29 March 2016
CW2174.RM.REV

Alberta Environment and Parks
9820 106 Street NW
Edmonton AB T5K 2J6

Attention: David McKenna
Project Manager, Resilience and Mitigation Branch

Dear Mr. McKenna:

Re: Impacts of Proposed Flood Protection Dikes at Bragg Creek on Flood Conditions at Redwood Meadows

1.0 Introduction

1.1 Background

The Hamlet of Bragg Creek (Bragg Creek) is located at the confluence of Bragg Creek and Elbow River, approximately 40 km southwest of the City of Calgary (**Figure 1.1**). The June 2013 flood event caused extensive damage along the Elbow River in Bragg Creek. In response to the flooding, Amec Foster Wheeler Environment & Infrastructure (Amec Foster Wheeler, 2014) was retained by the Government of Alberta's Southern Alberta Flood Recovery Taskforce to undertake conceptual design of flood mitigation measures for the Elbow River basin including at Bragg Creek. The conceptual design for mitigation measures at Bragg Creek included flood dikes (to the 1% annual exceedance probability flood level plus 1 m freeboard) with French drains. The preliminary design for the dike system was undertaken by MPE Engineering Ltd. (MPE, 2015). The detailed design and construction of these protection works is forthcoming.

The Townsite of Redwood Meadows (Redwood Meadows) is a community on the Tsuu T'ina First Nation land, located approximately 4.5 km downstream of Bragg Creek (**Figure 1.1**). The June 2013 flood event caused significant flood damage to houses in the Redwood Meadows area, primarily due to the movement of groundwater through alluvial surficial geology upon which most of the houses are constructed. The existing flood protection works suffered minor damage which has since been repaired based on our understanding.

1.2 Description of Problem

A fundamental concept in flood risk management is that the construction of flood protection works should not cause an increase in flood risk elsewhere. Diking may result in increased water levels and velocities due to channel constriction and lost floodplain storage during high flow events. To assess the change in flood conditions at the community of Redwood Meadows due to the proposed dikes in Bragg Creek, the hydraulic effects need to be quantified. This assessment was carried out by reviewing the hydrology and employing a hydraulic model. A desktop review of potential groundwater movements through the alluvial surficial geology was not undertaken at this time.

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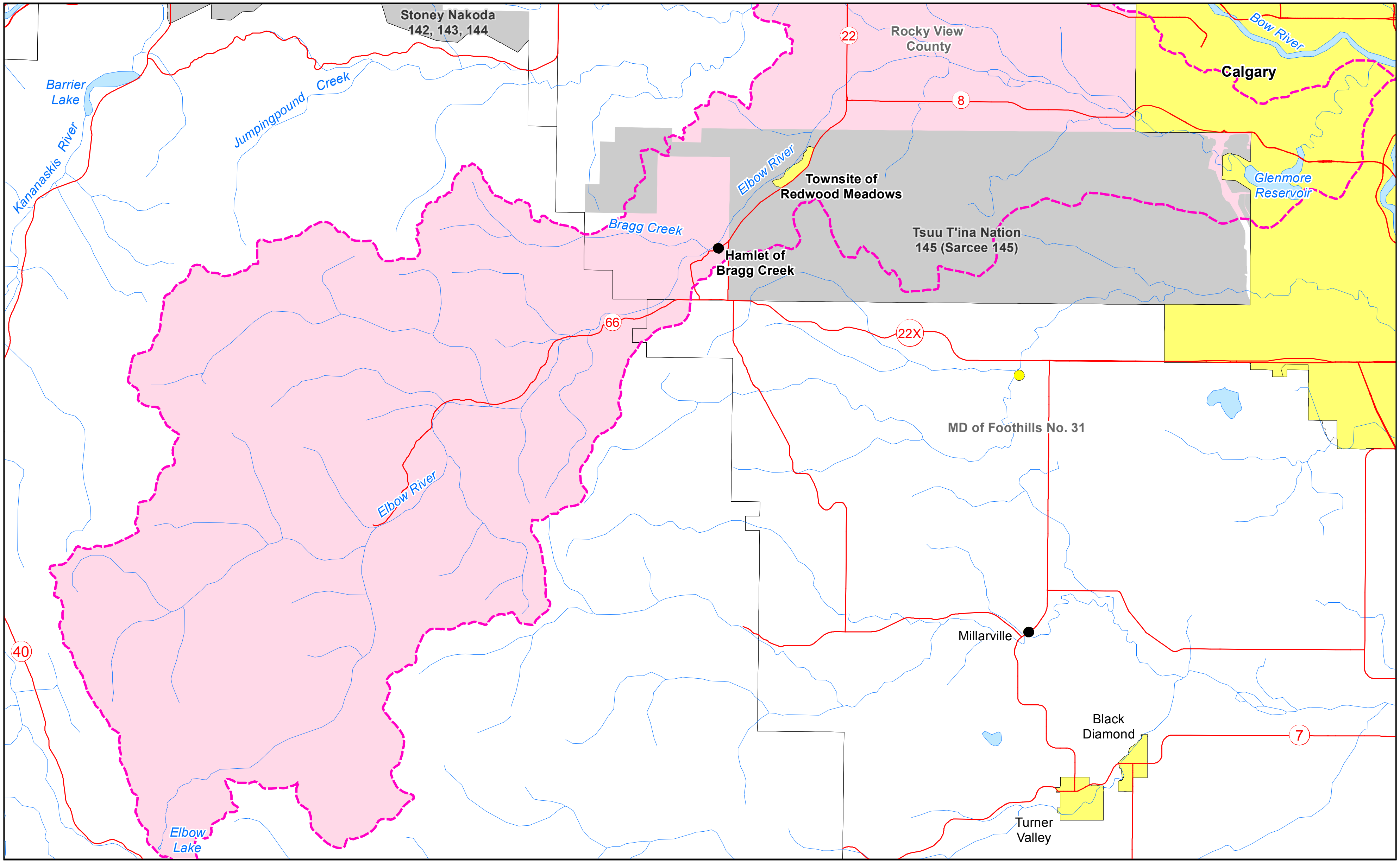

Legend

Community · Lake · Elbow River Basin
Reserve Land · River · Highway

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DATE: March 2016	PROJECT: CW2174.RM.REV	
PROJECTION/DATUM: 10TM/NAD83	ANALYST: JH	

Alberta Environment and Parks
Impacts of Dikes at Bragg Creek
on Flood Conditions at
Redwood Meadows

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Location Map

Figure 1.1

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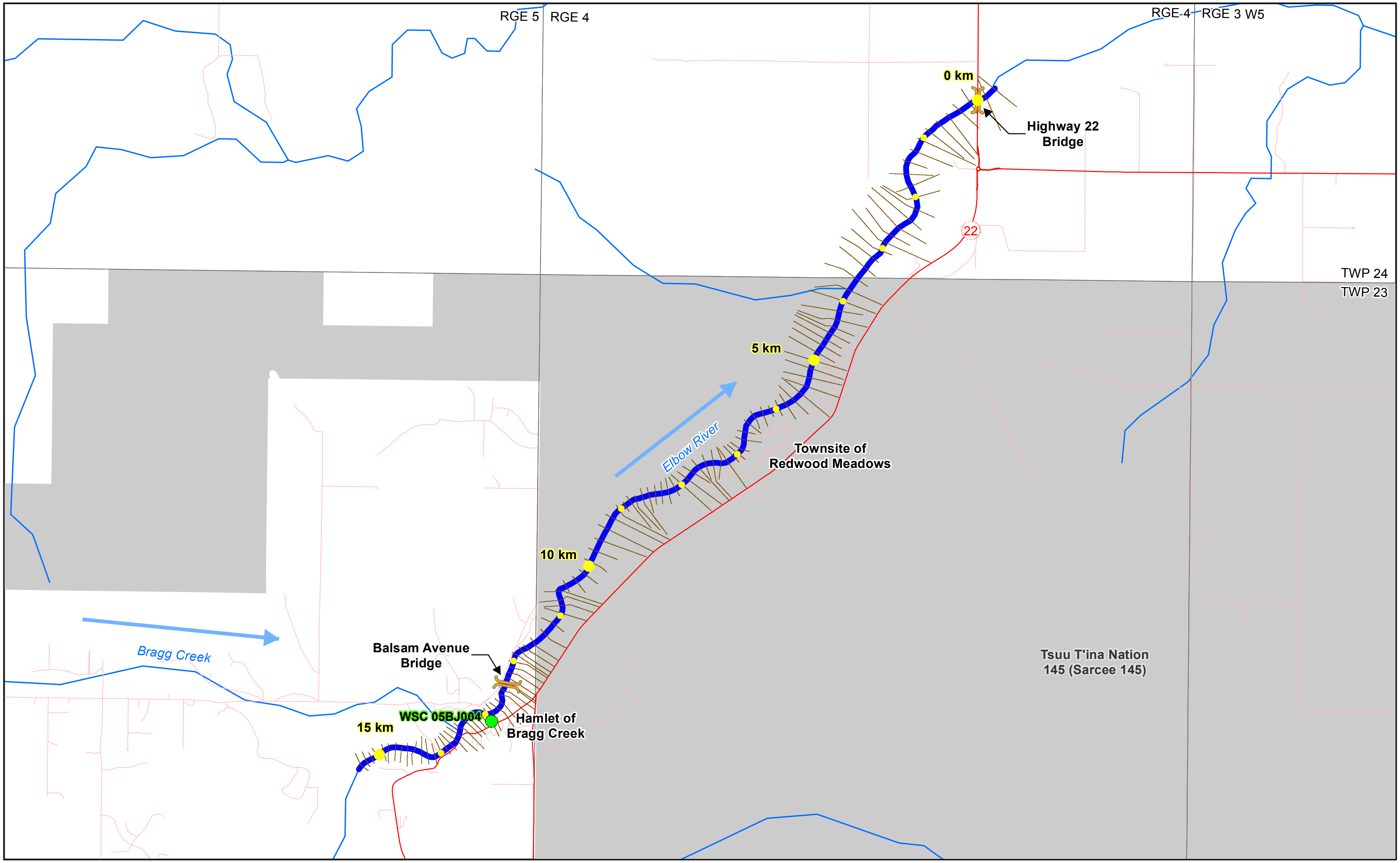
1.3 Description of Study Reach

The 15.5 km long reach of the Elbow River chosen for the study, extending from Bragg Creek to the Highway 22 Bridge (BF 13545), is shown in **Figure 1.2**. The Elbow River at Bragg Creek is located approximately 60 km downstream of the source, Elbow Lake, and approximately 60 km upstream of the confluence with the Bow River (Kellerhals et al., 1972) (**Figure 1.1**).

In the study reach, the Elbow River flows northeast through Bragg Creek and Tsuu T'ina First Nation land before reaching the Highway 22 Bridge. This reach of the river has a stream-cut valley with a wide floodplain. The channel is sinuous and braided where no lateral constrictions are present. The channel bed consists of shallow gravel over moderately erodible rock, while the channel banks are made of sand and gravel, lacustrine deposits, and erodible rock (Kellerhals et al., 1972).



Legend

Reserve Land	Study Reach	1km
River	Cross Section	5 km
Highway	Bridge	WSC Station
Flow Direction		

Note:
Distances are measured upstream from the Highway 22 Bridge.
The study reach extends approximately 300 m downstream of the Highway 22 Bridge.

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DATE: March 2016	PROJECT: CW2174.RM.REV
PROJECTION/DATUM: 10TM/NAD83	ANALYST: JH

Alberta Environment and Parks
Impacts of Dikes at Bragg Creek on Flood Conditions at Redwood Meadows

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Study Reach

Figure 1.2

2.0 Hydrology

2.1 Floods to be Evaluated

The damage caused by the 2013 flood at Bragg Creek led to a proposal for flood protection works consisting of dikes to contain flood flows. The flood magnitude being considered for design is the annual flood with a 1% annual exceedance probability. One of the concerns with the proposed works is the effect the dikes might have on modifying the magnitudes and durations of downstream water levels due to eliminating overbank or floodplain storage. Analysis of those effects requires unsteady routing of the flood hydrograph through the affected river reach.

It is understood that updated estimation of the 1% annual exceedance probability flood hydrology is in progress by others as part of the Elbow River Flood Hazard Study for Alberta Environment and Parks. The results are not yet available. In addition to evaluation of the 1% annual exceedance probability flood, the 2013 flood is also of interest as a benchmark event. Since the 1% annual exceedance probability flood has not yet been defined, the 2013 flood hydrograph was used for hydraulic analysis and modeling. However, additional work was done to attempt to characterize flood hydrograph shapes and to relate flood runoff volumes with flood peak discharges. The results of that work should be applicable to the definition of the 1% annual exceedance probability flood hydrograph, once the peak discharge magnitude has been defined.

2.2 Available Data

The Water Survey of Canada (WSC) hydrometric station Elbow River at Bragg Creek is located at the reach of interest; thus, the data for that station is directly applicable. The station is located approximately 700 m downstream of the confluence with Bragg Creek (**Figure 1.2**). The station information is summarized in **Table 2.1**.

Table 2.1 Elbow River at Bragg Creek – Station Information

Station Number	Latitude	Longitude	Drainage Area (km^2)	Period of Record		
				Start	End	Years
05BJ004	50.94893	-114.571	790.8	1934	present	81

2.3 2013 Flood Hydrograph

The 2013 flood hydrograph is of special interest. The 2013 flood was a spring flood produced by heavy rains combined with snowmelt. The 15 minute instantaneous discharge values were obtained from AEP for the 10-day period extending from 18 June through 28 June (AEP, 2016a). That hydrograph, along with the hydrograph of the mean daily values superimposed, is plotted on **Figure 2.1**. Note that the 2013 discharge data are still considered "preliminary" and subject to revision. The peak instantaneous discharge was 874 m^3/s, and the mean daily peak was 478 m^3/s, both occurring on 20 June.

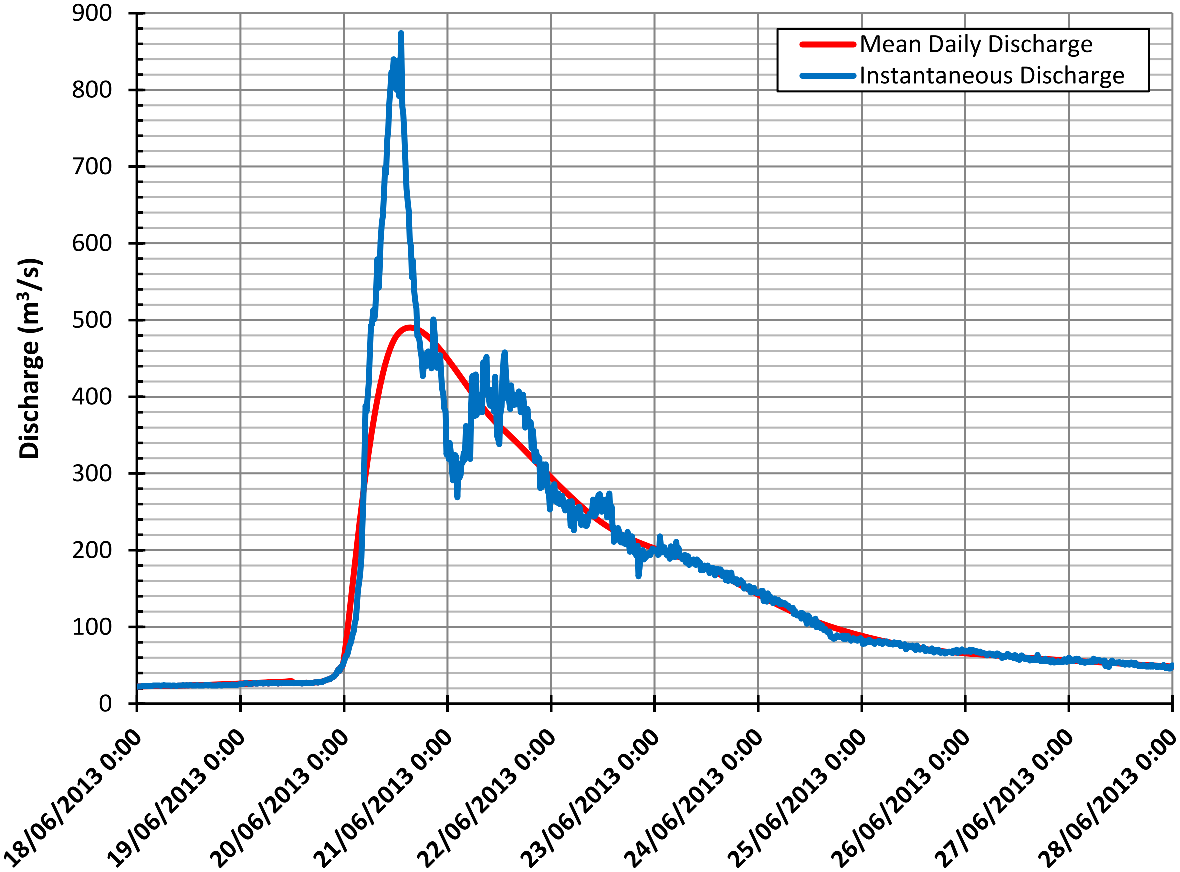

Figure 2.1 Elbow River at Bragg Creek – June 2013 Flood Hydrographs

Note that the mean daily values are plotted as points at the mid-point of each day, then joined with a smoothed line; thus, the latter does not accurately portray the hydrograph volume.

2.4 Development of Design Flood Hydrograph

2.4.1 Flood Hydrograph Characteristics

The available annual peak discharge data were collected from the WSC website and augmented with preliminary data for the most recent years (2013, 2014, and 2015), obtained from Alberta Environment and Parks (AEP, 2016b). The data are listed in **Table A.1** in **Appendix A**. Peak discharge magnitudes for flood events prior to the establishment of the gauging station in 1934 were made by Alberta Environment (1990); however, those data were not considered relevant in the analyses of hydrograph shapes and runoff volumes.

To help characterize the design flood hydrograph, daily discharge data for the entire period of record were also downloaded from the WSC website, and the flood hydrographs for each year were plotted to facilitate visual analysis and comparison. Those plots are not provided here due to the large number of plots involved.

2.4.1.1 Spring Flood Characteristics

The annual flood on the Elbow River at Bragg Creek is usually the spring snowmelt flood, which typically lasts from about 10 to 30 days within the period of early May to the end of June. Spring flood hydrographs are highly variable, frequently showing multiple peaks and not uncommonly having an extended time base. These characteristics are a consequence of the complex upstream watershed, with a network of many tributaries extending over a large elevation range with variable exposures and sensitivities to radiant and convective energy inputs driving snowmelt processes. The annual spring flood is typically produced by combined rainfall and snowmelt, with the latter often spread over an extended time period. Typical spring flood hydrographs are shown in **Figure 2.2**. The 2013 hydrograph peak has been truncated to facilitate better visibility of the other hydrographs. Note that these represent mean daily discharge values which are plotted as point values with a smoothed curve.

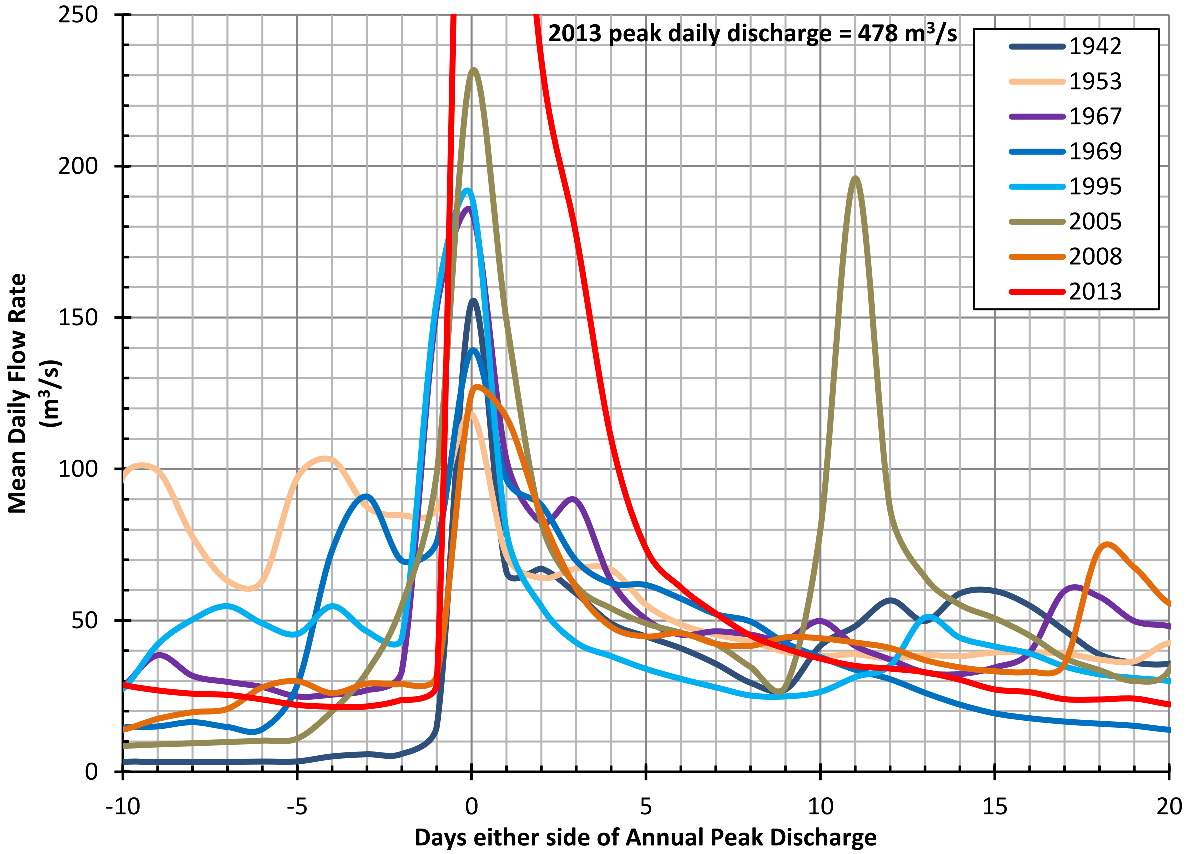

Figure 2.2 Elbow River at Bragg Creek (05BJ004) Typical Spring Flood Hydrographs

2.4.1.2 Summer Flood Characteristics

The annual peak flood has been a summer rainstorm flood for 15 out of the 81 years of record, i.e., approximately one in five annual maximum floods is produced by a summer rainfall event. These floods occur from about the end of June through into mid-September, and, similar to spring floods, can also have secondary peaks and an extended recession limb. Typical summer flood hydrographs are shown in **Figure 2.3**.

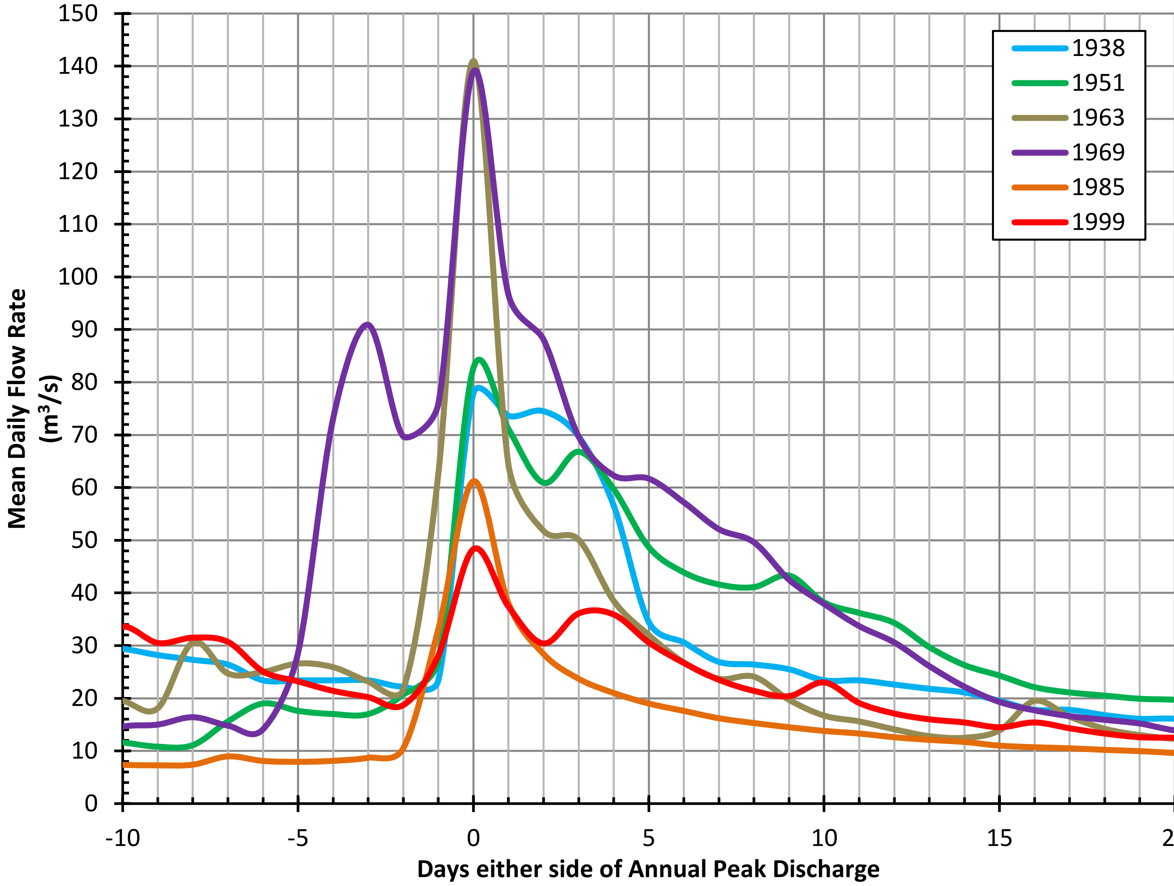

Figure 2.3 Elbow River at Bragg Creek (05BJ004) Typical Summer Flood Hydrographs

2.4.2 Flood Hydrograph Analysis

2.4.2.1 Instantaneous and Mean Daily Peaks

Annual peaks were observed manually for the years 1935 to 1949; thus, only mean daily peaks are available for those years. Since 1950, the station was provided with an automatic recording gauge which provided both instantaneous (Q_i) and mean daily (Q_d) peaks, except for 1993 when there was a malfunction. The relationship between the instantaneous and the mean daily annual peaks was developed by plotting the two peak values for each year and defining a trend line.

All annual peaks were first examined for coincidence. Seven years were identified as having produced an annual instantaneous peak from a different event than the event producing the annual mean daily peak. For each such year, the mean daily peak coincident with the instantaneous peak was extracted from the daily discharge data set and added to the tabulation of annual peaks (**Table A.1**), and those coincident values were then used in the plot of Q_i vs. Q_d. The plot is shown below as **Figure 2.4**.

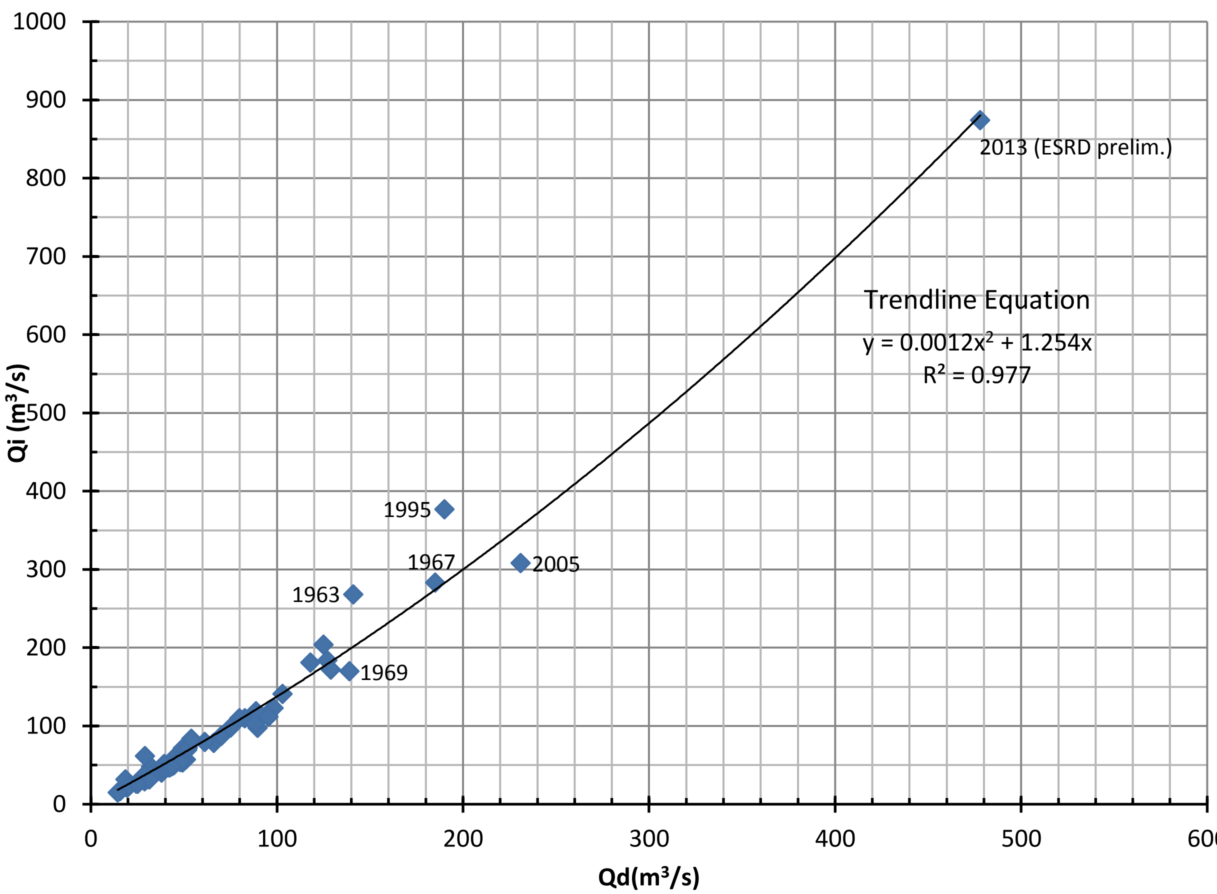

Figure 2.4 Elbow River at Bragg Creek (05BJ004) – Annual Floods Peak Instantaneous Discharge vs. Peak Daily Discharge (1950 to 2015)

The best-fit trend line was found to be a second order polynomial:

$$Q_i = 0.0012\ (Q_d)^2\ 1.254\ Q_d$$

Annual instantaneous peak values were then estimated for the years for which those values were missing, using the trend line equation. The estimated values are shown in **Table A.1** in italics.

2.4.2.2 Flood Volume Analysis

Due to the significant variability in historical flood hydrograph shapes (see Section 2.3 above), the runoff volume of each annual flood was extracted directly from the flood hydrograph, with the start date and end date selected by visual estimation. Subtraction of a base flow volume was considered impractical due to the complex shapes of many of the hydrographs and the difficulty in consistently defining a base flow.

The flood volume was plotted versus the flood daily peak for each flood, with spring and summer events identified separately. The results are presented in **Figure 2.5**, with manually-drawn envelope lines capturing the plotted points[1].

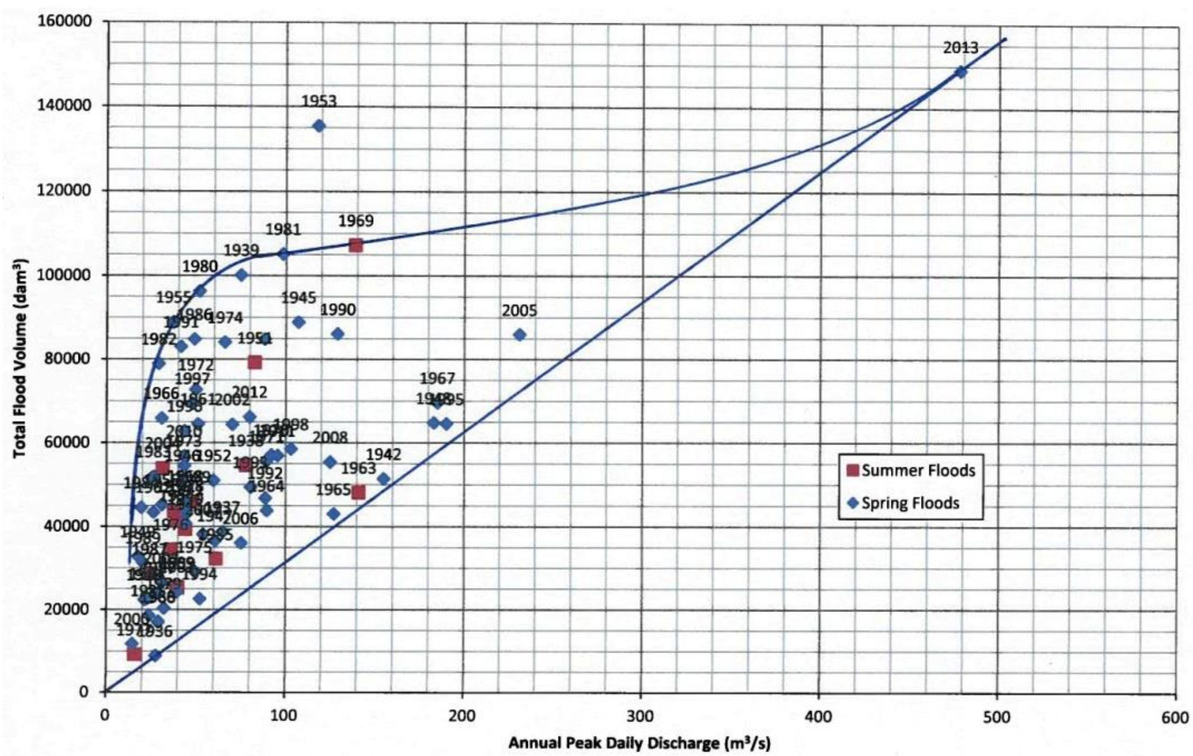

Figure 2.5 Elbow River at Bragg Creek (05BJ004) – Annual Floods Total Flood Runoff Volume vs. Peak Daily Discharge (1934 to 2015)

As is evident from the plot, the flood runoff volume is highly variable, with the plotted points falling within an envelope of values rather than a trend line relationship with flood peaks. There is large variability for the lower peaks, with the range decreasing as the peaks become larger. Both spring floods and summer floods show the same variability.

[1] The 1953 data point is considered an outlier, perhaps due to improper selection of hydrograph end points.

A possible explanation for this variability in runoff volume is that the larger volume floods are generated by snowmelt or rainfall events which extend over all or most of the watershed area, while the lower volume floods tend to be generated by snowmelt or rainfall over only portions of the watershed. Both higher volume and lower volume floods could be associated with similar magnitudes of peak discharge, as the complexity of the upstream watershed typically produces flood runoff hydrographs with multiple peaks and extended durations (see Section 2.3.1 above). The range of variability in flood volume would decrease as the flood peak discharge becomes greater, since larger peaks would need to be generated by runoff from increasingly larger fractions of the watershed area. The envelope of plotted points would thus be expected to diminish in width and approach a single line with increasing peak discharge, as suggested in **Figure 2.5**.

2.4.3 Recommended Design Flood

Once the 1% annual exceedance probability flood peak magnitude has been defined, a hydrograph shape can be developed. The most conservative shape would be a rapid rise to a single peak, much like the 2013 hydrograph, which shows a rise to peak time of about 12 hours. The recession limb shape should then be a smooth decline without secondary peaks; again, the 2013 hydrograph shape can serve as a template, with the secondary peak smoothed out. It is further recommended that the characteristics of the 1% annual exceedance probability flood hydrograph generally conform to the relationships described by **Figures 2.4 and 2.5**, as shown above.

3.0 Hydraulic Analysis

3.1 Introduction

A one-dimensional hydrodynamic model was developed using the U.S. Army Corps of Engineers (USACE) HEC-RAS modeling program to determine the effect of constructing dikes in the Hamlet of Bragg Creek on water level and discharge at the Townsite of Redwood Meadows. An unsteady flow analysis based on the June 2013 hydrograph data was completed for the study reach, which extends from the Hamlet of Bragg Creek to the Highway 22 Bridge (BF 13545) (**Figure 1.2**).

3.2 Available Data

3.2.1 HEC-2 Modeling

The Elbow River at Bragg Creek was modeled by UMA Engineering Ltd. (UMA) using the USACE HEC-2 model as part of the previous flood hazard study of the Elbow River and Bragg Creek through Bragg Creek (UMA, 1992). The model was based on survey data and a peak discharge for the Elbow River of 842 m^3/s, which was the design event at the time. **Figure 3.1** shows the extents and location of the previously developed flood hazard model along the Elbow River. The survey data has a total of 44 cross sections, 38 of which are located along a 3.9 km reach of the Elbow River at the Hamlet of Bragg Creek. The remaining six are located along a flow split which begins 3.3 km downstream of the start of the study reach and approximately 200 m downstream of the Balsam Avenue Bridge. These six cross sections are part of a floodplain “spill” area on the right bank of the Elbow River.

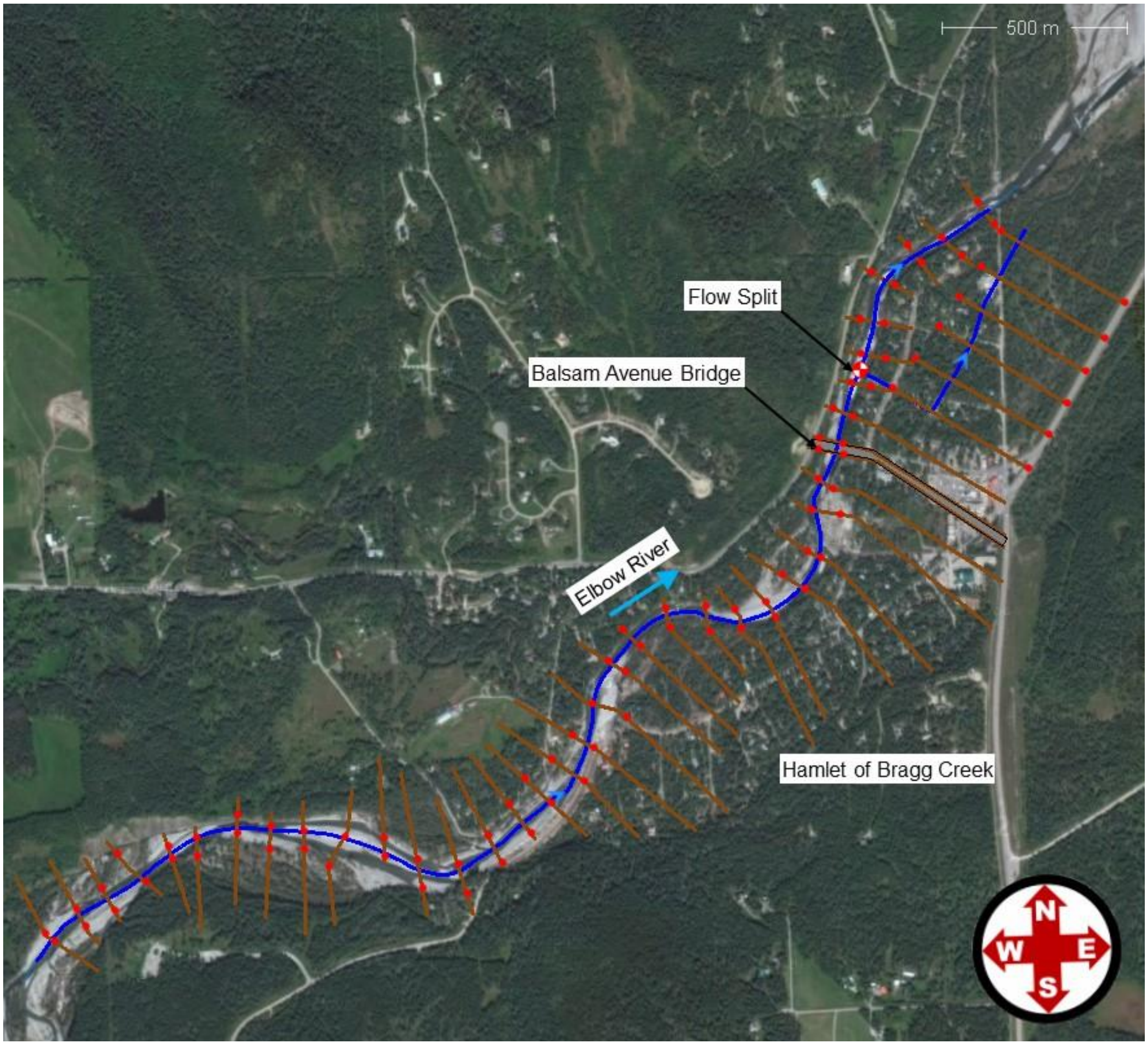

Figure 3.1 Model Configuration at the Elbow River at Hamlet of Bragg Creek based on UMA 1992.

3.2.2 LiDAR Data

Amec Foster Wheeler obtained Light Detection and Ranging (LiDAR) data from AEP (2015). The LiDAR data was flown on 2 October 2015, which is normally a time of low flow in the Elbow River. At the time the LiDAR data was flown, the discharge in the Elbow River at Bragg Creek was in the range of 10 m^3/s (AEP, 2016b). The data has a vertical accuracy of ±15 cm at the 95% confidence interval.

3.2.3 Dikes

In the preliminary design report for the dike system undertaken by MPE, the following five dikes are proposed: Bracken Road Dike, West Dike South, West Dike North, East Dike, and Yoho Tinda Dike (MPE, 2015). The table below shows the approximate length and height of each dike. The dike heights were based on the results of USACE HEC-RAS steady flow model constructed by MPE, which simulated a 1% annual exceedance probability of 1,050 m^3/s, plus 300 mm of freeboard (MPE, 2015). **Figure 3.2**, adapted from the MPE report (2015), outlines the location of the dikes in blue.

Table 3.1 Dike Lengths and Heights from the MPE Engineering Ltd. Report (2015)

Dike Name	Length (m)	Range of Height (m)
Bracken Road Dike	1,260	0.8
West Dike South	1,420	1.3 to 3.1
West Dike North	1,380	1.3 to 1.7
East Dike	3,040	0.5 to 2.9
Yoho Tinda Dike	1,320	0.4 to 1.1

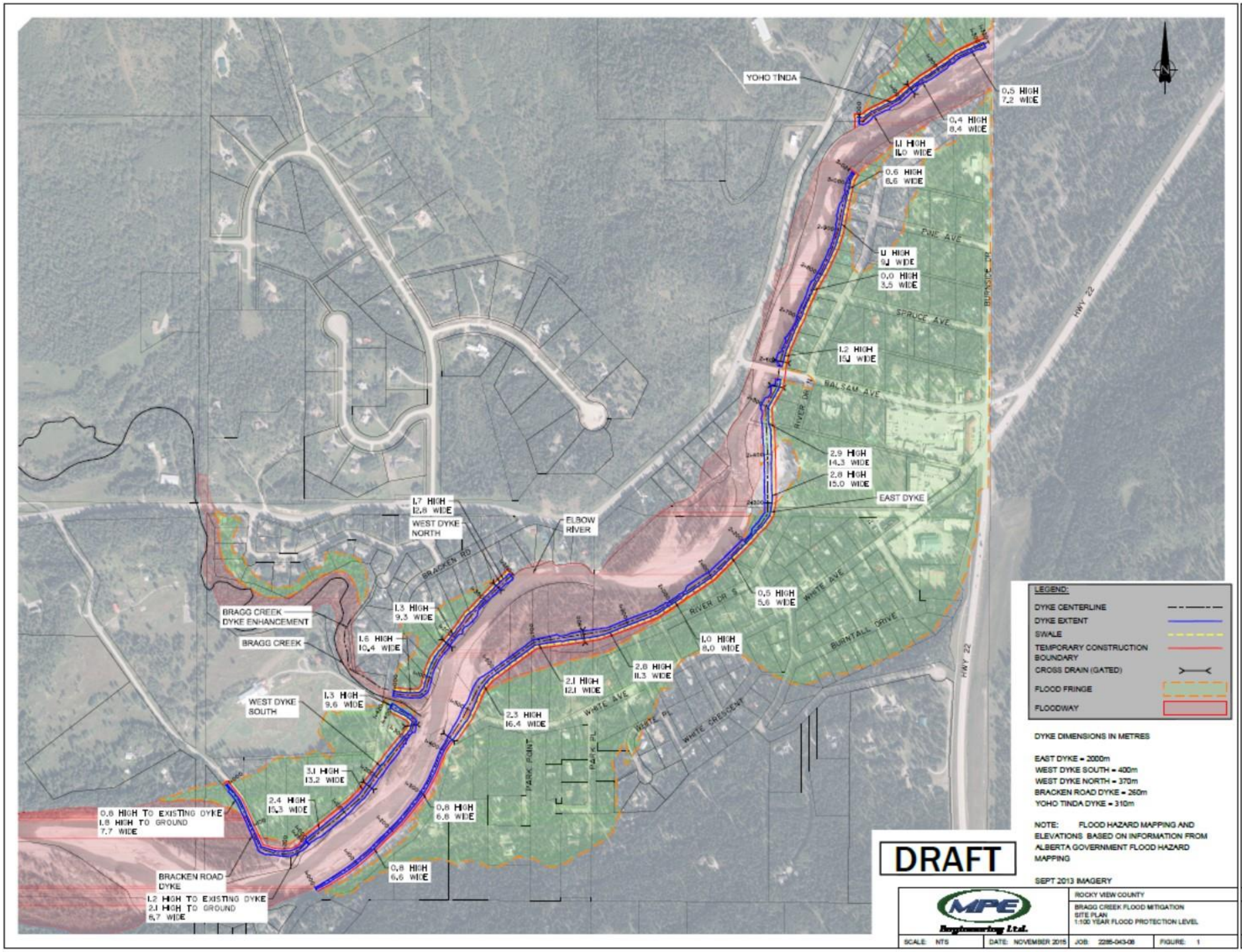

Figure 3.2 Preliminary Dike Locations adapted from MPE Engineering Ltd. Report (2015)

3.3 Methodology

3.3.1 Model Construction

LiDAR data obtained from AEP (2015) was incorporated into the previously developed flood hazard model and compared to survey data used by UMA (1992). Significant differences existed between the data sets, including inconsistent variance in elevations at the cross sections of up to 0.80 m, as well as inconsistent differences in the horizontal datum. These discrepancies were assumed to be caused by erosion and channel migration arising from notably high flows in 1995, 2005, and 2013, as well as the possibility that the survey data has a different datum than the LiDAR data. Since the bare earth LiDAR data was the most recent and detailed data available, it was used for the overbank and channel areas for all cross sections in the model. It is important to note that LiDAR does not penetrate water. Since the LiDAR was obtained at a time of low flow in the river, the water surface is assumed to be approximately equal to the riverbed and representative for the purpose of this assessment.

Model cross sections were placed in the same locations along Elbow River in Bragg Creek as those of the HEC-2 model. Downstream of Bragg Creek, cross sections were spaced between 100 and 200 m apart, extending to approximately 300 m downstream of the Highway 22 Bridge, for a total model length of 15.44 km (**Figure 1.2**). For the purpose of this report, the Highway 22 Bridge crossing is located at 0 km. Bragg Creek is located between 11.57 and 14.27 km upstream of this point; Redwood Meadows is located between 4.49 and 7.69 km upstream of this point.

The inflow hydrograph used was based on preliminary 15-minute interval data from the June 2013 event, as provided by AEP (2016a) and shown in **Figure 2.1**. The peak instantaneous discharge modeled was 874 m^3/s. The downstream boundary condition used was a normal depth with friction slope of 0.005052 m/m, as based on LiDAR data. The downstream boundary is sufficiently far away from Redwood Meadows as to not have a measurable effect on the results at points of interest.

3.3.2 Model Calibration

The model was calibrated using observed High Water Marks (HWM) from Flood Imagery Air Photos from AEP (AEP, 2013) and HWM given by MPE (2015), from AEP HWM surveys. The table below shows the approximate location of the HWM, HWM elevation, and modeled water surface elevations. Differences in elevation between observed and simulated water surface elevations can be attributed to the following factors:

- The amount of debris transported within the channel during the high flow event;
- Uncertainty in discharge measurement, as the most recent estimates of discharge data are still considered “preliminary” and subject to revision;
- Uncertainty in HWMs, as these are based on air imagery often obtained after the peak flow; and
- Model cross sections were based solely on LiDAR obtained at a time of low flow, because no current bathymetric survey data were available.

The differences between simulated and observed water surface elevations are comparable with the hydraulic analyses conducted by MPE (2015) and UMA (1992).

Table 3.2 Comparison of High Water Marks for the June 2013 Flood

Location	Distance upstream of Highway 22 Bridge (km)	Source*	Maximum Water Surface Elevation (m)		
			Observed or Estimated	Simulated	Elevation Difference
Hamlet of Bragg Creek	14.05	MPE	1309.8	1309.31	-0.49
	13.44	MPE	1306.4	1305.62	-0.78
	13.17	AEP	1303.4	1303.52	0.12
	12.84	AEP	1300.5	1300.94	0.44
	12.59	AEP	1298.2	1298.92	0.72
	12.41	MPE	1297.1	1298.52	1.42
	11.86	MPE	1293.1	1293.26	0.16
Tsuu T'ina Nation	11.54	MPE	1290.8	1291.01	0.21
	9.04	AEP	1270.5	1271.57	1.07
Redwood Meadows	7.19	AEP	1259.1	1258.46	-0.64
	5.22	AEP	1244.1	1243.68	-0.42
Rocky View County	2.76	AEP	1223.0	1223.25	0.25

* MPE - MPE Engineering Ltd. (2015)
AEP - Alberta Environment and Parks (2013)

A Manning's "n" roughness value of 0.042 was determined for the channel, as based on HWM information shown above. This value is similar to the channel roughness of 0.04 used by MPE (2015). By comparison, the 1992 HEC-2 model used a channel roughness value of 0.053.

An overbank roughness value of 0.173 was used for overbank areas within the Hamlet of Bragg Creek because of the significant amount of vegetation and building obstructions, while the remaining overbank areas were assigned a roughness value of either 0.085 or 0.065 based on aerial imagery of the study area. These three values were based on the overbank roughness values from the 1992 HEC-2 model. MPE used a roughness value of 0.1 for all overbank areas (2015). Based on trial runs, water levels are not highly sensitive to selection of overbank roughness.

3.3.3 Model Simulation Conditions

Model simulations were carried out for the following three conditions:

- Existing conditions (no dikes);
- Proposed dikes (based on MPE design profiles); and
- Dikes with no overtopping.

The proposed dikes condition incorporated the preliminary dike design based on drawings in the MPE report (2015). After modeling the proposed dikes condition, it was noted that some of the dikes in Bragg Creek were overtopped, so a third condition, called dikes with no overtopping, was also included as part of this study. Similar to the proposed dikes condition, the dikes with no overtopping condition is based on the preliminary dike design, with several of the dikes raised to prevent overtopping at peak water levels.

3.4 Model Simulation Results

Unsteady flow modeling was carried out on the three conditions described above, with the June 2013 inflow hydrograph using HEC-RAS. **Table 3.3** below, shows the maximum water surface elevation and discharge at points of interest along the study reach. The upstream end of the study reach has the highest maximum water surface elevation and discharge for all three model conditions. The maximum water surface elevation at the upstream edge of the Balsam Avenue Bridge (BF 07425) is between 0.40 and 0.80 m higher than the maximum water surface elevation at the downstream edge of the Balsam Avenue Bridge for all three model conditions. This demonstrates the strong backwater effect created by the flow constriction at the bridge. By about 400 m downstream of the Balsam Avenue Bridge, differences in maximum water surface elevation are considerably smaller. The difference in maximum water surface elevation continues to decrease downstream and at Redwood Meadows, which is located 7.08 km downstream of the Balsam Avenue Bridge (5.28 km upstream of the Highway 22 Bridge), the maximum water surface elevation is effectively the same for all three model conditions. The values of peak total discharge change dramatically throughout the study reach due to the flow attenuation which occurs at the Balsam Avenue Bridge. The difference in peak discharge between each model condition is less than 10 m^3/s. At Redwood Meadows, the discharge for all three conditions varies by a maximum of 1.2 m^3/s. This difference is within the anticipated accuracy of the model and thus it is insignificant.

Table 3.3 Location, Maximum Water Surface Elevation, and Discharge at Various Points along the Study Reach

Location Description	Distance Upstream of Highway 22 Bridge (km)	Maximum Water Surface Elevation (m)			Discharge (m^3/s)		
		Existing Conditions	Difference With		Existing Conditions	Difference With	
			Proposed Dikes	Dikes With No Overtopping		Proposed Dikes	Dikes With No Overtopping
The farthest upstream cross section in the study reach, about 3.1 km upstream of Balsam Avenue Bridge	15.44	1,318.63	0	0	874.0	0	0
530 m upstream of Balsam Avenue Bridge	12.90	1,301.45	0.09	0.01	833.1	-2.8	-2.7
35 m upstream of Balsam Avenue Bridge	12.41	1,298.52	0.95	1.37	829.7	-5.4	-8.6
35 m downstream of Balsam Avenue Bridge	12.34	1,298.08	0.57	1.43	829.7	-5.4	-8.6
475 m downstream of the Balsam Avenue Bridge	11.90	1,293.70	-0.15	-0.15	823.6	-2.3	-2.7
In Redwood Meadows	5.28	1,244.09	0	0	815.6	-1.2	-0.2
The farthest downstream cross section in the study reach	-0.31	1,203.62	0	0	813.7	-0.2	0.4

3.4.1 Maximum Water Surface Elevation

The profile plots from the HEC-RAS model of the three conditions are shown below. **Figure 3.3** shows the profile from 0.31 km downstream of the Highway 22 Bridge to 7.69 km upstream of the Highway 22 Bridge. Redwood Meadows is located between 4.49 and 7.69 km upstream of the Highway 22 Bridge. **Figure 3.3** illustrates the water surface profiles for all model conditions are coincident at the downstream part of the study reach.

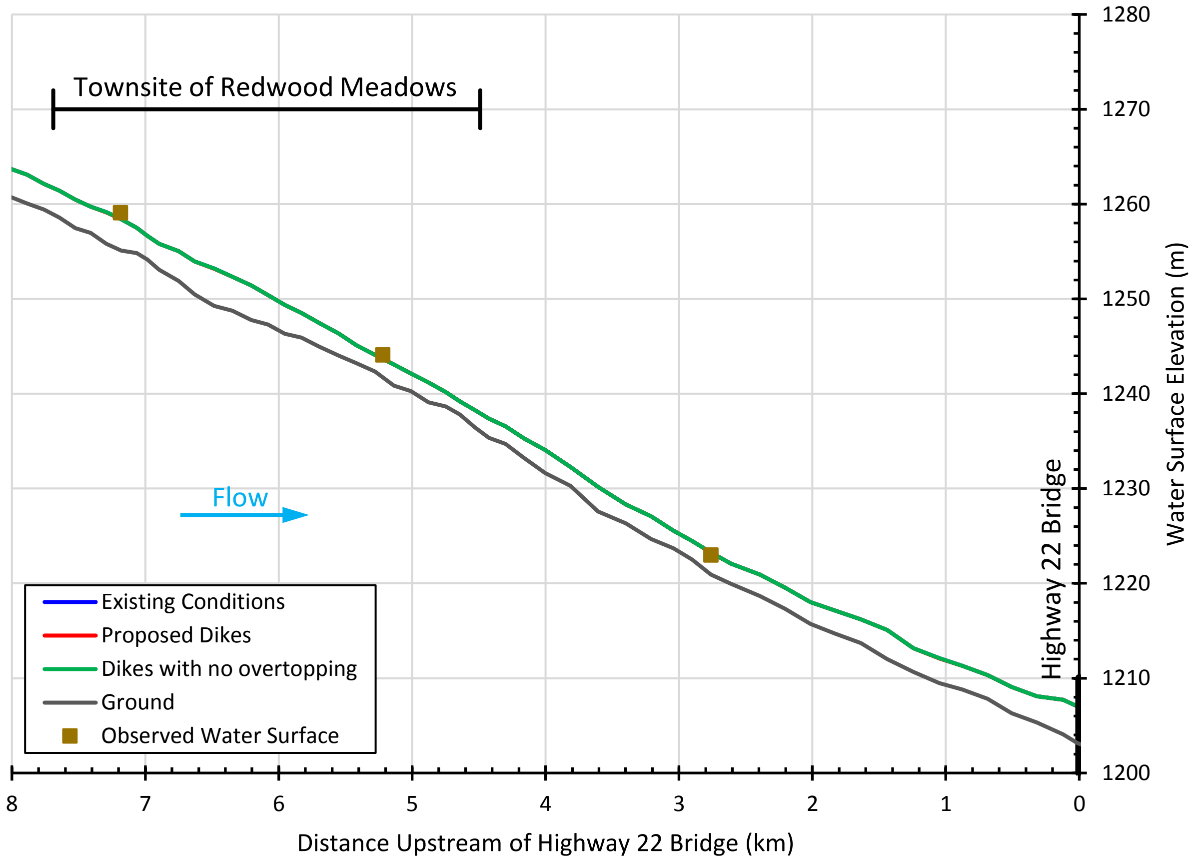

Figure 3.3 Simulated Maximum Water Surface Profiles from Redwood Meadows to Highway 22.

Figure 3.4 shows the simulated maximum water surface profiles within the Hamlet of Bragg Creek. The Hamlet of Bragg Creek is located between 11.57 and 14.27 km upstream of the Highway 22 Bridge. **Figure 3.4** shows the dikes significantly increase the water level in the channel upstream of the Balsam Avenue Bridge within the Hamlet of Bragg Creek. In the proposed dikes condition, the largest difference in water level is 1.09 m above the existing conditions, which occurs about 1 km upstream of the Balsam Avenue Bridge within the Hamlet of Bragg Creek. The modeled water levels from the June 2013 inflow hydrograph in the Proposed Dykes condition overtop the dikes. In the dikes with no overtopping condition, an increase of 1.43 m occurs at the downstream edge of the Balsam Avenue Bridge, which is the most significant difference in water level, when compared with the existing conditions. This illustrates how the addition of dikes amplifies the backwater effect created by the bridge.

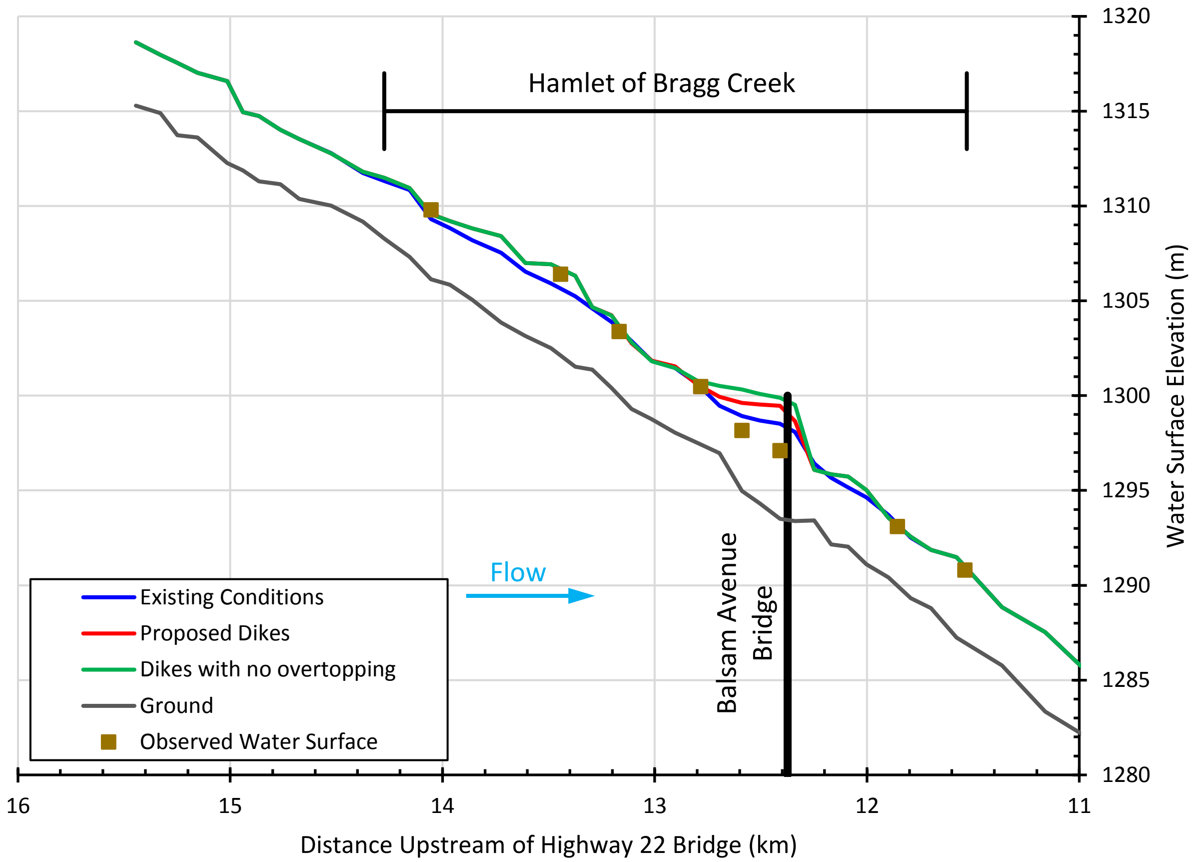

Figure 3.4 Simulated Maximum Water Surface Profiles through the Hamlet of Bragg Creek.

Figure 3.5 displays the difference in maximum water surface elevation along the study reach between each of the model conditions with dikes and the existing condition. This figure helps to illustrate the considerable difference in maximum water surface elevation between the proposed dikes and existing condition (red line) and the dikes with no overtopping and existing condition (green line) near the Balsam Avenue Bridge (about 12.4 km upstream of the Highway 22 Bridge). Note that the slight decrease and subsequent increase in water surface elevation downstream of Balsam Avenue Bridge can be attributed to a local expansion and constriction of the channel. Further investigation of this discrepancy could be carried out using more detailed river bathymetry local to the bridge.

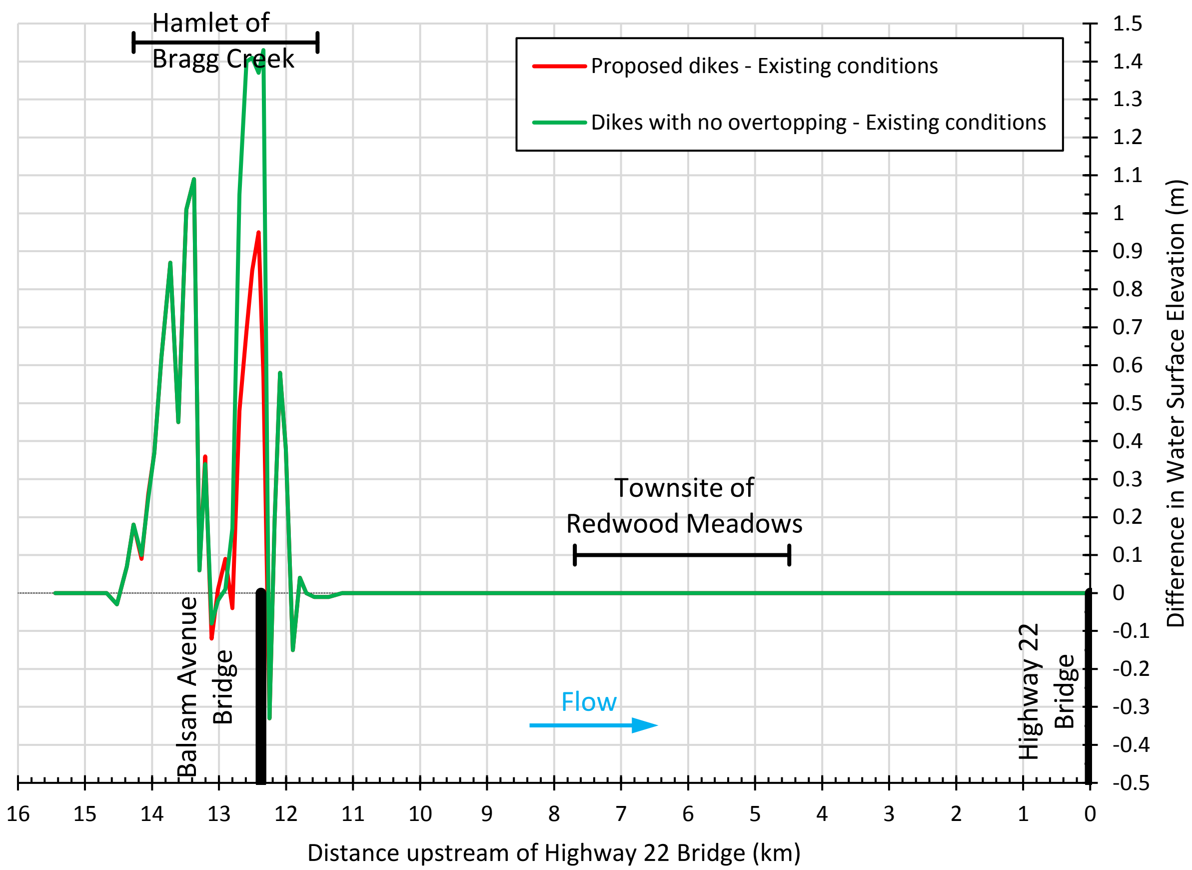

Figure 3.5 Difference in Maximum Water Surface Elevation along the Elbow River from Bragg Creek to Redwood Meadows

3.4.2 Discharge

Figure 3.6 shows the peak total discharge along the study reach. The values of peak total discharge change by approximately 60 m^3/s throughout the study reach. The greatest amount of flow attenuation occurs as a result of flow constriction at the Balsam Avenue Bridge.

The difference in peak discharge between each model condition is minor. At the Balsam Avenue Bridge, the greatest change in peak discharge is 8.6 m^3/s between the existing conditions and dikes with no overtopping. This shows how the increased backwater effect due to the dikes upstream of the bridge dampens the flood wave. Downstream of the Balsam Avenue Bridge, at approximately 11.7 km upstream of the Highway 22 Bridge, the proposed dikes become overtopped again within in the model, resulting in the proposed dikes condition having a lower maximum discharge downstream of this point. At Redwood Meadows, the peak total discharge varies by 1.2 m^3/s between the existing condition and proposed dikes.

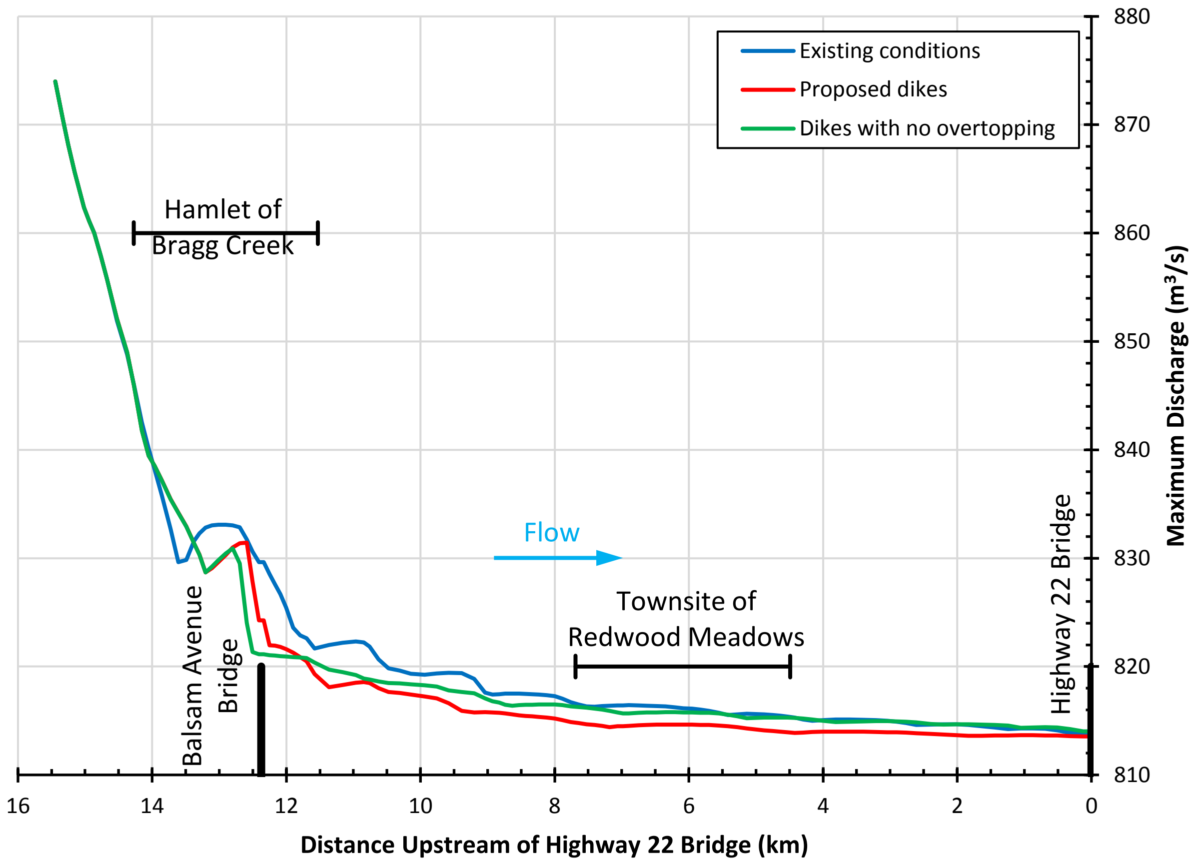

Figure 3.6 Peak Total Discharge along the Elbow River Profile from Bragg Creek to Redwood Meadows

4.0 Summary

An unsteady, one-dimensional hydrodynamic model was used to examine potential impacts of proposed flood protection dikes at Bragg Creek on the flood conditions at Redwood Meadows. Three conditions were created to assess the effects, which included the existing conditions, proposed dikes, and a dikes with no overtopping, as it was found that the proposed dikes were overtopped.

Model results showed that there was no difference in maximum water surface elevation and a highest difference of 1.2 m^3/s in discharge at Redwood Meadows due to the proposed dikes in Bragg Creek. These model results indicate that the flood conditions at the Townsite of Redwood Meadows will not be affected by the proposed construction of dikes in the Hamlet of Bragg Creek.

Upstream of the Balsam Avenue Bridge in the Hamlet of Bragg Creek, model results show a difference of 1.43 m in maximum water surface elevation between the dikes with no overtopping condition and the existing conditions, and 1.09 m in maximum water surface elevation between the proposed dikes and the existing conditions. At the Balsam Avenue Bridge in Bragg Creek, model results showed an 8.6 m^3/s decrease between the existing conditions and dikes with no overtopping. These findings illustrate that the constriction of the bridge creates a strong backwater flow effect upstream of the bridge and resulting flow attenuation. The addition of dikes amplify these effects by further restricting the flow to the channel and increasing the volume of water directed to the area upstream of the bridge.

As discussed in this report, the effect of the proposed dikes and hydraulic constriction at the Balsam Avenue Bridge have indicated that maximum water levels for the June 2013 flood event exceed the design crest elevation of the dikes upstream of the bridge. The hydraulic implications of the bridge and dikes should be investigated further using current field-surveyed river bathymetry to update the hydraulic analysis local to the bridge during detailed design of the dikes.

5.0 Closure

This report has been prepared for the exclusive use of Alberta Environment and Parks. This report is based on, and limited by, the interpretation of data, circumstances, and conditions available at the time of completion of the work as referenced throughout the report. It has been prepared in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

Should you require any additional information, please contact the undersigned.

Yours truly,

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AH/NVG/CF/clm

Attach.

Permit to Practice No. P-4546

6.0 References

Amec Foster Wheeler. (2014). Southern Alberta Flood Recovery Task Force Flood Mitigation Measures for the Bow River, Elbow River and Oldman River Basins, Volume 4 – Flood Mitigation Measures. Calgary, AB.

MPE Engineering Ltd. (2015). Bragg Creek Flood Mitigation Preliminary Engineering Report. Calgary, AB.

UMA Engineering Ltd. (1992). Elbow River at Bragg Creek Hydraulic Study. Calgary, AB.

Kellerhals, R., Neill, C.R., and D.I. Bray. (1972). Hydraulic and Geomorphic Characteristics of Rivers in Alberta. Alberta Environment, Cooperative Research Program in Highway and River Engineering Technical Report. Edmonton, AB.

Alberta Environment. (1990). Flood Frequency Analysis of the Elbow River and Bragg Creek at Bragg Creek. Report by Water Resources Management Service. Edmonton, AB.

Alberta Environment and Parks (2016a). Email from Christy Donley, Data Management Section, 29 February 2016 (esrd.dmnrt@gov.ab.ca).

Alberta Environment and Parks. (2016b). Email from Darcy Talma, Data Management Section, 23 February 2016 (esrd.dmnrt@gov.ab.ca).

Alberta Environment and Parks (2015). LiDAR Data on hard drive (accessed 22 February 2016).

Alberta Environment and Parks (2013). Flood Imagery Air Photos. Available online at http://aep.alberta.ca/forms-maps-services/air-photos/default.aspx (accessed 7 March 2016).

Appendix A

Hydrology Data Tables

Table A.1 Elbow River at Bragg Creek (05BJ004) Annual Peak Discharges (1935–2015)

Year	Peak Daily Discharge (m^3/s)	Date		Peak Instantaneous Discharge (m^3/s)	Date	
		Month	Day		Month	Day
1935	23.6	6	17	*30.2*	*6*	*17*
1936	27.5	6	1	*35.4*	*6*	*1*
1937	64.8	6	13	*86.2*	*6*	*13*
1938	77.9	7	2	*104.9*	*7*	*2*
1939	74.8	6	22	*100.4*	*6*	*22*
1940	21.2	5	25	*27.1*	*5*	*25*
1941	44.5	6	2	*58.1*	*6*	*2*
1942	155	5	11	*223.0*	*5*	*11*
1943	44.2	7	3	*57.7*	*7*	*3*
1944	22.9	6	13	*29.3*	*6*	*13*
1945	107	5	26	*147.8*	*5*	*26*
1946	42.5	5	29	*55.4*	*5*	*29*
1947	60.6	5	10	*80.3*	*5*	*10*
1948	183	5	23	*269.4*	*5*	*23*
1949	17.6	5	22	*22.4*	*5*	*22*
1950	44.2	6	15	58	6	15
1951	82.7	8	30	110	8	30
1952	49.3	6	12	71.4	6	12
	59.7	6	23			
1953	118	6	13	181	6	13
1954	39.4	8	25	43.9	8	25
1955	32.3	5	20	47.6	5	19
	37.1	6	12			
1956	30.9	5	21	35.4	5	21
1957	28.9	6	8	30	6	8
1958	37.9	7	13	40.2	7	13
1959	39.9	6	27	45.9	6	27
1960	28.9	6	3	29.4	6	3
1961	51	5	27	57.2	5	27
1962	26.1	6	16	28.9	6	16
1963	141	6	30	268	6	29
1964	89.5	6	8	97.4	6	8
1965	127	6	18	184	6	18
1966	30.6	6	5	32.3	6	5
1967	185	5	31	283	5	31
1968	43.9	6	8	50.7	6	10
1969	139	6	29	170	6	29
1970	92	6	14	112	6	14
1971	89.2	6	6	116	6	6
1972	49.8	6	1	56.1	6	1

Year	Peak Daily Discharge (m^3/s)	Date		Peak Instantaneous Discharge (m^3/s)	Date	
		Month	Day		Month	Day
1973	43.3	5	26			
	42.2	6	7	47	6	7
1974	66	6	17	78.4	6	17
1975	49.3	6	20	53.5	6	20
1976	36	8	6	42.5	8	6
1977	15.8	8	14	17.1	8	13
1978	44.5	6	6	49.8	6	6
1979	32.1	5	27	38.4	5	27
1980	51.7	6	3	69.3	6	4
1981	98.2	5	26	123	5	26
1982	28.9	6	16	30.7	6	14
1983	18.6	4	25	31.5	4	25
	26.2	5	30			
1984	19.4	6	12	21.1	6	12
1985	61.2	9	13	79.7	9	13
1986	48.9	5	28	57.2	5	28
1987	24.3	7	19	26.5	7	19
1988	28.9	6	8	37.3	6	8
1989	20.4	6	9	23.1	6	9
1990	129	5	26	172	5	26
1991	41.4	5	21			
	41	6	22	47.2	6	21
1992	88.7	6	15	119	6	15
1993	80.4	6	16	*108.5*	*6*	*16*
1994	52.1	6	7	72	6	7
1995	190	6	7	377	6	6
1996	43.5	6	8	48.4	6	9
1997	47.8	5	31	54.2	6	1
1998	103	5	28	141	5	28
1999	48.3	7	15	53.7	7	15
2000	14.4	6	10	15	6	10
2001	39.4	6	5	45.2	6	4
2002	70.2	6	16	87.3	6	17
2003	29.1	4	25	61.5	4	25
	30.9	5	25			
2004	31.4	8	26	31.9	8	26
2005	231	6	7	308	6	7
2006	75.1	6	16	97.9	6	15
2007	54	6	7	83.7	6	6
2008	125	5	24	204	5	24
2009	39.4	7	14	51.2	7	13
2010	43.3	6	18	48.4	6	17
2011	95.5	5	27	112	5	27

Year	Peak Daily Discharge (m^3/s)	Date		Peak Instantaneous Discharge (m^3/s)	Date	
		Month	Day		Month	Day
2012	79.8	6	6	110	6	6
	83.2	6	24			
2013	478	6	20	874	6	20
2014	88.2	6	20	103	6	20
2015	25	5	28	26	5	28

Notes:
1. Italicized values of instantaneous discharges were estimated from daily values using the historical relationship between coincident annual instantaneous and daily peaks.
2. Years for which the annual maximum discharge was a summer storm are highlighted.
3. For years when the annual daily peak and the annual instantaneous peak represent different peak events, the daily peak coincident with the instantaneous peak was added to the data set.
4. The 1974 WSC tabulated peak instantaneous value was found to be in error and was corrected using the detailed annual discharge data.