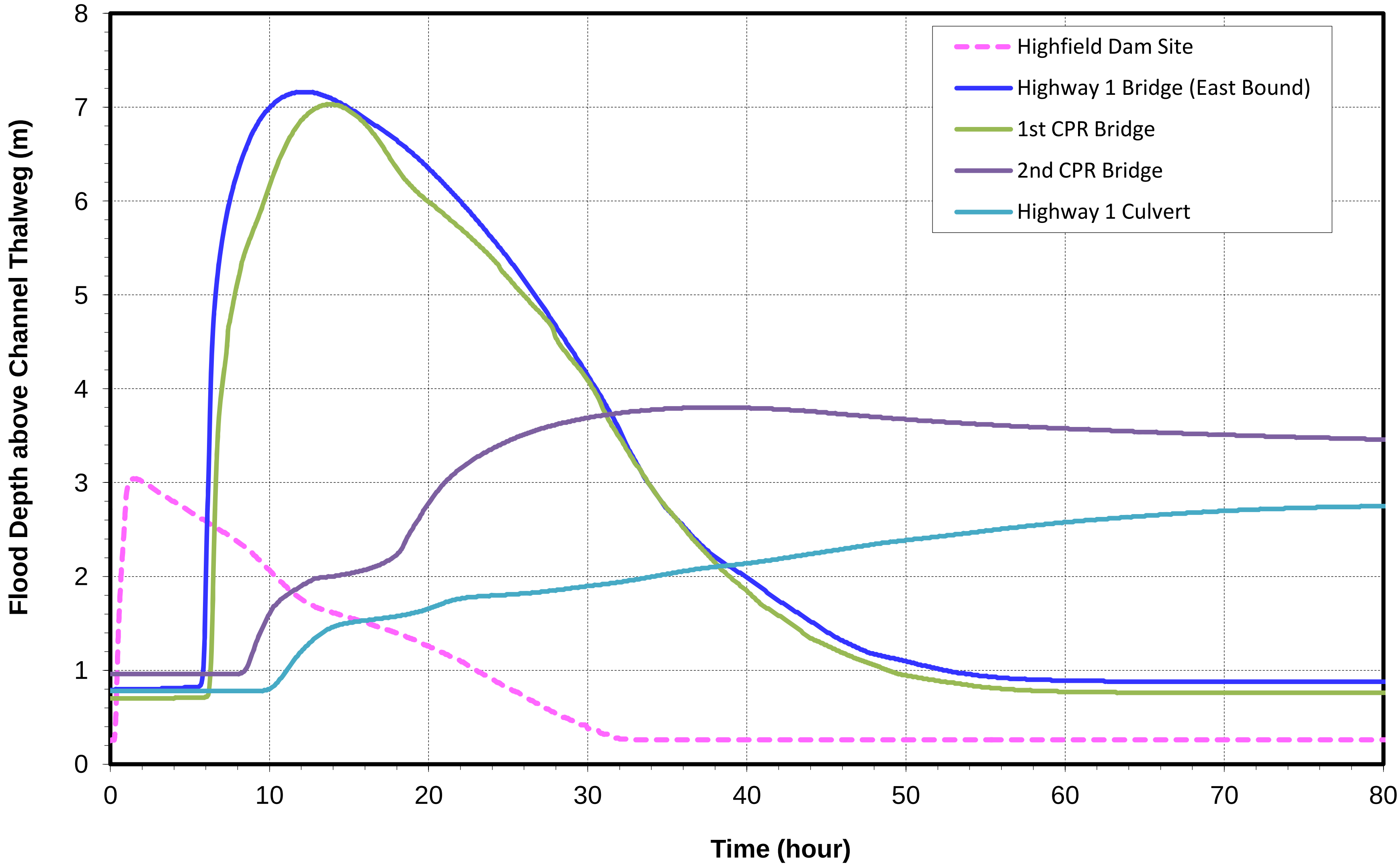


Figure B-17: Sensitivity of Predicted Maximum Flood Levels to the Assumed Dam Breach Widths for the Highfield Dam Piping Failure

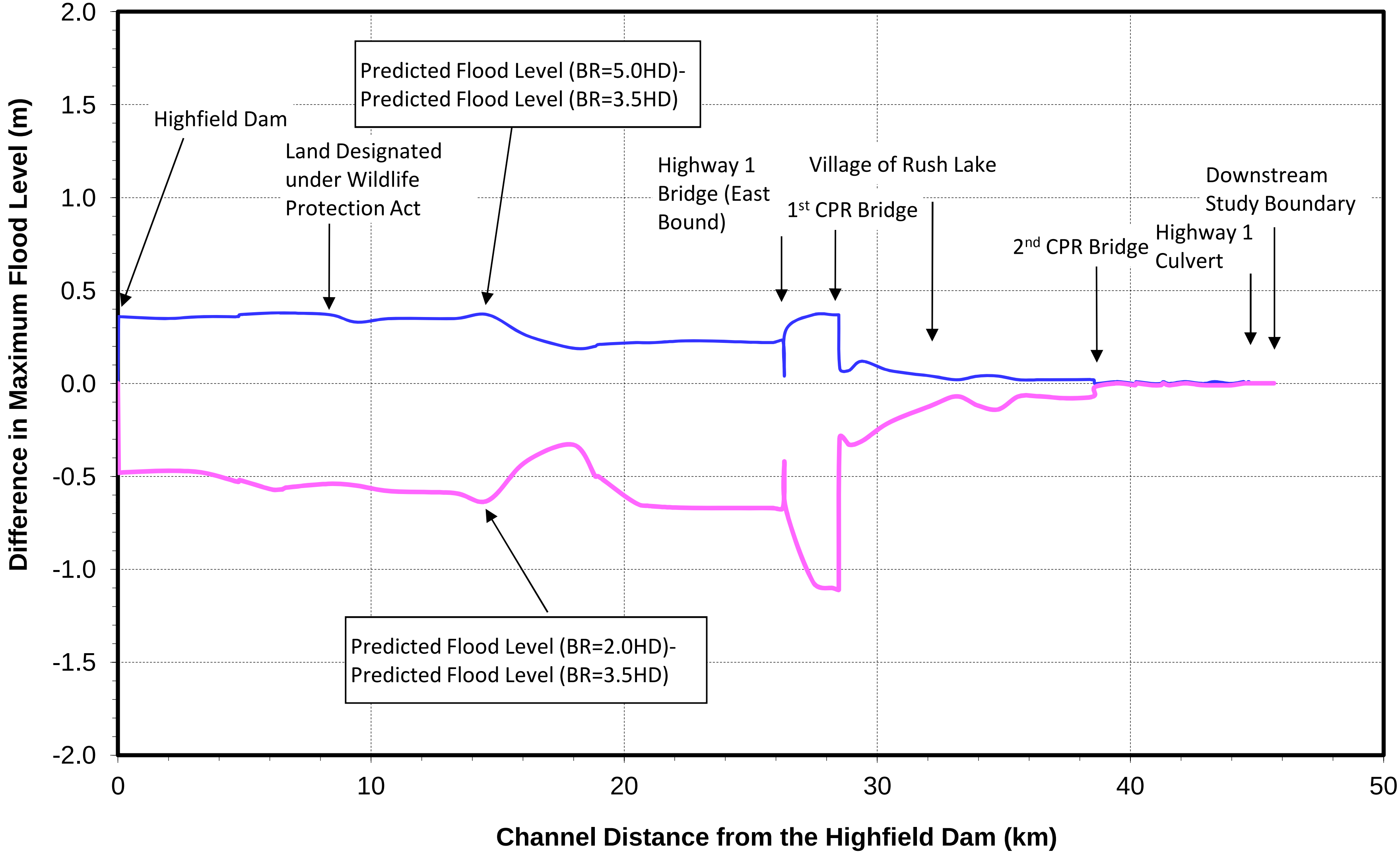


Figure B-18: Sensitivity of Predicted Maximum Flood Levels to the Assumed Times to Failure for the Highfield Dam Piping Failure

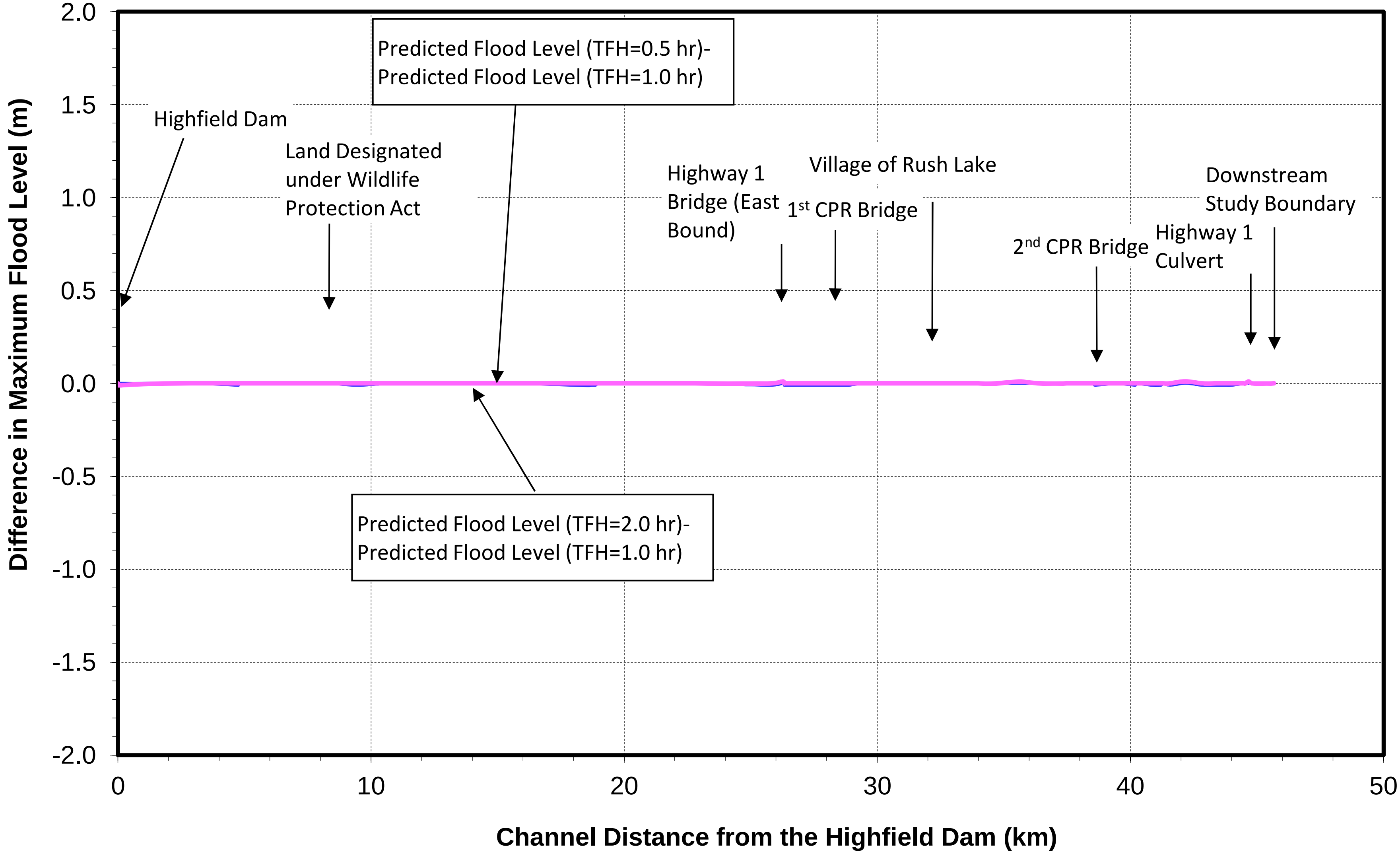


Figure B-19: Sensitivity of Predicted Maximum Flood Levels to the Assumed Manning's n for Routing the Highfield Dam Piping Failure

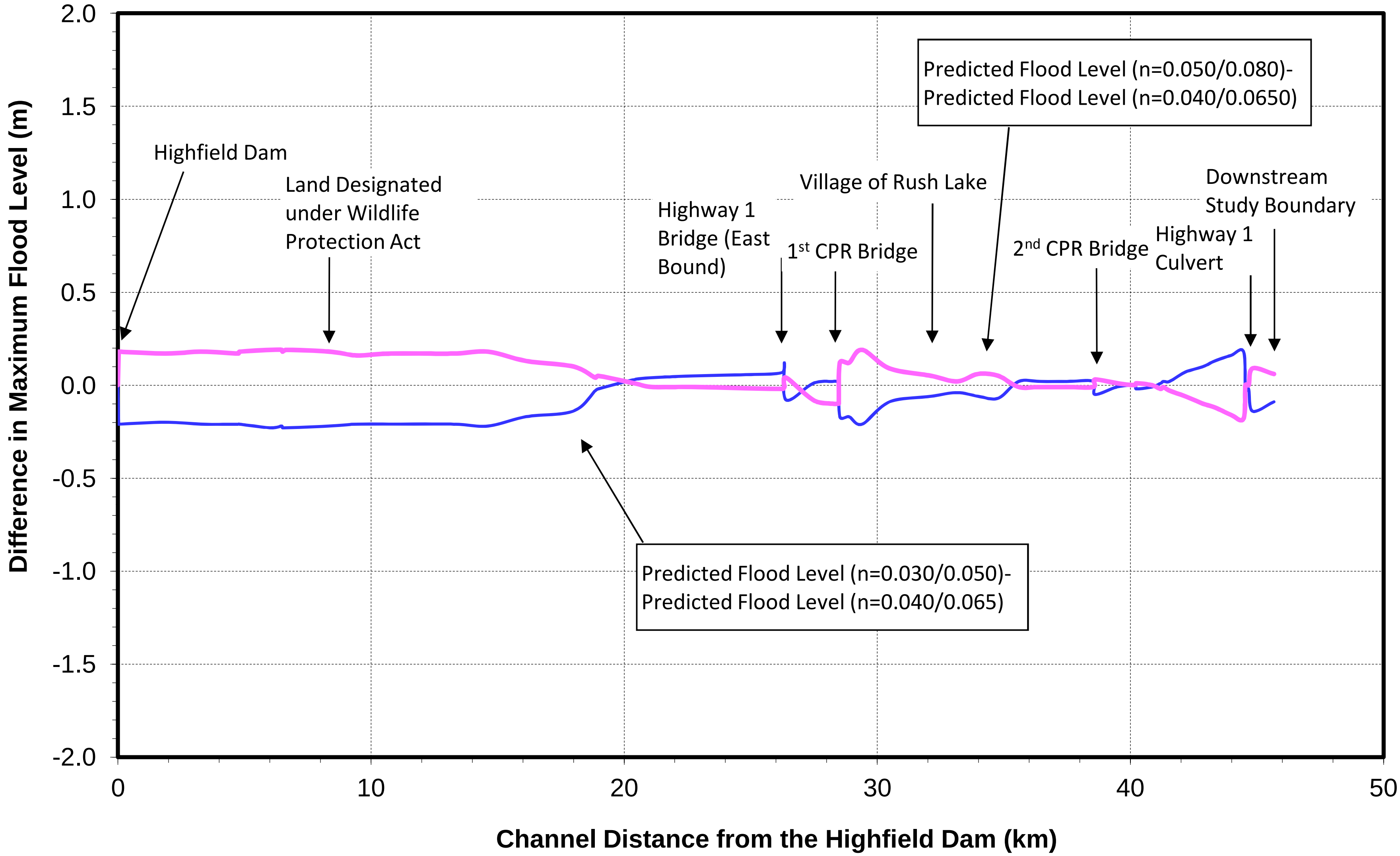


Figure B-20: Sensitivity of Predicted Maximum Flood Levels for the Proposed Highfield Dam Crest Elevation under Overtopping Failure

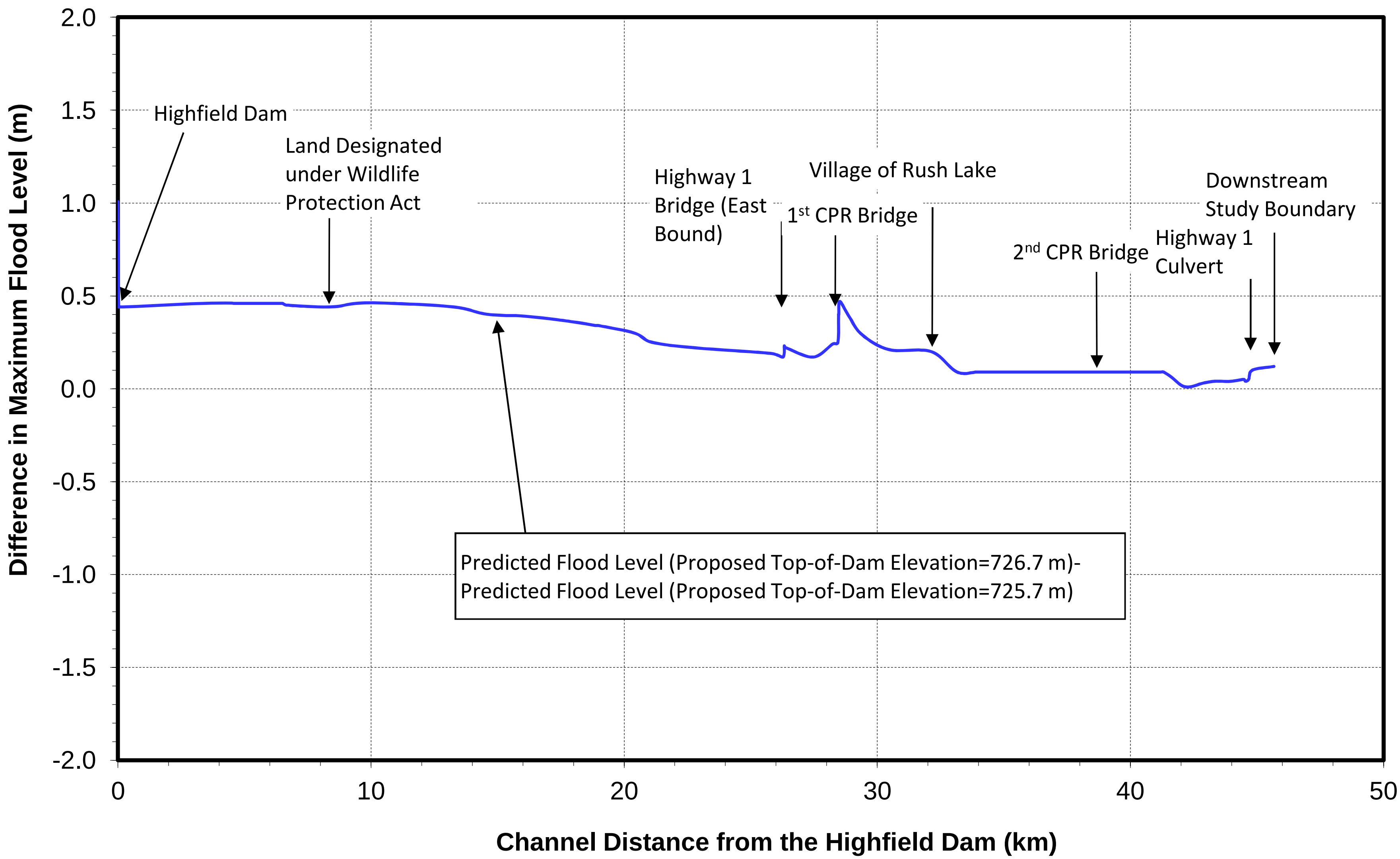


Table B-1: Dam Breach Modelling Results for the Highfield Dam

Cross-Section NO.	Location	Channel Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure					Overtopping Failure under the 1,000-Year Flood Event					Overtopping Failure under the 500-Year Flood Event					1,000-Year Flood No-Dam-Failure		500-Year Flood No-Dam-Failure	
					Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Estimated Depth above Bankfull (m)	Flood Peak Discharge (m³/s)	Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Estimated Depth above Bankfull (m)	Flood Peak Discharge (m³/s)	Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Estimated Depth above Bankfull (m)	Flood Peak Discharge (m³/s)	Maximum Flood Level (m)	Flood Peak Discharge (m³/s)	Maximum Flood Level (m)	Flood Peak Discharge (m³/s)
1	Highfield Dam	0.0	716.0	717.7	0.0	0.0	723.0	5.3	748	0.0	0.5	725.8	8.1	2589	0.0	0.5	725.8	8.1	2556	723.0	361	723.0	274
2	Rush Lake Creek	0.1	715.0	717.5	0.0	1.5	718.7	1.2	736	0.0	0.9	720.3	2.8	2548	0.0	1.4	720.2	2.7	2508	718.2	361	718.1	274
3	Rush Lake Creek	1.9	713.8	716.0	0.4	1.9	718.0	1.9	676	0.2	1.2	719.5	3.4	2301	0.2	1.6	719.4	3.4	2265	717.5	357	717.3	271
4	Rush Lake Creek	3.4	713.0	715.5	0.6	2.5	717.2	1.7	655	0.3	1.6	718.8	3.3	2200	0.4	2.1	718.7	3.2	2164	716.8	355	716.6	270
5	Local Culvert	4.8	712.3	715.5	0.9	3.0	716.8	1.3	630	0.4	1.8	718.4	2.9	2106	0.5	2.3	718.4	2.9	2069	716.4	353	716.2	268
6	Rush Lake Creek	6.1	711.6	714.5	1.2	3.4	716.4	1.9	598	0.6	1.9	718.1	3.6	1988	0.6	2.4	718.1	3.5	1947	716.0	351	715.7	266
7	Local Culvert	6.5	711.4	714.5	1.3	3.5	716.3	1.8	587	0.7	2.0	718.0	3.5	1965	0.7	2.5	717.9	3.4	1923	715.8	350	715.6	266
8	Rush Lake Creek	6.7	711.3	714.0	1.3	3.5	716.2	2.2	581	0.7	2.0	717.9	3.9	1950	0.7	2.5	717.8	3.8	1909	715.8	350	715.5	265
9	Rush Lake Creek	8.4	710.5	714.0	1.6	3.7	715.9	1.9	572	0.8	2.8	717.5	3.5	1916	0.8	3.3	717.5	3.5	1876	715.5	347	715.2	264
10	Rush Lake Creek	9.4	710.3	713.5	1.7	6.0	715.3	1.8	562	0.9	3.6	717.1	3.5	1826	0.9	4.2	717.0	3.5	1790	714.9	344	714.7	261
11	Rush Lake Creek	10.8	710.9	712.5	1.9	6.4	715.1	2.6	482	1.0	3.7	717.0	4.5	1547	1.0	4.3	716.9	4.4	1510	714.8	334	714.5	252
12	Local Road	13.3	710.7	711.6	2.3	6.5	715.0	3.3	433	1.3	3.8	716.8	5.1	1417	1.3	4.3	716.7	5.1	1377	714.7	329	714.3	248
13	Rush Lake Creek	14.6	709.5	711.5	2.7	6.6	714.5	3.0	430	1.4	3.9	716.2	4.7	1402	1.4	4.4	716.2	4.7	1364	714.1	329	713.8	248
14	Rush Lake Creek	16.1	708.5	711.5	3.0	6.9	712.8	1.3	429	1.6	4.3	714.3	2.8	1398	1.6	4.8	714.2	2.7	1360	712.6	329	712.3	248
15	Rush Lake Creek	18.0	708.3	710.5	3.5	7.1	711.7	1.2	428	1.9	5.4	713.2	2.7	1373	1.9	6.0	713.1	2.6	1337	711.7	327	711.4	247
16	Local Culvert	18.9	707.8	710.0	3.7	11.4	711.4	1.4	427	2.0	5.6	712.9	2.9	1334	2.0	6.2	712.9	2.9	1299	711.5	322	711.1	246
17	Rush Lake Creek	20.4	707.0	709.5	4.0	11.9	711.3	1.8	423	2.2	5.8	712.7	3.2	1303	2.2	6.4	712.7	3.2	1269	711.4	312	710.9	242
18	Rush Lake Creek	21.1	706.0	708.5	4.1	12.0	711.3	2.8	408	2.2	5.9	712.6	4.1	1284	2.2	6.5	712.6	4.1	1251	711.4	301	710.9	230
19	Rush Lake Creek	22.8	705.0	708.5	4.5	12.2	711.2	2.7	359	2.4	6.0	712.5	4.0	1257	2.4	6.6	712.5	4.0	1226	711.3	294	710.9	212
20	Highway 1 Bridge (East Bound)	26.3	704.0	707.0	5.2	12.1	711.2	4.2	270	2.8	6.0	712.4	5.4	1215	2.8	6.6	712.4	5.4	1180	711.3	289	710.8	207
21	Highway 1 Bridge (West Bound)	26.3	704.0	708.5	5.2	12.1	710.6	2.1	270	2.8	6.1	712.2	3.7	1215	2.8	6.7	712.2	3.7	1180	710.7	289	710.5	207
22	Rush Lake Creek	27.5	703.2	707.0	5.4	13.8	709.4	2.4	264	3.0	6.1	712.0	5.0	1212	3.0	6.8	712.0	5.0	1177	709.7	287	708.7	206
23	Local Bridge	28.4	702.4	706.2	5.7	13.8	709.4	3.2	260	3.2	6.2	711.9	5.6	1211	3.2	6.8	711.9	5.6	1176	709.7	286	708.7	206
24	1st CPR Bridge	28.5	702.4	705.1	5.7	13.8	709.4	4.3	260	3.2	6.2	710.5	5.4	1211	3.3	6.8	710.5	5.4	1176	709.7	286	708.7	206
25	Local Bridge	29.4	701.3	706.0	5.9	14.2	705.7	-0.3	260	3.3	7.4	707.0	1.0	1144	3.5	8.1	707.0	1.0	1114	705.8	286	705.5	206
26	Main Drainage Canal	30.5	700.0	704.0	6.2	17.1	704.4	0.4	253	4.1	7.7	705.4	1.4	1133	4.3	8.3	705.4	1.4	1103	704.5	280	704.3	202
27	Main Drainage Canal	32.1	698.7	703.0	6.5	18.6	703.3	0.3	238	5.3	9.0	704.1	1.1	1054	5.4	9.5	704.1	1.1	1024	703.4	279	703.3	202
28	Local Bridge	33.1	698.0	703.0	6.8	19.0	703.0	0.0	238	6.0	63.8	703.6	0.6	1042	6.2	9.8	703.5	0.5	1010	703.3	279	702.9	201
29	Main Drainage Canal	34.0	697.9	701.5	6.9	19.6	702.1	0.6	238	6.3	63.9	703.6	2.1	1039	6.5	59.9	703.4	1.9	1006	703.3	279	702.9	201
30	Local Bridge	34.8	697.7	701.5	7.0	20.1	700.9	-0.6	237	6.6	63.9	703.6	2.1	975	6.8	60.1	703.4	1.9	941	703.3	254	702.9	188
31	Main Drainage Canal	35.6	697.5	700.5	7.1	37.3	700.7	0.1	236	6.8	63.7	703.6	3.1	879	7.0	60.6	703.4	2.9	861	703.3	222	702.9	165
32	Local Bridge	36.4	697.3	700.5	7.3	37.5	700.7	0.1	223	7.0	63.9	703.6	3.1	789	7.2	59.7	703.4	2.9	773	703.3	186	702.9	137
33	Main Drainage Canal	37.5	697.1	700.0	7.5	37.5	700.7	0.6	154	7.3	63.6	703.6	3.6	697	7.5	59.8	703.4	3.4	682	703.3	150	702.9	109
34	2nd CPR Bridge	38.5	696.8	699.5	7.7	37.8	700.6	1.1	68	7.7	63.8	703.6	4.1	622	7.8	59.9	703.4	3.9	610	703.3	122	702.9	88
35	Main Drainage Canal	39.5	696.8	699.5	7.8	59.5	700.3	0.8	41	8.0	63.9	703.6	4.1	389	8.2	60.1	703.4	3.9	382	703.3	84	702.9	62
36	Local Culvert	40.2	696.7	700.0	7.9	71.0	700.2	0.2	26	8.3	64.1	703.6	3.6	256	8.5	60.1	703.4	3.4	251	703.3	62	702.9	47
37	Main Drainage Canal	40.8	696.7	700.0	8.1	84.1	700.1	0.1	19	8.5	63.7	703.6	3.6	146	8.7	60.3	703.4	3.4	144	703.3	43	702.9	35
38	Main Drainage Canal	41.2	696.6	700.3	8.2	84.7	700.1	-0.2	14	8.6	63.9	703.6	3.3	77	8.8	60.4	703.4	3.1	65	703.3	41	702.9	27
39	Main Drainage Canal	42.2	696.6	702.0	8.5	85.2	699.9	-2.1	14	8.9	64.1	703.6	1.6	76	9.0	59.9	703.3	1.3	45	703.3	41	702.8	24
40	Main Drainage Canal	43.3	696.3	701.0	8.9	85.6	699.8	-1.2	14	9.3	64.0	703.5	2.5	76	9.3	59.7	703.3	2.3	45	703.3	41	702.8	24
41	Main Drainage Canal	44.0	696.1	700.5	9.0	85.6	699.7	-0.8	14	9.5	63.9	703.5	3.0	76	9.5	59.7	703.3	2.8	45	703.3	41	702.8	24
42	Local Culvert	44.5	696.0	699.5	9.2	85.6	699.7	0.1	14	9.6	63.8	703.5	4.0	76	9.6	60.4	703.3	3.8	45	703.3	41	702.8	24
43	Highway 1 Culvert	44.7	695.9	699.0	9.3	85.5	698.7	-0.4	14	9.7	64.2	701.9	2.9	76	9.7	60.5	701.7	2.6	45	701.6	41	700.1	24
44	Main Drainage Canal	44.8	695.9	699.5	9.3	85.5	698.2	-1.3	14	9.7	64.3	699.6	0.1	76	9.7	60.8	699.2	-0.3	45	699.1	41	698.7	24
45	Main Drainage Canal	45.7	695.7	697.5	9.6	85.7	697.8	0.3	14	10.0	64.8	699.0	1.5	76	10.1	60.7	698.6	1.1	45	698.5	41	698.1	24

Table B-2 Highfield Dam Breach Incremental Impacts on Residences

Residence I.D.	Channel Distance from the Highfield Dam (km)	Ground Elevation (m)	Piping Failure					Overtopping Failure					1,000-Year Flood No-Dam-Failure	
			Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Flood Peak Velocity (m/s)	Maximum Flood Depth (m)	Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Flood Peak Velocity (m/s)	Maximum Flood Depth (m)	Maximum Flood Level (m)	Maximum Flood Depth (m)
1	29.2	705.9	5.9	14.1	705.8	-	No Flooding	3.3	7.1	707.1	0.8	1.2	705.5	No Flooding
2	30.1	711.7*	6.1	16.1	704.8	-	No Flooding	3.8	7.6	706.0	-	No Flooding	704.9	No Flooding
3	30.4	707.7	6.1	16.9	704.5	-	No Flooding	4.0	7.7	705.5	-	No Flooding	704.6	No Flooding
4	31.2	711.7	6.3	17.8	703.9	-	No Flooding	4.6	8.2	704.8	-	No Flooding	704.0	No Flooding
5	31.4	710.3	6.4	18.0	703.8	-	No Flooding	4.8	8.4	704.7	-	No Flooding	703.9	No Flooding
6	31.6	710.1	6.4	18.2	703.7	-	No Flooding	4.9	8.5	704.5	-	No Flooding	703.7	No Flooding
7	31.8	710.5	6.5	18.3	703.5	-	No Flooding	5.0	8.7	704.4	-	No Flooding	703.6	No Flooding
8	32.0	710.2	6.5	18.5	703.4	-	No Flooding	5.2	8.9	704.2	-	No Flooding	703.5	No Flooding
9	32.2	709.1	6.5	18.7	703.3	-	No Flooding	5.3	13.2	704.1	-	No Flooding	703.4	No Flooding
10	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding	703.3	No Flooding
11	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding	703.3	No Flooding
12	33.2	706.8	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding	703.3	No Flooding
13	33.1	708.6	6.7	19.0	703.0	-	No Flooding	6.0	61.3	703.7	-	No Flooding	703.3	No Flooding
14	33.5	706.2*	6.8	19.3	702.6	-	No Flooding	6.1	63.8	703.7	-	No Flooding	703.3	No Flooding
15	33.7	709.3*	6.8	19.4	702.4	-	No Flooding	6.2	63.8	703.7	-	No Flooding	703.3	No Flooding
16	34.0	709.6*	6.9	19.6	702.1	-	No Flooding	6.3	63.9	703.7	-	No Flooding	703.3	No Flooding
17	34.1	709.0*	6.9	19.7	702.0	-	No Flooding	6.3	63.9	703.7	-	No Flooding	703.3	No Flooding
18	44.2	704.4*	9.1	85.6	699.7	-	No Flooding	9.5	63.9	703.5	-	No Flooding	703.3	No Flooding

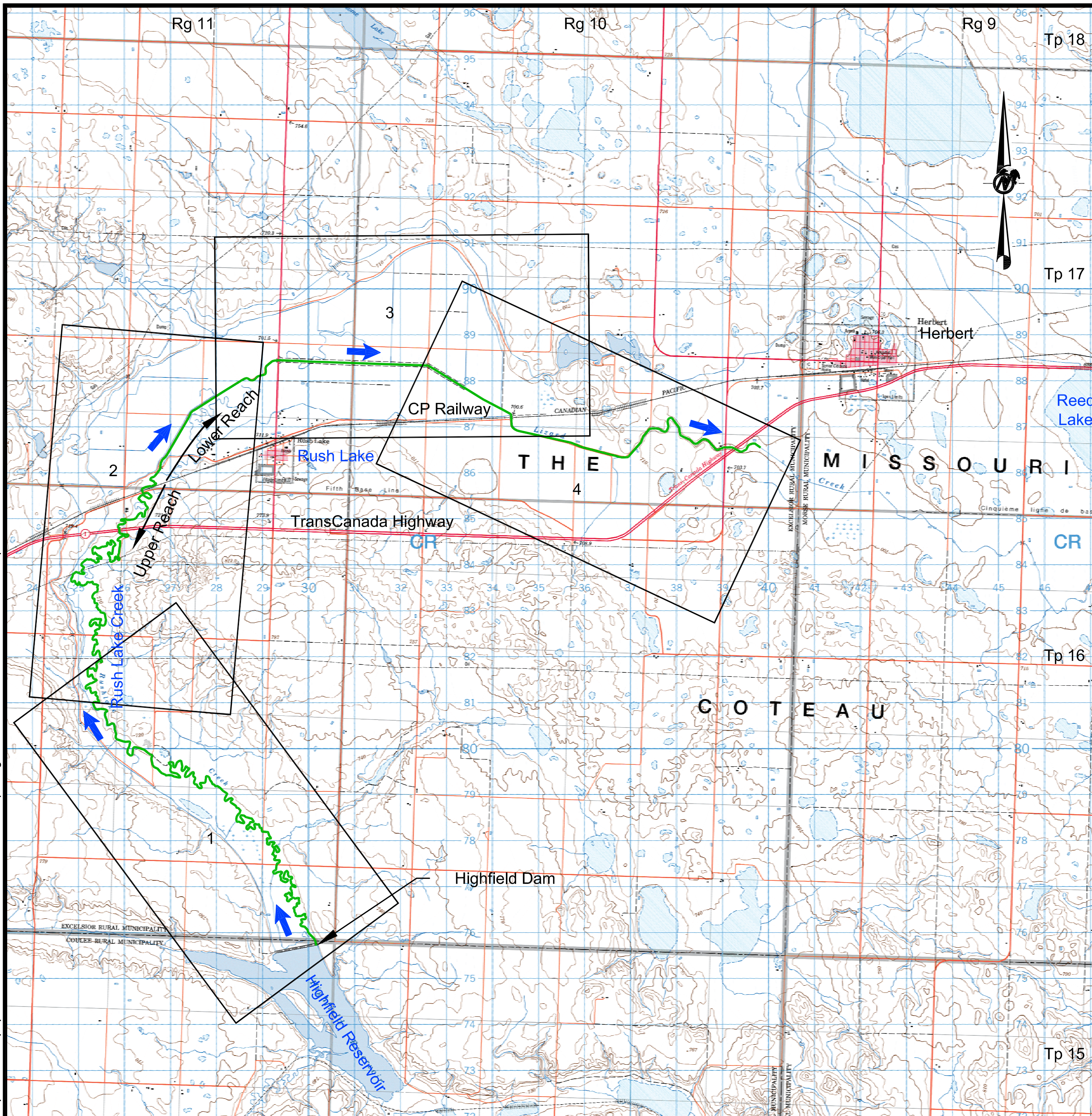
Notes: * denotes ground elevation was obtained based on Lidar data.

Table B-3 Highfield Dam Breach Flood Impacts on Culverts and Bridges

Structures Locations	Channel Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure					Overtopping Failure					1,000-Year Flood No-Dam-Failure	
				Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Maximum Flow Velocity (m/s)	Maximum Flood Depth above Road Surface or Bridge Deck (m)	Flood Arrival Time (hour)	Time to Maximum Flood Level (hour)	Maximum Flood Level (m)	Maximum Flow Velocity (m/s)	Maximum Flood Depth above Road Surface or Bridge Deck (m)	Maximum Flood Level (m)	Maximum Flood Depth above Road Surface or Bridge Deck (m)
Local Culvert	4.8	712.3	715.5	0.9	3.0	716.8	1.0	1.3	0.4	1.8	718.4	1.5	2.9	716.4	0.9
Local Culvert	6.5	711.4	715.0	1.3	3.5	716.3	1.8	1.3	0.7	2.0	718.0	1.9	3.0	715.8	0.8
Local Culvert	18.9	707.8	710.5	3.7	11.4	711.4	0.7	0.9	2.0	5.6	712.9	1.2	2.4	711.5	1.0
Highway 1 Bridge (East Bound)	26.3	704.0	711.3	5.2	12.1	711.2	1.2	No Overtopping	2.8	6.0	712.4	1.4	1.1	711.3	0.0
Highway 1 Bridge (West Bound)	26.3	704.0	709.8	5.2	12.1	710.6	1.8	0.9	2.8	6.1	712.2	1.9	2.4	710.7	0.9
Local Bridge	28.4	702.4	707.3	5.7	13.8	709.4	0.9	2.1	3.2	6.2	711.9	1.0	4.6	709.7	2.4
1st CPR Bridge	28.5	702.4	710.7	5.7	13.8	709.4	5.4	No Overtopping	3.2	6.2	710.5	6.0	No Overtopping	709.7	No Overtopping
Local Bridge	29.4	701.3	706.3	5.9	14.2	705.7	1.8	No Overtopping	3.3	7.4	707.0	3.4	0.7	705.8	No Overtopping
Local Bridge	33.1	698.0	703.2	6.8	19.0	703.0	1.8	No Overtopping	6.0	63.8	703.6	1.7	0.4	703.3	0.1
Local Bridge	34.8	697.7	701.6	7.0	20.1	700.9	0.9	No Overtopping	6.6	63.9	703.6	0.7	2.0	703.3	1.7
Local Bridge	36.4	697.3	700.4	7.3	37.5	700.7	1.0	0.3	7.0	63.9	703.6	0.2	3.3	703.3	3.0
2nd CPR Bridge	38.5	696.8	702.1	7.7	37.8	700.6	1.2	No Overtopping	7.7	63.8	703.6	0.1	1.5	703.3	1.2
Local Culvert	40.2	696.7	700.3	7.9	71.0	700.2	0.6	No Overtopping	8.3	64.1	703.6	0.1	3.3	703.3	3.0
Local Culvert	44.5	696.0	703.0	9.2	85.6	699.7	0.5	No Overtopping	9.6	63.8	703.5	0.3	0.5	703.3	0.3
Highway 1 Culvert	44.7	695.9	702.1	9.3	85.5	698.7	0.5	No Overtopping	9.7	64.2	701.9	0.3	No Overtopping	701.6	No Overtopping

APPENDIX C

Highfield Dam Breach Flood Inundation Maps



LEGEND

- STUDY REACH
- HIGHWAY
- RAILWAY
- WATERCOURSE
- FLOOD FLOW DIRECTION

2.5 0 2.5

SCALE KILOMETRES

REFERENCE

TOPOGRAPHIC MAP 72J/06(© 1999) OBTAINED FROM MapTown. HER MAJESTY THE QUEEN IN RIGHT OF CANADA. DEPARTMENT OF NATURAL RESOURCES. ALL RIGHTS RESERVED. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD83 COORDINATE SYSTEM: UTM ZONE 13

PROJECT

DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE

DAM BREACH FLOOD INUNDATION INDEX MAP

AAFC DRAWING NO. 208553

Golder Associates
Calgary, Alberta

PROJECT	11132600459000		FILE No11132600459000FG001
DESIGN	JC	09/03/11	SCALE AS SHOWN REV. 0
CADD	YW	04/11/11	
CHECK	HZ	04/11/11	FIGURE: 1
REVIEW	AB	04/11/11	

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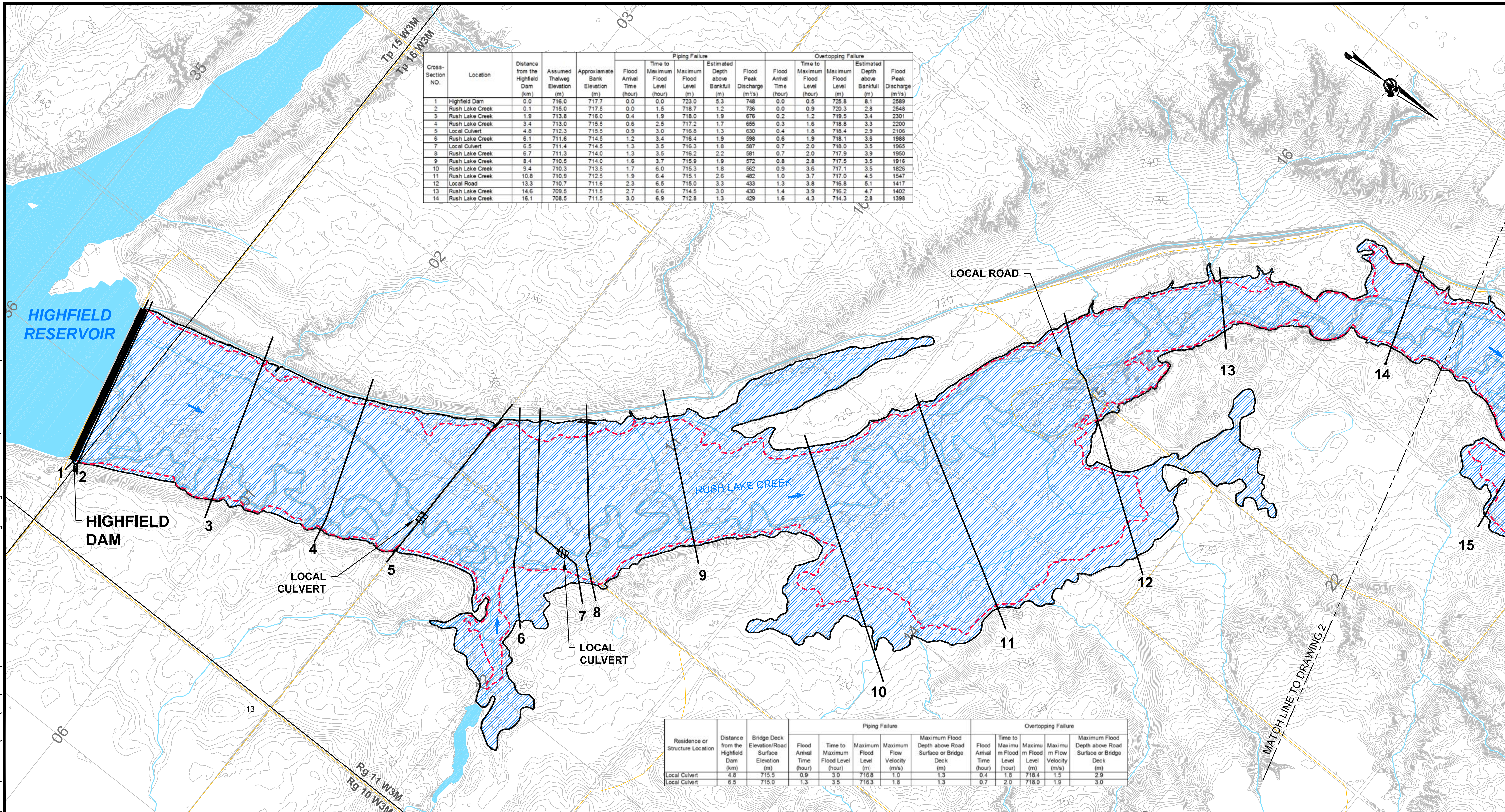


Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m³/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m³/s)
1	Highfield Dam	0.0	716.0	717.7	0.0	0.0	723.0	5.3	748	0.0	0.5	725.8	8.1	2589
2	Rush Lake Creek	0.1	715.0	717.5	0.0	1.5	718.7	1.2	736	0.0	0.9	720.3	2.8	2548
3	Rush Lake Creek	1.9	713.8	716.0	0.4	1.9	718.0	1.9	676	0.2	1.2	719.5	3.4	2301
4	Rush Lake Creek	3.4	713.0	715.5	0.6	2.5	717.2	1.7	655	0.3	1.6	718.8	3.3	2200
5	Local Culvert	4.8	712.3	715.5	0.9	3.0	716.8	1.3	630	0.4	1.8	718.4	2.9	2106
6	Rush Lake Creek	6.1	711.6	714.5	1.2	3.4	716.4	1.9	598	0.6	1.9	718.1	3.6	1988
7	Local Culvert	6.5	711.4	714.5	1.3	3.5	716.3	1.8	587	0.7	2.0	718.0	3.5	1965
8	Rush Lake Creek	6.7	711.3	714.0	1.3	3.5	716.2	2.2	581	0.7	2.0	717.9	3.9	1950
9	Rush Lake Creek	8.4	710.5	714.0	1.6	3.7	715.9	1.9	572	0.8	2.8	717.5	3.5	1916
10	Rush Lake Creek	9.4	710.3	713.5	1.7	6.0	715.3	1.8	562	0.9	3.6	717.1	3.5	1826
11	Rush Lake Creek	10.8	710.9	712.5	1.9	6.4	715.1	2.6	482	1.0	3.7	717.0	4.5	1547
12	Local Road	13.3	710.7	711.6	2.3	6.5	715.0	3.3	433	1.3	3.8	716.8	5.1	1417
13	Rush Lake Creek	14.6	709.5	711.5	2.7	6.6	714.5	3.0	430	1.4	3.9	716.2	4.7	1402
14	Rush Lake Creek	16.1	708.5	711.5	3.0	6.9	712.8	1.3	429	1.6	4.3	714.3	2.8	1398

Residence or Structure Location	Distance from the Highfield Dam (km)	Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
Local Culvert	4.8	715.5	0.9	3.0	716.8	1.0	1.3	0.4	1.8	718.4	1.5	2.9
Local Culvert	6.5	715.0	1.3	3.5	716.3	1.8	1.3	0.7	2.0	718.0	1.9	3.0

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES; TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13; CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

500 0 500
SCALE METRES

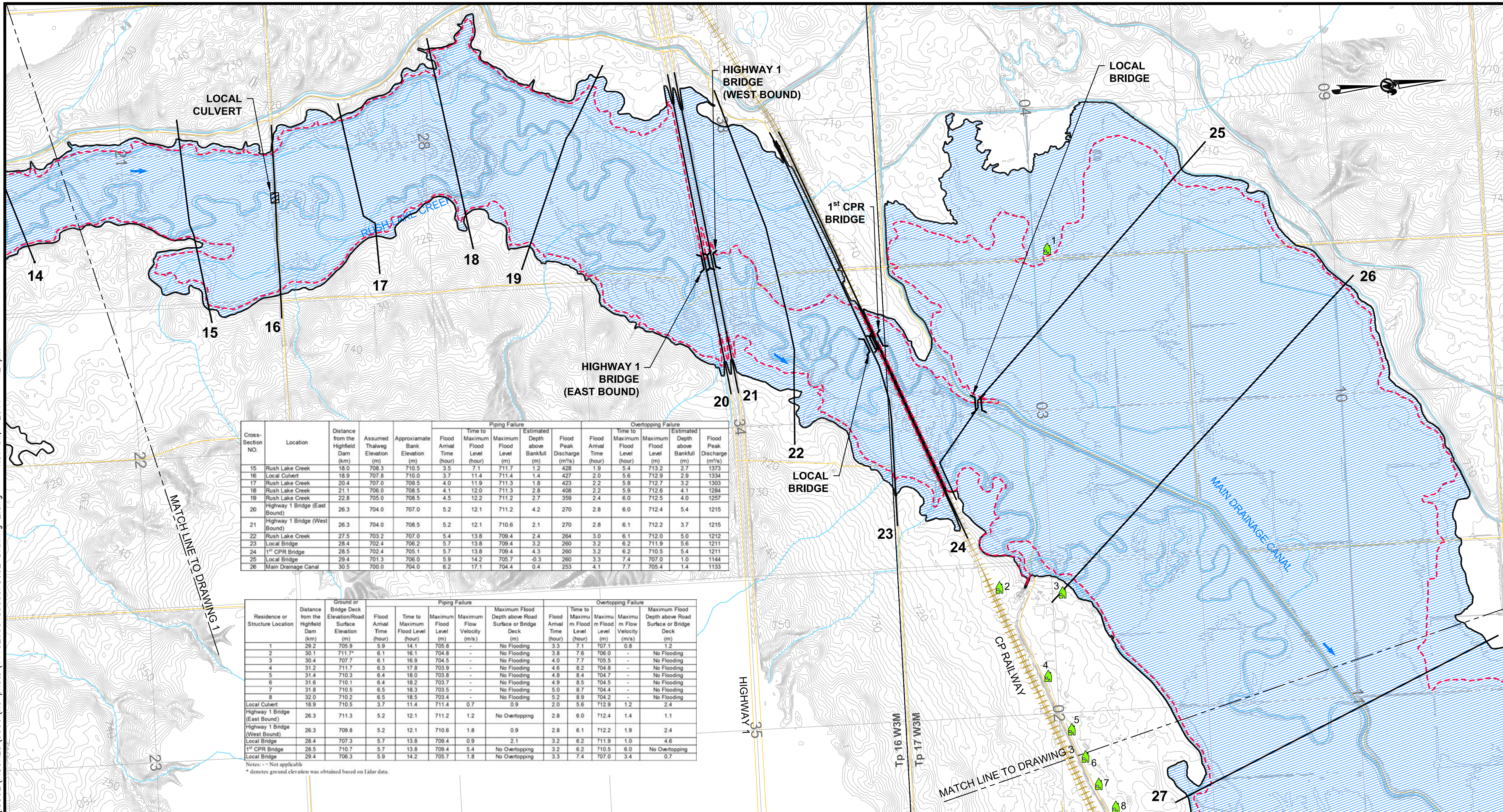
PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MAP SHEET 1 OF 4

AADC DRAWING NO. 208554

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No	111326004590000FG001
DESIGN	JC	13/09/11	SCALE AS SHOWN	REV. 0
CADD	YW	04/11/11		
CHECK	HZ	04/11/11	DRAWING: 1	
REVIEW	AB	04/11/11		



Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m³/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m³/s)
15	Rush Lake Creek	18.0	708.3	710.5	3.5	7.1	711.7	1.2	428	1.9	5.4	713.2	2.7	1373
16	Local Culvert	18.9	707.8	710.0	3.7	11.4	711.4	1.4	427	2.0	5.6	712.9	2.9	1334
17	Rush Lake Creek	20.4	707.0	709.5	4.0	11.9	711.3	1.8	423	2.2	5.8	712.7	3.2	1303
18	Rush Lake Creek	21.1	706.0	708.5	4.1	12.0	711.3	2.8	408	2.2	5.9	712.6	4.1	1284
19	Rush Lake Creek	22.8	705.0	708.5	4.5	12.2	711.2	2.7	359	2.4	6.0	712.5	4.0	1257
20	Highway 1 Bridge (East Bound)	26.3	704.0	707.0	5.2	12.1	711.2	4.2	270	2.8	6.0	712.4	5.4	1215
21	Highway 1 Bridge (West Bound)	26.3	704.0	708.5	5.2	12.1	710.6	2.1	270	2.8	6.1	712.2	3.7	1215
22	Rush Lake Creek	27.5	703.2	707.0	5.4	13.8	709.4	2.4	264	3.0	6.1	712.0	5.0	1212
23	Local Bridge	28.4	702.4	706.2	5.7	13.8	709.4	3.2	260	3.2	6.2	711.9	5.6	1211
24	1st CPR Bridge	28.5	702.4	705.1	5.7	13.8	709.4	4.3	260	3.2	6.2	710.5	5.4	1211
25	Local Bridge	29.4	701.3	706.0	5.9	14.2	705.7	-0.3	260	3.3	7.4	707.0	1.0	1144
26	Main Drainage Canal	30.5	700.0	704.0	6.2	17.1	704.4	0.4	253	4.1	7.7	705.4	1.4	1133

Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
1	29.2	705.9	5.9	14.1	705.8	-	No Flooding	3.3	7.1	707.1	0.8	1.2
2	30.1	711.7*	6.1	16.1	704.8	-	No Flooding	3.8	7.6	706.0	-	No Flooding
3	30.4	707.7	6.1	16.9	704.5	-	No Flooding	4.0	7.7	705.5	-	No Flooding
4	31.2	711.7	6.3	17.8	703.9	-	No Flooding	4.6	8.2	704.8	-	No Flooding
5	31.4	710.3	6.4	18.0	703.8	-	No Flooding	4.8	8.4	704.7	-	No Flooding
6	31.6	710.1	6.4	18.2	703.7	-	No Flooding	4.9	8.5	704.5	-	No Flooding
7	31.8	710.5	6.5	18.3	703.5	-	No Flooding	5.0	8.7	704.4	-	No Flooding
8	32.0	710.2	6.5	18.5	703.4	-	No Flooding	5.2	8.9	704.2	-	No Flooding
Local Culvert	18.9	710.5	3.7	11.4	711.4	0.7	0.9	2.0	5.6	712.9	1.2	2.4
Highway 1 Bridge (East Bound)	26.3	711.3	5.2	12.1	711.2	1.2	No Overtopping	2.8	6.0	712.4	1.4	1.1
Highway 1 Bridge (West Bound)	26.3	709.8	5.2	12.1	710.6	1.8	0.9	2.8	6.1	712.2	1.9	2.4
Local Bridge	28.4	707.3	5.7	13.8	709.4	0.9	2.1	3.2	6.2	711.9	1.0	4.6
1st CPR Bridge	28.5	710.7	5.7	13.8	709.4	5.4	No Overtopping	3.2	6.2	710.5	6.0	No Overtopping
Local Bridge	29.4	706.3	5.9	14.2	705.7	1.8	No Overtopping	3.3	7.4	707.0	3.4	0.7

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES; TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13; CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

SCALE 500 0 500 METRES

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MAP SHEET 2 OF 4

AAFC DRAWING NO. 208555

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No.	11132600459000FG002	
DESIGN	JC	13/09/11	SCALE	AS SHOWN	REV. 0
CADD	YW	04/11/11	DRAWING: 2		
CHECK	HZ	04/11/11			
REVIEW	AB	04/11/11			

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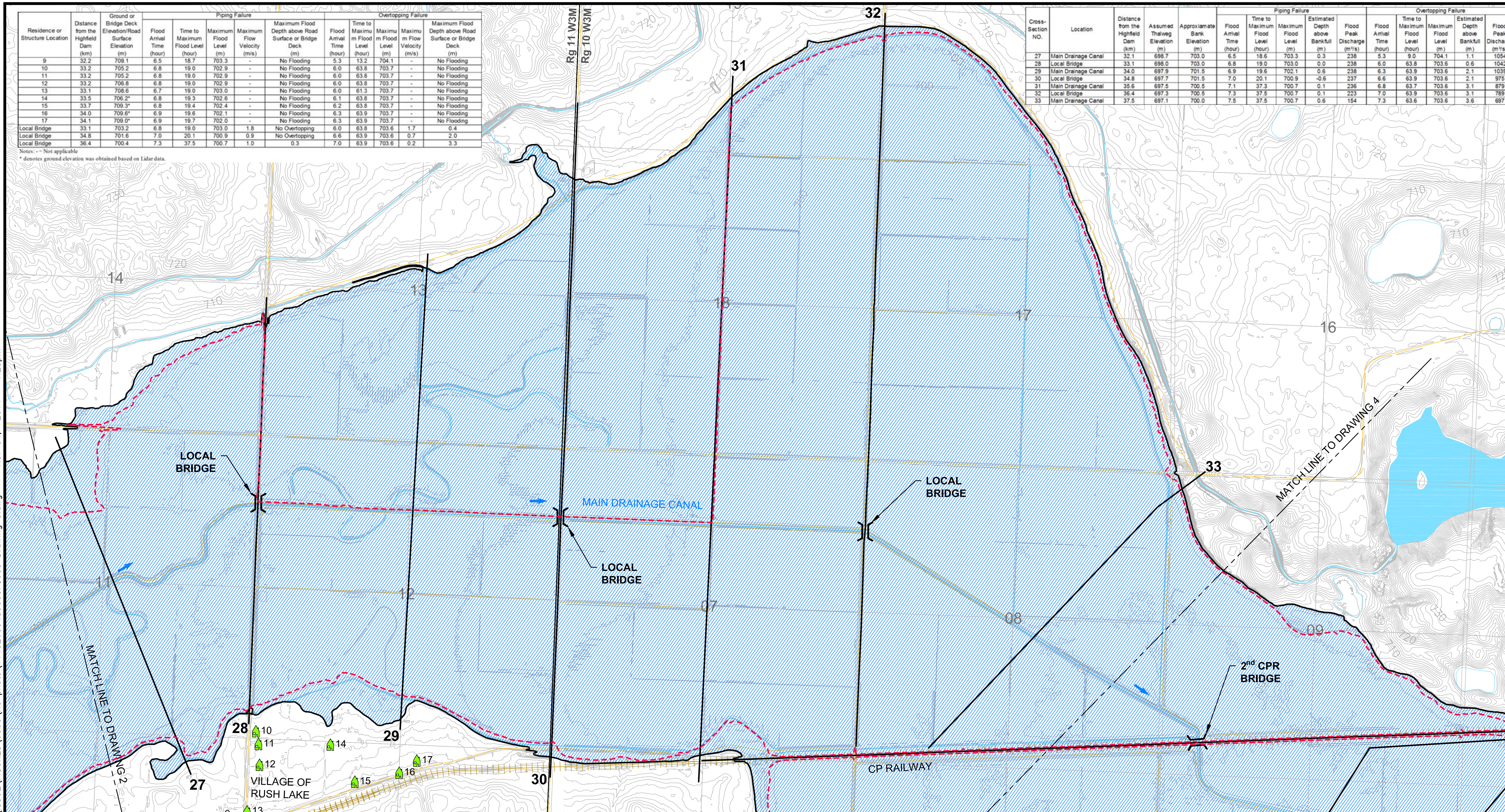
Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
9	32.2	709.1	6.5	18.7	703.3	-	No Flooding	5.3	13.2	704.1	-	No Flooding
10	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
11	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
12	33.2	706.8	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
13	33.1	708.6	6.7	19.0	703.0	-	No Flooding	6.0	61.3	703.7	-	No Flooding
14	33.5	706.2*	6.8	19.3	702.6	-	No Flooding	6.1	63.8	703.7	-	No Flooding
15	33.7	709.3*	6.8	19.4	702.4	-	No Flooding	6.2	63.8	703.7	-	No Flooding
16	34.0	709.6*	6.9	19.6	702.1	-	No Flooding	6.3	63.9	703.7	-	No Flooding
17	34.1	709.0*	6.9	19.7	702.0	-	No Flooding	6.3	63.9	703.7	-	No Flooding
Local Bridge	33.1	703.2	6.8	19.0	703.0	1.8	No Overtopping	6.0	63.8	703.6	1.7	0.4
Local Bridge	34.8	701.6	7.0	20.1	700.9	0.9	No Overtopping	6.6	63.9	703.6	0.7	2.0
Local Bridge	36.4	700.4	7.3	37.5	700.7	1.0	0.3	7.0	63.9	703.6	0.2	3.3

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m^3/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m^3/s)
27	Main Drainage Canal	32.1	698.7	703.0	6.5	18.6	703.3	0.3	238	5.3	9.0	704.1	1.1	1054
28	Local Bridge	33.1	698.0	703.0	6.8	19.0	703.0	0.0	238	6.0	63.8	703.6	0.6	1042
29	Main Drainage Canal	34.0	697.9	701.5	6.9	19.6	702.1	0.6	238	6.3	63.9	703.6	2.1	1039
30	Local Bridge	34.8	697.7	701.5	7.0	20.1	700.9	-0.6	237	6.6	63.9	703.6	2.1	975
31	Main Drainage Canal	35.6	697.5	700.5	7.1	37.3	700.7	0.1	236	6.8	63.7	703.6	3.1	879
32	Local Bridge	36.4	697.3	700.5	7.3	37.5	700.7	0.1	223	7.0	63.9	703.6	3.1	789
33	Main Drainage Canal	37.5	697.1	700.0	7.5	37.5	700.7	0.6	154	7.3	63.6	703.6	3.6	697



LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES: TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13; CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

500 0 500
SCALE METRES

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MAP SHEET 3 OF 4

AAFC DRAWING NO. 208556

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No. 1132600459000FG003	
DESIGN	JC	13/09/11	SCALE AS SHOWN	REV. 0
CADD	YW	04/11/11	**DRAWING: 3**	
CHECK	HZ	04/11/11		
REVIEW	AB	04/11/11		

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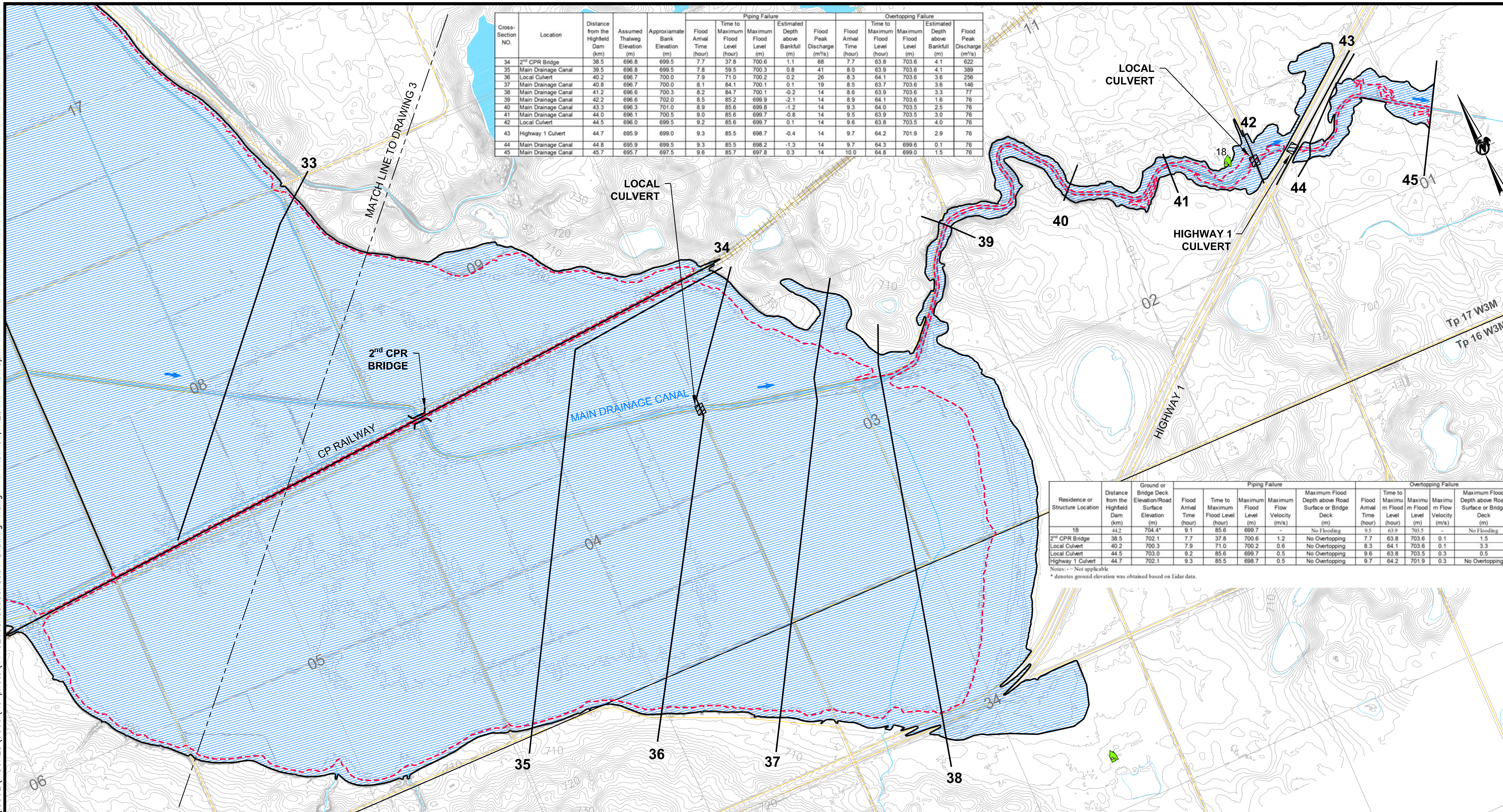

Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m^3/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m^3/s)
34	2nd CPR Bridge	38.5	696.8	699.5	7.7	37.8	700.6	1.1	68	7.7	63.8	703.6	4.1	622
35	Main Drainage Canal	39.5	696.8	699.5	7.8	59.5	700.3	0.8	41	8.0	63.9	703.6	4.1	389
36	Local Culvert	40.2	696.7	700.0	7.9	71.0	700.2	0.2	26	8.3	64.1	703.6	3.6	256
37	Main Drainage Canal	40.8	696.7	700.0	8.1	84.1	700.1	0.1	19	8.5	63.7	703.6	3.6	146
38	Main Drainage Canal	41.2	696.6	700.3	8.2	84.7	700.1	-0.2	14	8.6	63.9	703.6	3.3	77
39	Main Drainage Canal	42.2	696.6	702.0	8.5	85.2	699.9	-2.1	14	8.9	64.1	703.6	1.6	76
40	Main Drainage Canal	43.3	696.3	701.0	8.9	85.6	699.8	-1.2	14	9.3	64.0	703.5	2.5	76
41	Main Drainage Canal	44.0	696.1	700.5	9.0	85.6	699.7	-0.8	14	9.5	63.9	703.5	3.0	76
42	Local Culvert	44.5	696.0	699.5	9.2	85.6	699.7	0.1	14	9.6	63.8	703.5	4.0	76
43	Highway 1 Culvert	44.7	695.9	699.0	9.3	85.5	698.7	-0.4	14	9.7	64.2	701.9	2.9	76
44	Main Drainage Canal	44.8	695.9	699.5	9.3	85.5	698.2	-1.3	14	9.7	64.3	699.6	0.1	76
45	Main Drainage Canal	45.7	695.7	697.5	9.6	85.7	697.8	0.3	14	10.0	64.8	699.0	1.5	76

Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
18	44.2	704.4*	9.1	85.6	699.7	-	No Flooding	9.5	63.9	703.5	-	No Flooding
2nd CPR Bridge	38.5	702.1	7.7	37.8	700.6	1.2	No Overtopping	7.7	63.8	703.6	0.1	1.5
Local Culvert	40.2	700.3	7.9	71.0	700.2	0.6	No Overtopping	8.3	64.1	703.6	0.1	3.3
Local Culvert	44.5	703.0	9.2	85.6	699.7	0.5	No Overtopping	9.6	63.8	703.5	0.3	0.5
Highway 1 Culvert	44.7	702.1	9.3	85.5	698.7	0.5	No Overtopping	9.7	64.2	701.9	0.3	No Overtopping

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES: TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13;
CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

500 0 500
SCALE METRES

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MAP SHEET 4 OF 4

AAFC DRAWING NO. 208557

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No. 1132600459000FG004	
DESIGN	JC	13/09/11	SCALE AS SHOWN	REV. 0
CADD	YW	04/11/11	DRAWING: 4	
CHECK	HZ	04/11/11		
REVIEW	AB	04/11/11		

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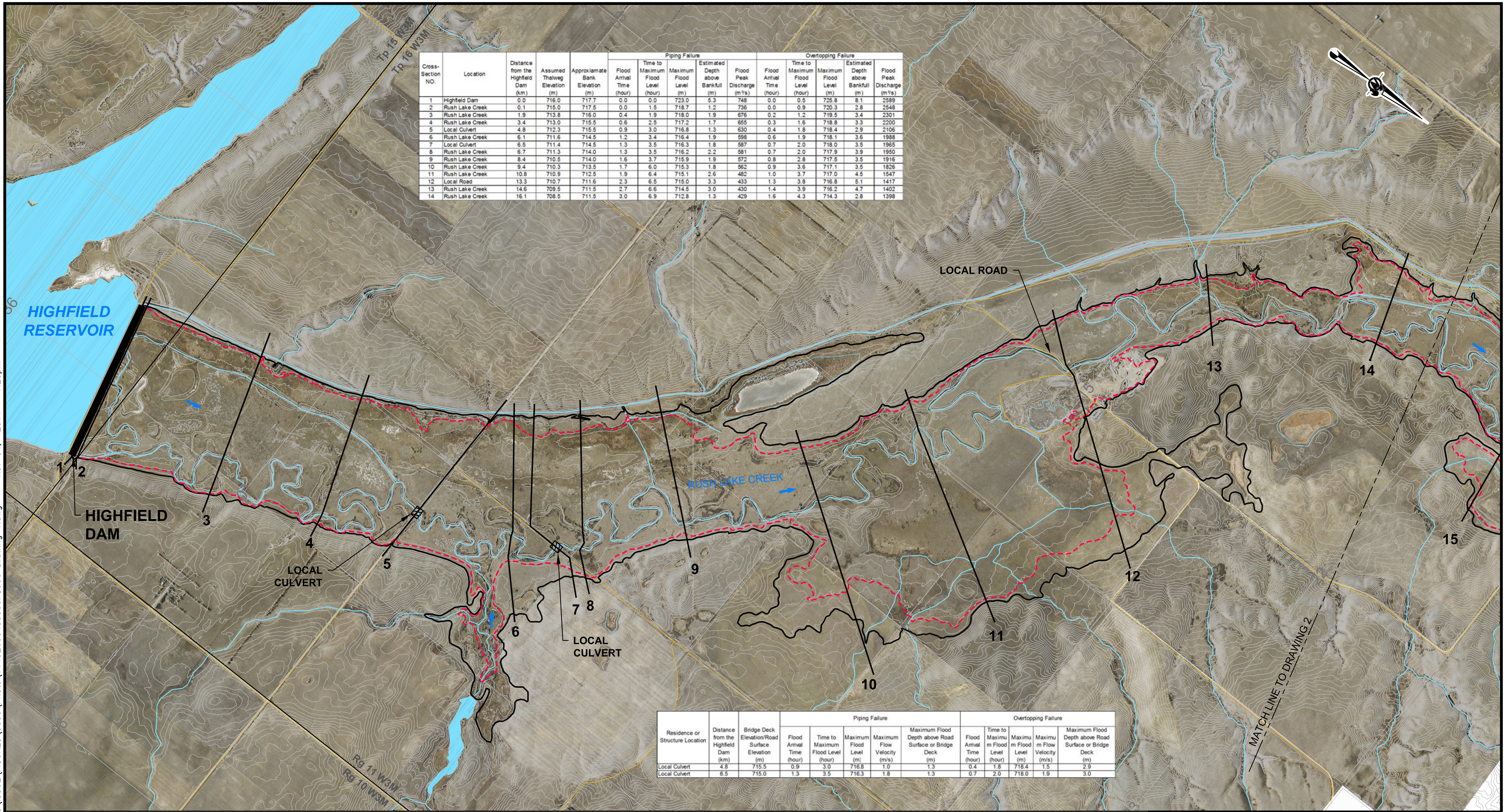
APPENDIX D

Dam Breach Flood Inundation Mosaic Maps



Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m³/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m³/s)
1	Highfield Dam	0.0	716.0	717.7	0.0	0.0	723.0	5.3	748	0.0	0.5	725.8	8.1	2589
2	Rush Lake Creek	0.1	715.0	717.5	0.0	1.5	718.7	1.2	736	0.0	0.9	720.3	2.8	2548
3	Rush Lake Creek	1.9	713.8	716.0	0.4	1.9	718.0	1.9	676	0.2	1.2	719.5	3.4	2301
4	Rush Lake Creek	3.4	713.0	715.5	0.6	2.5	717.2	1.7	655	0.3	1.6	718.8	3.3	2200
5	Local Culvert	4.8	712.3	715.5	0.9	3.0	716.8	1.3	630	0.4	1.8	718.4	2.9	2106
6	Rush Lake Creek	6.1	711.6	714.5	1.2	3.4	716.4	1.9	598	0.6	1.9	718.1	3.6	1988
7	Local Culvert	6.5	711.4	714.5	1.3	3.5	716.3	1.8	587	0.7	2.0	718.0	3.5	1965
8	Rush Lake Creek	6.7	711.3	714.0	1.3	3.5	716.2	2.2	581	0.7	2.0	717.9	3.9	1950
9	Rush Lake Creek	8.4	710.5	714.0	1.6	3.7	715.9	1.9	572	0.8	2.8	717.5	3.5	1916
10	Rush Lake Creek	9.4	710.3	713.5	1.7	6.0	715.3	1.8	562	0.9	3.6	717.1	3.5	1826
11	Rush Lake Creek	10.8	710.9	712.5	1.9	6.4	715.1	2.6	482	1.0	3.7	717.0	4.5	1547
12	Local Road	13.3	710.7	711.6	2.3	6.5	715.0	3.3	433	1.3	3.8	716.8	5.1	1417
13	Rush Lake Creek	14.6	709.5	711.5	2.7	6.6	714.5	3.0	430	1.4	3.9	716.2	4.7	1402
14	Rush Lake Creek	16.1	708.5	711.5	3.0	6.9	712.8	1.3	429	1.6	4.3	714.3	2.8	1398

Residence or Structure Location	Distance from the Highfield Dam (km)	Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
Local Culvert	4.8	715.5	0.9	3.0	716.8	1.0	1.3	0.4	1.8	718.4	1.5	2.9
Local Culvert	6.5	715.0	1.3	3.5	716.3	1.8	1.3	0.7	2.0	718.0	1.9	3.0

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES; TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13;
CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

500 0 500
SCALE METRES

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MOSAIC MAP SHEET 1 OF 4

AAFC DRAWING NO. 208558

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No.	1132600459000FG006	
DESIGN	JC	13/09/11	SCALE	AS SHOWN	REV. 0
CADD	YW	04/11/11			
CHECK	HZ	04/11/11	DRAWING: 1		
REVIEW	AB	04/11/11			

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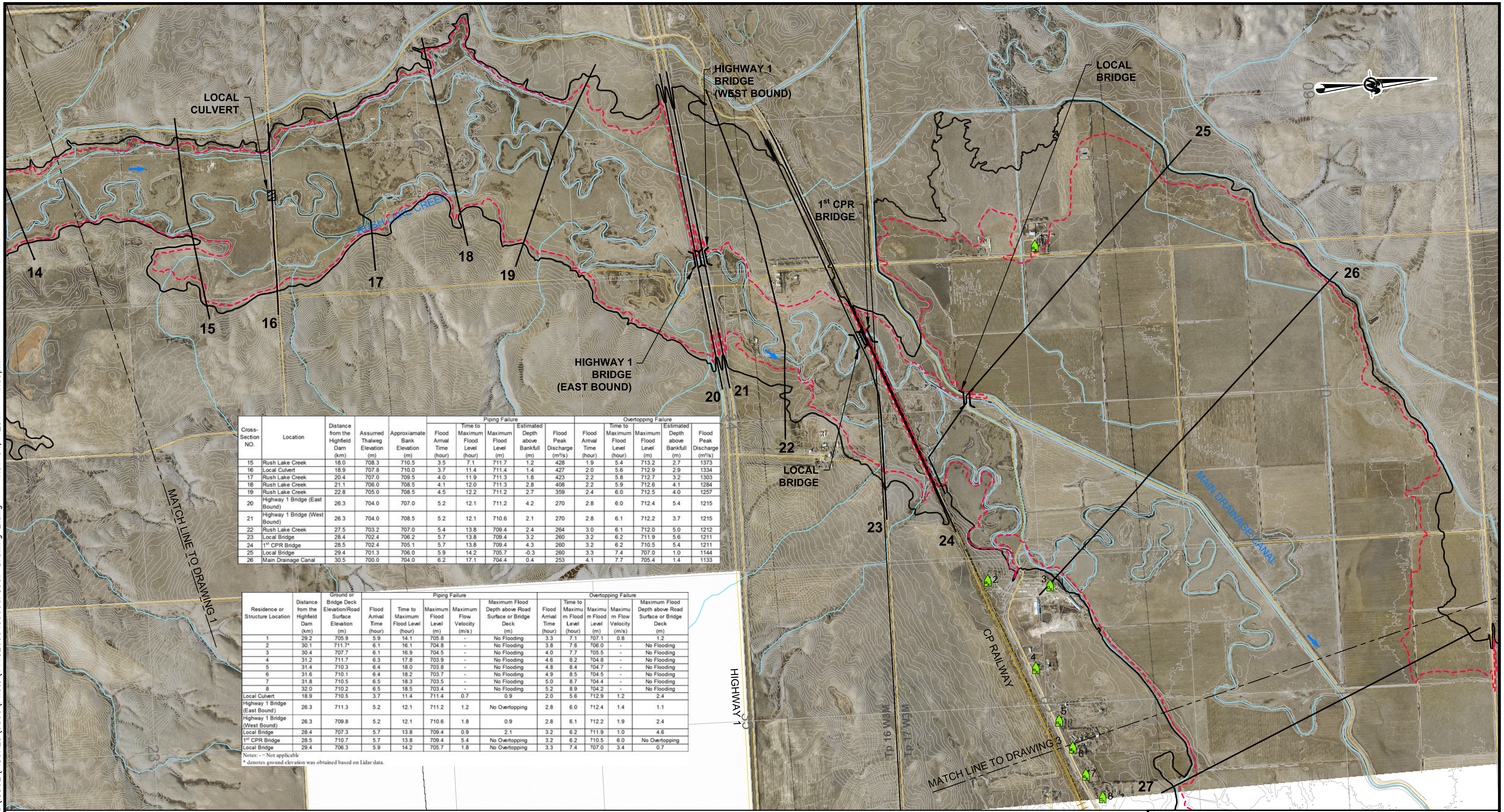

Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m³/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m³/s)
15	Rush Lake Creek	18.0	708.3	710.5	3.5	7.1	711.7	1.2	428	1.9	5.4	713.2	2.7	1373
16	Local Culvert	18.9	707.8	710.0	3.7	11.4	711.4	1.4	427	2.0	5.6	712.9	2.9	1334
17	Rush Lake Creek	20.4	707.0	709.5	4.0	11.9	711.3	1.8	423	2.2	5.8	712.7	3.2	1303
18	Rush Lake Creek	21.1	706.0	708.5	4.1	12.0	711.3	2.8	408	2.2	5.9	712.6	4.1	1284
19	Rush Lake Creek	22.8	705.0	708.5	4.5	12.2	711.2	2.7	359	2.4	6.0	712.5	4.0	1257
20	Highway 1 Bridge (East Bound)	26.3	704.0	707.0	5.2	12.1	711.2	4.2	270	2.8	6.0	712.4	5.4	1215
21	Highway 1 Bridge (West Bound)	26.3	704.0	708.5	5.2	12.1	710.6	2.1	270	2.8	6.1	712.2	3.7	1215
22	Rush Lake Creek	27.5	703.2	707.0	5.4	13.8	709.4	2.4	264	3.0	6.1	712.0	5.0	1212
23	Local Bridge	28.4	702.4	706.2	5.7	13.8	709.4	3.2	260	3.2	6.2	711.9	5.6	1211
24	1st CPR Bridge	28.5	702.4	705.1	5.7	13.8	709.4	4.3	260	3.2	6.2	710.5	5.4	1211
25	Local Bridge	29.4	701.3	706.0	5.9	14.2	705.7	-0.3	260	3.3	7.4	707.0	1.0	1144
26	Main Drainage Canal	30.5	700.0	704.0	6.2	17.1	704.4	0.4	253	4.1	7.7	705.4	1.4	1133

Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
1	29.2	705.9	5.9	14.1	705.8	-	No Flooding	3.3	7.1	707.1	0.8	1.2
2	30.1	711.7*	6.1	16.1	704.8	-	No Flooding	3.8	7.6	706.0	-	No Flooding
3	30.4	707.7	6.1	16.9	704.5	-	No Flooding	4.0	7.7	705.5	-	No Flooding
4	31.2	711.7	6.3	17.8	703.9	-	No Flooding	4.6	8.2	704.8	-	No Flooding
5	31.4	710.3	6.4	18.0	703.8	-	No Flooding	4.8	8.4	704.7	-	No Flooding
6	31.6	710.1	6.4	18.2	703.7	-	No Flooding	4.9	8.5	704.5	-	No Flooding
7	31.8	710.5	6.5	18.3	703.5	-	No Flooding	5.0	8.7	704.4	-	No Flooding
8	32.0	710.2	6.5	18.5	703.4	-	No Flooding	5.2	8.9	704.2	-	No Flooding
Local Culvert	18.9	710.5	3.7	11.4	711.4	0.7	0.9	2.0	5.6	712.9	1.2	2.4
Highway 1 Bridge (East Bound)	26.3	711.3	5.2	12.1	711.2	1.2	No Overtopping	2.8	6.0	712.4	1.4	1.1
Highway 1 Bridge (West Bound)	26.3	709.8	5.2	12.1	710.6	1.8	0.9	2.8	6.1	712.2	1.9	2.4
Local Bridge	28.4	707.3	5.7	13.8	709.4	0.9	2.1	3.2	6.2	711.9	1.0	4.6
1st CPR Bridge	28.5	710.7	5.7	13.8	709.4	5.4	No Overtopping	3.2	6.2	710.5	6.0	No Overtopping
Local Bridge	29.4	706.3	5.9	14.2	705.7	1.8	No Overtopping	3.3	7.4	707.0	3.4	0.7

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

TOPOGRAPHIC MAP 072J/06 OBTAINED FROM Canmatrix.© 2007 HER MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES: TRANSVERSE MERCATOR DATUM: NAD83; COORDINATE SYSTEM: UTM ZONE 13;
CONTOUR DATA PROVIDE BY McELHANNEY CONSULTING SERVICES LTD.

500 0 500
SCALE METRES

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MOSAIC MAP SHEET 2 OF 4

AAFC DRAWING NO. 208559

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No.	1132600459000FG007
DESIGN	JC	13/09/11	SCALE AS SHOWN	REV. 0
CADD	YW	04/11/11		
CHECK	HZ	04/11/11	DRAWING: 2	
REVIEW	AB	04/11/11		

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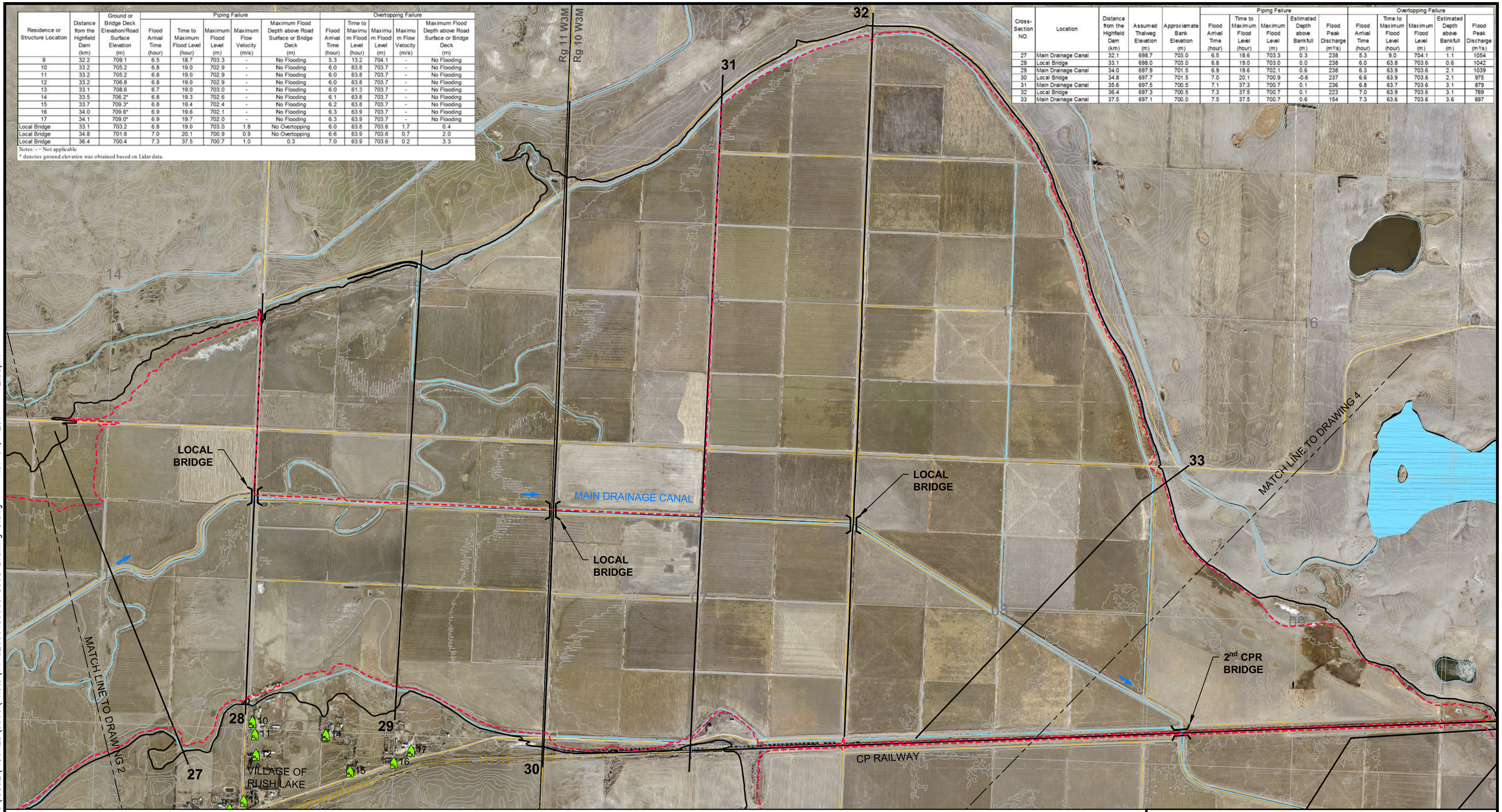
Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
9	32.2	709.1	6.5	18.7	703.3	-	No Flooding	5.3	13.2	704.1	-	No Flooding
10	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
11	33.2	705.2	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
12	33.2	706.8	6.8	19.0	702.9	-	No Flooding	6.0	63.8	703.7	-	No Flooding
13	33.1	708.6	6.7	19.0	703.0	-	No Flooding	6.0	61.3	703.7	-	No Flooding
14	33.5	706.2*	6.8	19.3	702.6	-	No Flooding	6.1	63.8	703.7	-	No Flooding
15	33.7	709.3*	6.8	19.4	702.4	-	No Flooding	6.2	63.8	703.7	-	No Flooding
16	34.0	709.6*	6.9	19.6	702.1	-	No Flooding	6.3	63.9	703.7	-	No Flooding
17	34.1	709.0*	6.9	19.7	702.0	-	No Flooding	6.3	63.9	703.7	-	No Flooding
Local Bridge	33.1	703.2	6.8	19.0	703.0	1.8	No Overtopping	6.0	63.8	703.6	1.7	0.4
Local Bridge	34.8	701.6	7.0	20.1	700.9	0.9	No Overtopping	6.6	63.9	703.6	0.7	2.0
Local Bridge	36.4	700.4	7.3	37.5	700.7	1.0	0.3	7.0	63.9	703.6	0.2	3.3

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m³/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m³/s)
27	Main Drainage Canal	32.1	698.7	703.0	6.5	18.6	703.3	0.3	238	5.3	9.0	704.1	1.1	1054
28	Local Bridge	33.1	698.0	703.0	6.8	19.0	703.0	0.0	238	6.0	63.8	703.6	0.6	1042
29	Main Drainage Canal	34.0	697.9	701.5	6.9	19.6	702.1	0.6	238	6.3	63.9	703.6	2.1	1039
30	Local Bridge	34.8	697.7	701.5	7.0	20.1	700.9	-0.6	237	6.6	63.9	703.6	2.1	975
31	Main Drainage Canal	35.6	697.5	700.5	7.1	37.3	700.7	0.1	236	6.8	63.7	703.6	3.1	879
32	Local Bridge	36.4	697.3	700.5	7.3	37.5	700.7	0.1	223	7.0	63.9	703.6	3.1	789
33	Main Drainage Canal	37.5	697.1	700.0	7.5	37.5	700.7	0.6	154	7.3	63.6	703.6	3.6	697



LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
- BRIDGE
- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

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SCALE METRES

AAFC DRAWING NO. 208560

PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MOSAIC MAP SHEET 3 OF 4

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No.	1132600459000FG008
DESIGN	JC	13/09/11	SCALE AS SHOWN	REV. 0
CADD	YW	04/11/11		
CHECK	HZ	04/11/11		
REVIEW	AB	04/11/11		

DRAWING: 3

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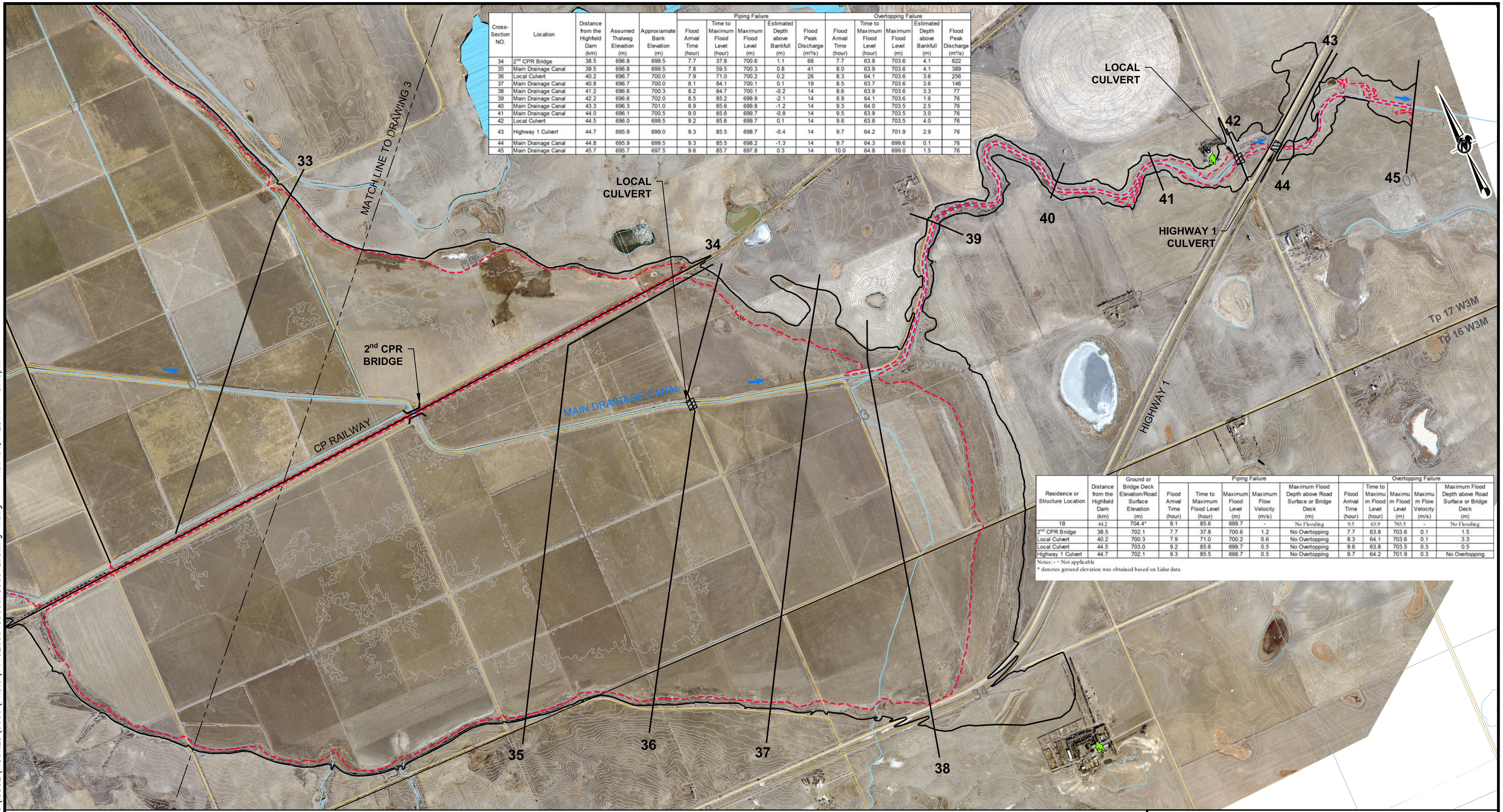

Cross-Section NO.	Location	Distance from the Highfield Dam (km)	Assumed Thalweg Elevation (m)	Approxiamate Bank Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Estimated Depth above Bankfull (m)	Piping Failure: Flood Peak Discharge (m^3/s)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Estimated Depth above Bankfull (m)	Overtopping Failure: Flood Peak Discharge (m^3/s)
34	2nd CPR Bridge	38.5	696.8	699.5	7.7	37.8	700.6	1.1	68	7.7	63.8	703.6	4.1	622
35	Main Drainage Canal	39.5	696.8	699.5	7.8	59.5	700.3	0.8	41	8.0	63.9	703.6	4.1	389
36	Local Culvert	40.2	696.7	700.0	7.9	71.0	700.2	0.2	26	8.3	64.1	703.6	3.6	256
37	Main Drainage Canal	40.8	696.7	700.0	8.1	84.1	700.1	0.1	19	8.5	63.7	703.6	3.6	146
38	Main Drainage Canal	41.2	696.6	700.3	8.2	84.7	700.1	-0.2	14	8.6	63.9	703.6	3.3	77
39	Main Drainage Canal	42.2	696.6	702.0	8.5	85.2	699.9	-2.1	14	8.9	64.1	703.6	1.6	76
40	Main Drainage Canal	43.3	696.3	701.0	8.9	85.6	699.8	-1.2	14	9.3	64.0	703.5	2.5	76
41	Main Drainage Canal	44.0	696.1	700.5	9.0	85.6	699.7	-0.8	14	9.5	63.9	703.5	3.0	76
42	Local Culvert	44.5	696.0	699.5	9.2	85.6	699.7	0.1	14	9.6	63.8	703.5	4.0	76
43	Highway 1 Culvert	44.7	695.9	699.0	9.3	85.5	698.7	-0.4	14	9.7	64.2	701.9	2.9	76
44	Main Drainage Canal	44.8	695.9	699.5	9.3	85.5	698.2	-1.3	14	9.7	64.3	699.6	0.1	76
45	Main Drainage Canal	45.7	695.7	697.5	9.6	85.7	697.8	0.3	14	10.0	64.8	699.0	1.5	76

Residence or Structure Location	Distance from the Highfield Dam (km)	Ground or Bridge Deck Elevation/Road Surface Elevation (m)	Piping Failure: Flood Arrival Time (hour)	Piping Failure: Time to Maximum Flood Level (hour)	Piping Failure: Maximum Flood Level (m)	Piping Failure: Maximum Flow Velocity (m/s)	Piping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)	Overtopping Failure: Flood Arrival Time (hour)	Overtopping Failure: Time to Maximum Flood Level (hour)	Overtopping Failure: Maximum Flood Level (m)	Overtopping Failure: Maximum Flow Velocity (m/s)	Overtopping Failure: Maximum Flood Depth above Road Surface or Bridge Deck (m)
18	44.2	704.4*	9.1	85.6	699.7	-	No Flooding	9.5	63.9	703.5	-	No Flooding
2nd CPR Bridge	38.5	702.1	7.7	37.8	700.6	1.2	No Overtopping	7.7	63.8	703.6	0.1	1.5
Local Culvert	40.2	700.3	7.9	71.0	700.2	0.6	No Overtopping	8.3	64.1	703.6	0.1	3.3
Local Culvert	44.5	703.0	9.2	85.6	699.7	0.5	No Overtopping	9.6	63.8	703.5	0.3	0.5
Highway 1 Culvert	44.7	702.1	9.3	85.5	698.7	0.5	No Overtopping	9.7	64.2	701.9	0.3	No Overtopping

Notes: - = Not applicable
* denotes ground elevation was obtained based on Lidar data.

LEGEND

- HOUSE SURVEY LOCATION
- 6 CROSS-SECTION LOCATION AND NUMBER
- FLOOD EXTENT FOR OVERTOPPING FAILURE
- FLOOD EXTENT FOR PIPING FAILURE
- CULVERT
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- FLOOD FLOW DIRECTION
- CONTOUR - 1 m
- ROAD OR STREET
- RAILWAY
- WATERCOURSE
- LAKE OR RESERVOIR

REFERENCE

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SCALE METRES

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PROJECT
DAM CLASSIFICATION AND HYDRO TECHNICAL STUDY FOR THE HIGHFIELD DAM

TITLE
HIGHFIELD DAM BREACH FLOOD INUNDATION MOSAIC MAP SHEET 4 OF 4

Golder Associates
Calgary, Alberta

PROJECT	11.1326.0045.9000		FILE No. 1132600459000FG009	
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CADD	YW	04/11/11		
CHECK	HZ	04/11/11	DRAWING: 4	
REVIEW	AB	04/11/11		

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APPENDIX E

Incremental Consequence Assessment Methodology

INCREMENTAL CONSEQUENCE ASSESSMENT METHODOLOGY

E.1 INTRODUCTION

According to the Canadian Dam Association's 2007 Dam Safety Guidelines, the standard of care and due diligence expected of a dam owner relates to the incremental losses due to a dam failure, that is, losses above and beyond those that would have occurred due to a natural event if the dam had not failed. The incremental consequences of failure are defined as the total damage from an event with dam failure minus the damage that would have resulted from the same event had the dam not failed. For a fair-weather failure scenario, the incremental consequences of a dam failure are the same as the total consequences.

According to the CDA 2007 Dam Safety Guidelines, the incremental consequence classification of a dam takes into consideration consequences that fall into three broad categories: (1) potential loss of life, (2) infrastructure and economic losses, and (3) losses of environmental and cultural values. The purpose of this Appendix is to describe empirical approaches used to estimate potential loss of life and infrastructure and economics losses (third party damages and loss of water impacts). The Appendix also outlines an approach to estimate the cost of repairing a dam breach following a fair-weather failure.

E.2 LOSS OF LIFE

The CDA 2007 Dam Safety Guidelines state that, in addition to economic and environmental losses, the consequences of a dam failure should be evaluated in terms of life safety. The population at risk (PAR) in an inundated area provides an indication of the number of people exposed to the hazard. The CDA 2007 Dam Safety Guidelines provide a qualitative definition of PAR as "the number of people who would be exposed to floodwaters and would experience consequences that could range from inconvenience and economic losses to loss of life".

The potential loss of life (LOL) would be a proportion of the PAR, depending on factors such as warning time, location, elevation, flood depth, flow velocity, season of the year, and time of day or night. The PAR would include people who are permanently in the potential flood path downstream of a dam and those who are temporarily in the flood path, such as recreational users, traffic on roads or bridges, or seasonal cottage owners.

The CDA 2007 Dam Safety Guidelines state that the potential for loss of life would depend on many highly uncertain and variable factors, including depth of flow, flow velocity, time of day, advance warning, etc. The CDA 2007 Dam Safety Guidelines also recognize that consistent estimates of expected loss of life are very difficult to develop, with no simple, reliable, or universally applicable methodology available.

E.2.1 Empirical Methods for Estimating Loss of Life

There are a number of empirical approaches for estimating the potential loss of life (LOL) from a hypothetical dam failure. McClelland and Bowles (2002) provide a detailed description and discussion of methods developed by various agencies and researchers for estimating LOL. Empirical approaches for estimating LOL can result in a range of estimates depending on whether there is adequate warning time for downstream populations to evacuate, whether there is an existing evacuation plan, the level of emergency preparedness, the topography of the downstream areas (narrow and fast flowing waters or flat and slow flowing waters but of greater depth), etc. Given the uncertainties in LOL estimates, a range of LOL estimates, with a best estimate of LOL, is generally specified during an incremental consequence assessment. Three approaches to estimate potential loss of life from McClelland and Bowles (2002) are described in the following sections. The approaches are identified by the author and year the approach was published. The approach suggested by Graham (1999) is the one discussed in detail in this Appendix.

E.2.1.1 Graham 1999

The general approach suggested by Graham (1999) is to divide the PAR into subpar, classify each subPAR according to a trichotomous division of flood severity (Low, Medium, High), a trichotomous division of (official) warning time (No warning, Some Warning, Adequate warning), and a dichotomous division of flood severity understanding (Vague, Precise).

Flood severity (FS) is classified as low when homes are flooded but not destroyed; medium when some homes or businesses are destroyed but others remain un-submerged; and high when the flood plain is swept clean. To distinguish between low and medium severity, Graham (1999) suggested two criteria, one based on depth and the other based on a composite flood severity index parameter (DV).

Graham (1999) defined DV as DV = $(Q_{df} – Q_{2.33})/W_{df}$, where:

Q_{df} = discharge at a particular site caused by the dam failure.

$Q_{2.33}$ = mean annual flood discharge at that site (approximately bankfull flow rate).

W_{df} = maximum width of flooding caused by the dam failure at the same site.

When flood depths are less than 3.3 m (10 ft) or DV is less than 4.6 m^2/s (50 ft^2/s), flood severity should be low. When depths are greater than or equal to 3.3 m, or DV is greater than 4.6 m^2/s, then flood severity should be medium when not high. Flood severity should only be classified as high when a dam fails nearly instantaneously, thereby failing with seconds, and only where flood waters are close enough to the dam to be “very deep”.

Warning is defined as one that comes from the media or an official source and warning time (WT) is categorized as follows:

None: Only the sight and sound of the approaching flood serves as a warning, quantified as less than (<=) 15 mins.

Some: Officials or the media begin warning the subpopulation 15 – 60 minutes before the flood arrives.

Adequate: Officials or the media begin warning the subpopulation more than 60 minutes before the flood arrives.

Because warning time (WT) is a trichotomous division, for simplicity, the three categories, namely, “None”, “Some” and “Adequate”, are represented numerically and simplified as shown in Table E1.

Flood severity understanding (FSU) is either vague (V: warning issuers have not yet seen an actual dam failure or do not comprehend the true magnitude of the flooding) or precise (P: warning issuers have an excellent understanding of the flooding due to observations of the flooding by themselves or others). FSU is not applicable when WT is less than 15 mins.

Graham (1999) provides an expected (mean) value (mid), a low value and a high value for the proportional life loss (P) for each of the 15 possible categories of flood severity, warning time, and flood severity understanding. These values are shown in Table E.1.

Table E.1: Rate of Life Loss as a Function of Flood Characteristics

Flood Severity Index (m^2/s)	Warning Time (mins)	Warning Time (mins)	Warning Time (mins)	Flood Severity Understanding	Flood Severity	Loss of Life as a Proportion (P) of Population at Risk		
DV	WT: Trichotomous Defintion in Graham (1999)	WT - Numerical Definition	WT - Simplified Representation	FSU	FS	Low Value of P	Mid Value of P	High Value of P
DV <= 4.5	None	WT <= 15	15	N/A	L	0	0.01	0.02
DV <= 4.5	Some	15 < WT <= 60	30	V	L	0	0.007	0.015
DV <= 4.5	Some	15 < WT <= 60	30	P	L	0	0.002	0.004
DV <= 4.5	Adequate	WT > 60	90	V	L	0	0.0003	0.0006
DV <= 4.5	Adequate	WT > 60	90	P	L	0	0.0002	0.0004
4.5 < DV <= 10	None	WT <= 15	15	N/A	M	0.03	0.15	0.35
4.5 < DV <= 10	Some	15 < WT <= 60	30	V	M	0.01	0.04	0.08
4.5 < DV <= 10	Some	15 < WT <= 60	30	P	M	0.0005	0.02	0.04
4.5 < DV <= 10	Adequate	WT > 60	90	V	M	0.0005	0.03	0.06
4.5 < DV <= 10	Adequate	WT > 60	90	P	M	0.0002	0.01	0.02
DV > 10	None	WT <= 15	15	N/A	H	0.3	0.75	1
DV > 10	Some	15 < WT <= 60	30	V	H	0.3	0.75	1
DV > 10	Some	15 < WT <= 60	30	P	H	0.3	0.75	1
DV > 10	Adequate	WT > 60	90	V	H	0.3	0.75	1
DV > 10	Adequate	WT > 60	90	P	H	0.3	0.75	1

V: Vague	L: Low
P: Precise	M: Medium
N/A : Not Applicable when WT <= 15 mins	H: High

E.2.1.2 Brown and Graham 1988

Brown and Graham (1988) developed an empirical formula to estimate the loss of life due to a dam failure for the US Bureau of Reclamation (USBR). The regression-type equation relate loss of life to warning times based on fatalities recorded during past dam failures. The flood events they analyzed appeared to fall in two groups: cases in which warning times and implementation of evacuation plans were quite successful and loss of life was low or absent, and cases where warning was minimal or non-existent and the fatality rate was high. The warning time was defined as the time between when the people find out the dam is going to fail and when the dam actually fails. By measuring the loss of life against the total population of past dam failures, Brown and Graham (1988) constructed graphs for two cases: one for insufficient warning times, i.e., under an hour and a half, and another for sufficient warning times.

For insufficient warning time (defined as less than 1.5 hrs),

LOL $= PAR^{0.6}$

For adequate warning time (greater than 1.5 hrs),

LOL = 0.0002PAR

where,

LOL = estimated loss of life

PAR = population at risk

T = warning time (hrs)

If lead time is very short (less than 15 minutes), then DeKay and McClelland (1993), in reviewing the results of further analysis carried out by the USBR on the data used by Brown and Graham (1988), gives a very approximate estimate of LOL as:

LOL = 0.5(PAR)

E.2.1.3 DeKay and McClelland 1993

Using the same dataset as Graham and Brown (1988) and additional historical events, DeKay and McClelland (1993) developed an empirical equation relating LOL to PAR and warning time as a continuous variable:

L(p) = 0.146 - 0.478(ln[PAR]) - 1.518T

where,

L(p) = ln([LOL/PAR]/[1-{LOL/PAR}])

LOL = estimated loss of life

PAR = population at risk

T = warning time (hrs)

McClelland (2000) defines warning time as the difference in time from when the first warning is given of a dam break or an impending dam break and the time of the leading edge of potentially lethal flood waters first arrive at the leading edge of a PAR zone. DeKay and McClelland (1993) suggest that “no one who is more than 3 hr travel time below the dam should be included in the PAR”. DeKay and McClelland (1993) also modified their equation to consider cases where the flood is confined to narrow valleys (high flood depth and flow velocity) and other cases where the flood is conveyed along wide flood plains (low flood depth and flow velocity).

E2.2 Example Estimation of Loss of Life Using Graham (1999)

E2.2.1 Permanent Population at Risk

The permanent population at risk (PAR) includes people occupying residences, businesses, commercial entities, institutions, etc., for most of the day or night and who may be affected by a flood following the breach of a dam. The effect may range from inconvenience to loss of life.

An application of the approach suggested by Graham (1999) for estimating loss of life (LOL) from a permanent PAR, using hypothetical flood characteristics and example permanent populations at risk as shown in Table E2, is illustrated in Table E3. A permanent PAR of 3 is assumed in each house affected by the flood. Based on a flood arrival time of ½ hr and 2 hrs to peak flood level at House 1 (as given in Table E2), it is expected that there will be virtually no warning time at House 1, understood as the time before the occupants realize the severity of the situation, mobilize to evacuate the house and exit the danger zone before the flood reaches its peak level. The warning time would therefore be categorized as “None” (less than 15 mins warning) according to the definition in Graham (1999) and is simplified to “15” in Table E3. In contrast to the example of House 1 in Table E2, the flood wave would arrive at the village downstream of the dam about 3 hrs after the dam breaches and the time to peak flood level is about 8 hrs. The occupants of the 8 houses can be expected to have “some” warning of the impending flood either through the media alerted by upstream observers or through the appropriate local authorities. The warning time has been assumed to be less than 1 hr for the example in Table E3. Warning times can be greater than 1 hr if there is an effective emergency preparedness plan in place. However, given the distances between communities in rural prairie settings, notifications through the media or by authorities can take time. Hence, the warning time in Table E3 has been assumed to be between 15 and 60 mins (or, simplified to 30 mins). Given that there would likely not be an immediate understanding of the severity of the flood either by observers or the authorities, the flood severity understanding has been classified as “Vague” in the example in Table E3. Given that the village is affected about 3 hrs before the flood waves arrives at the industrial site, for the latter case, the warning time has been assumed to be greater than 1 hr (simplified to 90 mins in Table E3) and it is expected that by this time there would be a good understanding of the severity of the impending flood.

No loss of life is expected at the industrial site. The loss of life at House 1 with a permanent PAR of 3 could range from one (1) to three (3). In contrast, the potential LOL in the village downstream of the dam could range from nearly zero to two (2), even though eight (8) houses may be affected. The combination of the factors related to warning time and flood severity helps in reducing the potential LOL in the village. The total LOL from the permanent PAR ranges from one (1) to five (5).

Table E2: Flood Characteristics near Example Permanent Population at Risk

Community/Structure	Distance from Dam (km)	Flood Arrival Time After Dam Breach (hr)	Time to Maximum Flood Level After Dam Breach (hr)	Flood Peak Discharge (m^3/s)	Maximum Flood Level (m)	Ground Elevation (m)	Flood Depth above Ground Level (m)
House 1	2	0.5	2	1,000	528	525	3
Village (8 houses in flood plain)*	5	3	8	600	524	522	2
Industrial Site (10 staff)**	10	6	12	300	521.2	521	0.2

* All 8 houses are assumed to be at the same ground elevation.
** The entire industrial site is assumed to be at the same ground elevation.

Table E3: Example Application of Graham (1999) for Estimating Loss of Life from a Permanent Population at Risk

		Peak Flood Discharge	Mean Annual Flood Discharge	Maximum Width of Flooding	Flood Severity Index	Warning Time	Flood Severity Understanding	Flood Severity	Expected Loss of Life as a Fraction of Population at Risk			Population at Risk	Range of LOL		
Structure	**Location**	**Qdf (m^3/s)**	**Q2.33 (m^3/s)**	**Wdf (m)**	**DV (m^2/s)**	**WT (mins)**	**FSU**	**FS**	**Low**	**Mid**	**High**	**Permanent PAR**	**Low LOL**	**Mid LOL**	**High LOL**
House 1	e.g. on edge of stream and 2 km from dam	1000	5	90	11	15	N/A	H	0.300	0.750	1.000	3	0.9	2.3	3.0
Village	e.g. partially in flood plain (8 houses) and 5 km from dam	600	5	100	6	30	V	M	0.010	0.040	0.080	25	0.3	1.0	2.0
Industrial Site	e.g. on edge of flood plain, 10 staff at site and 10 km from dam	300	5	930	0.3	90	P	L	0.000	0.000	0.000	10	0.0	0.0	0.0
												Estimated LOL from Permanent PAR	1.2	3.3	5.0

E2.2.2 Temporary Population at Risk

The loss of life assessment also considers temporary population at risk, such as people in recreation areas adjacent to streams and traffic over bridges or crossings, and the likelihood of their exposure to the flood waves. The temporary PAR at road or bridge crossings can be estimated from traffic counts carried out by Saskatchewan Highways and Transportation. Table E4 illustrate the estimation of the temporary PAR at road and bridge crossings that are in the flood plain downstream of the dam.

The average daily traffic (ADT) at each road or bridge crossing is assumed to be over a 10-hr period during the day. Night time traffic is assumed to be about 30% of the ADT over a 10-hr period for major roads such as highways and 20% for minor roads such as access roads. It is assumed that the day-time traffic that could be affected by flooding of the crossings could be equivalent to that over a 1-hr period when the flood level is at its peak. It is anticipated that the appropriate authorities will have closed access to these roads within the 1-hr period if the flooding occurs during day time. The temporary PAR during the day time can be estimated as the product of the expected traffic over a one hour period and a probability that travellers could inadvertently enter the flood zone at the bridge or road crossings. The probability is a function of the flood arrival time and the time between flood arrival and peak flood water level, with probabilities of entering the flood zone decreasing with increasing flood arrival and peaking times because of the greater likelihood of the authorities being notified of an emerging hazardous situation.

The potential loss of life is expected to be higher during night time flooding of roads because of low visibility and perhaps slower notification to and response from emergency authorities. The duration that the flood levels remain just below or above the level of the road or bridge crossings at night may pose additional risk because the reduced traffic rate at night may diminish the chances that a car being swept away may be noticed by trailing traffic. The traffic that may be affected by the flood conditions is therefore estimated as the 1-hr night traffic increased by 25% for each additional hour that the flood level stays above the road crossings up to a maximum of twice the 1-hr traffic. It is expected that there would sufficient time within five hours of the flood overtopping the roads for the appropriate authorities to have been notified and to have responded by closing the affected roads. Notwithstanding the higher likelihood of fatalities at night, the first traffic arrivals near the flooded crossing may realize the dangerous conditions if the increase in flood levels is gradual rather than sudden. Hence, the probability that traffic will enter the flooded zone at a crossing will be dependent to some extent on the time that the flood takes to reach its peak level.

In the examples given in Table E4, there are two access road and two highway crossings. The average daily traffic count has been obtained from Saskatchewan Highways and Transportation for illustration purposes only. Based on the probabilities of traffic entering the flood zone and the flood characteristics, the day-time temporary PAR at the crossings vary from zero (0) to about two (2). The night-time temporary PAR in higher and ranges from about one (1) to about five (5).

Using the same flood considerations (severity, warning time, severity understanding) and the fatality rates as for a permanent PAR (see Table E4), Table E5 shows that the expected LOL due to flooding of the four example crossings in Table E5 ranges from about zero (0) to about one (1) according to Graham (1999).

E2.3 Total Loss of Life Estimate

Based on the discussion above, the estimated total loss of life (LOL), including both the temporary and permanent PAR, is expected to range from one (1) to six (6) for the hypothetical cases considered above.

Table E4: Estimation of Temporary Population at Risk at Road or Bridge Crossings from Traffic Counts

		Average Daily Traffic (ADT)*	Estimated Day Traffic over 1-hr period	Estimated Night Traffic over 1-hr period**	Flood Arrival Time	Time Between Flood Arrival Time and Time to Peak Flood Water Level	**Estimate of DAY TIME Temporary PAR**	Duration of Flood Overtopping of Structure	Estimated Night Traffic over D-hr (Flood Overtopping Period)***	**Estimate of NIGHT TIME Temporary PAR**	
Structure Downstream of Dam	**Road Classification**	**ADT**	**DAY-ADT-1 hr**	**NIGHT-ADT-1 hr**	**FAT (hr)**	**TPFWL (hr)**	**Temporary PAR Day**	**D (hr)**	**NIGHT-ADT-D hr**	**Temporary PAR-Night**	**Maximum of DAY or NIGHT PAR**
1st Crossing	Access Road	40	4	1	0.1	1	**0.6**	4	2	**0.6**	0.6
2nd Crossing	Access Road	25	3	1	1	1	**0.4**	8	1	**0.4**	0.4
3rd Crossing	Highway 668	370	37	11	2	3	**0.0**	8	22	**8.9**	8.9
4th Crossing	Highway 761	530	53	16	2	4	**0.0**	7	32	**6.4**	6.4

***** Based on 2006 Rural Municipal Traffic Count by Saskatchewan Highways and Transportation.
***** ADT ASSUMED TO BE OVER 10-HR DAYLIGHT TIME
****** NIGHT TIME TRAFFIC ASSUMED TO BE 30% OF ADT OVER 10-HR PERIOD FOR HIGHWAYS
****** NIGHT TIME TRAFFIC ASSUMED TO BE 20% OF ADT OVER 10-HR PERIOD FOR ACCESS ROADS
******* Night Traffic over D-hr assumed to be proportionally between (NIGHT-ADT-1 hr) and (NIGHT-ADT-1 hr * 2) for D between 1 and 4 hrs, and equal to (NIGHT-ADT-1 hr * 2) for D greater than 4 hrs

Probability of Temporay PAR Entering Flood Zone :

Probability of Entering Flood Zone Inadvertently During DAY TIME

FAT/TPFWL	1 hr	2 hrs	4 hrs	> 4 hrs
1 hr	0.15	0.1	0.05	0
2 hrs	0.1	0.05	0	0
4 hrs	0.05	0	0	0
> 4 hrs	0	0	0	0

Probability of Entering Flood Zone Inadvertently During NIGHT TIME

TPFWL	Probability
1 hr	0.4
4 hrs	0.2
8 hrs	0.05
> 8 hrs	0

Table E5: Example Application of Graham (1999) for Estimating Loss of Life from a Temporary Population at Risk

		Peak Flood Discharge	Mean Annual Flood Discharge	Maximum Width of Flooding	Flood Severity Index	Warning Time	Flood Severity Understanding	Flood Severity	Expected Loss of Life as a Fraction of Population at Risk			Population at Risk	Range of LOL		
Structure	**Location**	**Qdf (m^3/s)**	**Q2.33 (m^3/s)**	**Wdf (m)**	**DV (m^2/s)**	**WT (mins)**	**FSU**	**FS**	**Low**	**Mid**	**High**	**Temporary PAR***	**Low LOL**	**Mid LOL**	**High LOL**
1st Crossing	1 km from dam	1000	5	50	20	15		H	0.300	0.750	1.000	0.6	0.2	0.5	0.6
2nd Crossing	3 km from dam	800	5	100	8	15		M	0.030	0.150	0.350	0.4	0.0	0.1	0.1
3rd Crossing	7 km from dam	500	5	200	2	30	V	L	0.000	0.007	0.015	8.9	0.0	0.1	0.1
4th Crossing	9 km from dam	300	5	200	1	30	V	L	0.000	0.007	0.015	6.4	0.0	0.0	0.1
												Estimated LOL from Temporary PAR	0.2	0.6	1.0

* Temporary PAR estimated from Rural Municipal Traffic Count and Rough Probabilities of Entering Flood Zone at Crossing - see Table IV.4

E3 INFRASTRUCTURE AND ECONOMIC LOSS

Infrastructure losses due to the failure of a dam are based on approximate replacement or repair costs (present value) of such structures where damages (complete or partial) are expected. Economic losses due to the loss of water from the reservoir retained by the dam that failed can be assessed on an understanding of the current uses of water from the reservoir.

E3.1 Infrastructure Loss

An assessment of possible damage to road crossings, bridges and residential houses during a dam breach flood event can be based on the following considerations.

E3.1.1 Road and Bridge Crossings

Bridges (except for low level crossings) are not designed to be overtopped. A crossing that can pass the peak flow without drift touching the girders has a good chance of "surviving" the event, albeit with some damage possible at the abutments. During a dam breach flood event, substantial drift (beaver dams, trees, cows, etc.) picked up the flood wave can be expected. So, a freeboard (from maximum water level to bottom of bridge deck) of less than nominally one (1) m could result in significant structural damage.

Replacement costs of road or bridge crossings can be based on an average of $3,200/m^2 of deck area for typical concrete or timber bridges. The deck area for bridges is nominally estimated as the bridge span plus 3 m on either side for the abutments multiplied by a nominal average road width of 8 m. For culvert crossings, the deck area can be estimated as the number of culverts multiplied by their diameters plus 2 m on either side for end tie-ins multiplied by a nominal average road width of 8 m.

Damages to crossings due to the flood event from a dam breach can be estimated as a rough percentage of the total replacement cost, with the percentage value dependent on the depth of water above deck level and peak channel velocity. Costs for clean-up, regulatory approvals and engineering can nominally be up to 20% of the replacement cost where replacement or major repairs could be required.

Criteria that can be used for estimating damage to road or bridge crossings are as follows:

Flood Depth above Crossing Deck (m)	Maximum Channel Velocity (m/s)	Estimated Damage as a Percentage of the Crossing's Replacement Value
Greater than 0.5 m	Greater than 2 m/s	120%
Between 0 and 0.5 m	Greater than 2 m/s	100%
Between 0 and 0.5 m	Between 1 and 2 m/s	90%
Between -0.5 and 0 m	Greater than 1 m/s	70%
Between -0.5 and 0 m	Less than 1 m/s	30%
Between -1 and -0.5 m		20%
Less than -1 m		0%

E3.1.2 Residential Houses

Damages to houses can be estimated as a rough percentage of the total replacement cost, with the percentage value dependent on the depth of water above ground level near the houses. Costs for clean-up, furniture replacement and temporary accommodation have been estimated to be nominally 15% of the replacement cost where replacement or major repairs could be required.

Criteria that can be used for estimating damage to houses are as follows:

Flood Depth above Ground Level (m)	Estimated Damage as Rough Percentage of House Replacement Value
Greater than 2 m	115%
Between 1 and 2 m	90%
Between 0.5 and 1 m	65%
Between 0.25 and 0.5 m	30%
Between -0.25 and 0.25 m	20%
Less than -0.25 m	0%

E3.2 Loss of Water Uses

Case specific

A rough estimate of the economic costs of disruptions to a water supply can be calculated by estimating the amount of water that will require trucking to satisfy domestic, municipal and industrial users, based on an approximate cost of about \$100 per 15 m^3 of trucked water to the users (the travel distance factor assumed to be incorporated in the average cost).

E3.3 Total Infrastructure and Economic Loss

Total infrastructure and economic loss is therefore estimated as the sum of the each cost estimate obtained above.

E4 DAM REPAIR COST

The cost to repair the dam can be calculated from the volume of earthfill material required to back-fill the breach and a cost per cubic metre of material. For an approximate estimate repair cost, it can be assumed that it costs about \$25 per cubic metre of earth material to fill and grade the breach to bring the dam back to its original dimensions. The costs for the clean-up of the existing structure after breaching, clean-up of immediate downstream reaches, new power lines, regulatory approvals, etc. can be approximated as about 25% of the repair cost. The total cost is then the dam repair cost for the consequence assessment. The cost estimate is only for the purposes of the consequence assessment and should not be used for capital expenditure planning.

APPENDIX F

A CD Containing the Final Report, Dam Breach Flood Inundation Maps, and Model Data Files

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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