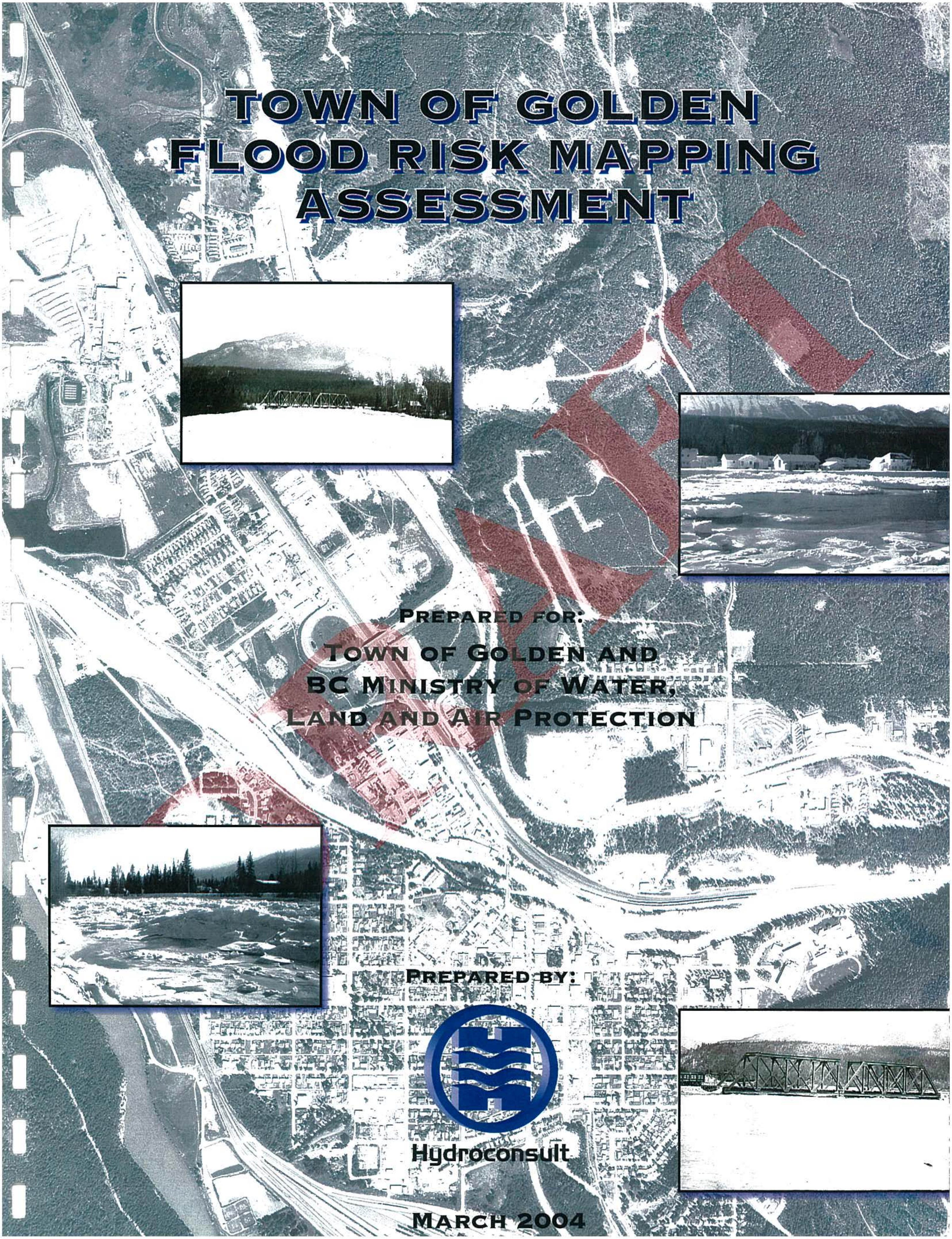
TOWN OF GOLDEN FLOOD RISK MAPPING ASSESSMENT



PREPARED FOR:

TOWN OF GOLDEN AND
BC MINISTRY OF WATER, LAND AND AIR PROTECTION

PREPARED BY:

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MARCH 2004

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Ministry of Water, Land and Air Protection, and
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Dear Sirs:

SUBJECT: Town of Golden, Kicking Horse River Flood Risk Assessment Draft Report

Hydroconsult is pleased to herein provide each of you with one copy of the above named draft report for your review. We have not prepared a flood risk map (Appendix C) as you will see based upon our conclusions and recommendations. After you each have had an opportunity to review this draft, I would suggest a conference call to discuss our results and the final report recommendations/edits etc.

I will call each of you in one to two weeks time to set an appropriate time for a conference call. In the meantime, please contact either Wim Veldman or myself, if you have any questions or require any additional information.

Yours truly,

HYDROCONSULT EN3 Services Ltd.

D. L. Cooper

Dave Cooper, P.Eng.
Vice President / Senior Project Manager

Small in Focus
Global in Reach

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ACKNOWLEDGEMENTS

Hydroconsult wishes to acknowledge Mr. Ron Buss, P. Eng., Manager of Operations, Town of Golden and Mr. Dwain Boyer, P. Eng., Senior Floodplain Officer, Ministry of Water, Land and Air Protection for providing valuable review and supporting information for this report. Mr. Buss collected the historic air photos in Appendix A, provided the Town survey and base mapping data, identified other information sources and described historic events concerning flooding and development along the Kicking Horse River.

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EXECUTIVE SUMMARY

Purpose

The Town of Golden, with funding and technical support provided by the British Columbia Ministry of Water, Land and Air Protection (MWLAP), commissioned this study to review and refine the flood risk mapping currently in place in the Town of Golden. The flood risk mapping and development restrictions currently applied are based on a previous study in 1979 that did not consider the protection provided by the existing dike system.

The present assessment is limited to the area on the south side (left bank as viewed looking downstream) of the Kicking Horse River where current development restrictions apply based on the 1979 flood risk map. Namely "*the flood construction level be three (3) feet above the level of the crown of the adjacent municipal road at the point nearest the building*". The combined effect of flooding from the Columbia River is incorporated into the study by assuming coincident flood peaks – potential overland flooding is primarily from the Kicking Horse River with some local backwater effect created by the Columbia River in the lower portion of the Town.

Scope and Methodology

The study consists of a review and analysis of the 200-year return period peak discharge value – the event that is applied for flood-risk mapping in the province under the Canada-British Columbia Flood Damage Reduction Program. A visual inspection and assessment of the current dike system including river and potential overland flow conditions and a top of dike survey were conducted. An updated 1 m contour interval digital map of the Town was prepared. Updated hydraulic modeling of the Kicking Horse River was conducted to update and compare with the results from a 1999 study and to evaluate more extreme case scenarios to assist in assessing the risk of potential dike overtopping and/or breaching. Overland flooding scenarios were evaluated and potential flood depths were estimated based upon assumed dike overtopping and breaching scenarios. These evaluations were used to assist with developing a flood risk mapping guideline for the south (left) side of the river within the Town of Golden.

Summary of Results

The 200-year return period peak discharge is estimated at 570 m^3/s. However, there is significant potential for variation in this estimate. An extreme upper envelope discharge is 1,000 m^3/s. During a flood the Kicking Horse River's discharge may be expected to stay near to 98% of the peak value for 3.5 hours and up to 95% of the peak value for as long as 10 hours.

Of the modes of potential dike failure evaluated, dike overtopping and subsequent breaching is expected to be the most probable mechanism causing overland flooding. Other modes such as direct erosion of the riverbank or geotechnical seepage or slip failures are much less probable.

River hydraulic modeling shows that the differences in water levels predicted between surveyed sections in 1997 and those in 2002 are minor and less than 10 cm at the 200-year flood level – indicating that any sedimentation impacts over this period have been minor. The existing south (left) dike freeboard at the computed 200-year flood level is greater than MWLAP's recommended minimum guideline of 0.6 m in all but three short locations where it may drop to as low as 0.5 m. These locations are at:

- a 140 m segment at the campground at the upstream end of Town;
- a short segment at the sub-channel downstream of Highway 95 (final grading and top surfacing following dike upgrading work in the fall of 2003 is expected to raise the dike grade to above the minimum guideline here); and

- at a couple of segments upstream of the CPR bridge where the impact from overtopping is minor because flood risk levels from the Columbia River are more significant.

A few similar low sections located along the right dike level are also identified.

Assumed extreme case scenario evaluations show the following:

- The left dike may not overtop at a higher flood peak discharge of 660 m^3/s (in the order of a 500-year return period event). The average increase in the 200-year flood level is 0.26 m at this higher discharge. In fact, overtopping may not occur until a discharge of at least 740 m^3/s, assuming that the model inputs and assumptions are correct at these high discharges.

- Maximum increases in assumed channel hydraulic roughness values that may account for extreme debris, sediment and turbulent flow conditions increase the predicted flood level by an average of 1.16 m. The maximum roughness values assumed in this scenario are considered extreme and unlikely. However, under this assumption, significant overtopping in the low areas along both the left and right dikes would occur.

- A debris jam that fully blocks the sub-channel at the Highway 95 bridge does not dramatically increase flood levels and on its own is not expected to result in overtopping of the left dike.

- Sediment deposition amounting to 1 m deep across the entire river bed in the reach downstream of Gould's Island increases the flood level by up to 1 m such that overtopping of the left dike may just begin to develop in the reach downstream of the island.

The above are all extreme cases. Some combination of the above conditions (sediment deposition, debris and increased roughness and a flood peak exceeding 570 m^3/s) may be required to cause overtopping of the dike. Ice conditions are not expected to pose as great a risk as an open water flood event.

Analysis of potential dike breaching as a result of overtopping indicates that the campground area has the greatest risk for a breach and a significant breakout flow. A breach 30 m wide by 1.2 m deep may result in a breakout flow of up to 45 m^3/s. Overland routing of this flow down the streets indicates that a flow depth of approximately 0.9 m may develop along the upper end of 9th Street. This depth of flow reduces as it spreads out downstream and may have an average depth of 0.3 m at Highway 95 with greater ponding in local low points. For such an event to occur, the following will have to occur:

- a water level must occur that is at least 0.7 m higher than the estimated 200-year flood level that is computed based on a discharge of 570 m^3/s;

- this water level must be sustained for a period of at least 2 hours;

- no remedial protection measures are taken during the event;

- erosion occurs on the downstream side of the dike leading to a significant breach; and

- the water level in the river would need to remain at or near the top of dike level for a further period of at least 2 hours for adequate water to break out and create significant downstream flooding.

Overtopping flows without dike breaching may be in the order of 5 to 15 m^3/s. Downstream overland flow depths are estimated to be in the order of 0.1 to 0.2 m under these breakout flows.

Conclusion
The extreme case scenarios show that some risk of flooding still exists on the left side of the Kicking Horse River within the area protected by the dike system. Although these risks are considered to be low, property owners are to be aware of the potential risks here.

The dike system is appropriately armoured and stable, with adequate freeboard in accordance with provincial guidelines (at least 0.6 m above the designated 200-year flood level equivalent to a peak discharge of 570 m^3/s). Therefore, no flood risk development restriction guidelines are recommended to be applied to the area protected by this dike system, provided that appropriate ongoing dike monitoring and maintenance is conducted. This includes:

- locally raising the left dike by about 0.1 m in the few identified low locations to meet the 0.6 m minimum freeboard criteria,

- annually inspecting the dike sideslopes, the protective riprap and the dike crest,

- replacing /stabilizing the riprap, as required, and

- continuing to conduct and evaluate river cross-section surveys at least every two or three years to assess sedimentation conditions.

In addition, an Emergency Preparedness Plan (EPP) should be in place, updated and regularly tested, as required.

1.0 INTRODUCTION

1.1 BACKGROUND

The Town of Golden is situated on the alluvial delta of the Kicking Horse River, as shown in Figure 1. The river exits from a narrow canyon at the upstream east side of the Town and flows into the Columbia River at the downstream west side of the Town. An armoured dike system along the Kicking Horse River has been constructed and upgraded at various times for flood protection purposes. This dike system combined with the existing road and rail embankments along the Columbia River act as flood protection for the Town.

A free active channel on an alluvial delta tends to continually shift and cut new channels. During high flows it flattens out by building up and depositing material over its fan. The development of the Town and dike system has restricted and modified this natural alluvial process on the Kicking Horse River. However, deposition in the restricted channel portion remains an on-going potential concern to future flooding. The growth of gravel bars in the lower reaches of the river reduces the flow capacity of the channel and decreases the level of protection provided by the dikes. Historically, the Town maintained the channel capacity by periodically removing gravel from the channel bars at various locations. These bars were last scalped at five locations in 1997. Due to potential aquatic impacts and concerns raised in recent years, justification for further excavation/scalping work now needs to be supported by hydraulic modeling and environmental assessments.

Several previous hydraulic studies have been conducted on the Kicking Horse and Columbia Rivers in Golden:

- In 1979, a Flood Risk Mapping study was conducted by the BC Ministry of Environment for the Town of Golden. Floodplains were delineated for the 20- and 200-year floods using the U.S. Army Corps of Engineer's HEC-2 hydraulic model with river survey data from 1975. According to the mapping, the majority of the town lies within the 20-year and 200-year floodplains, assuming no dikes are in place. The study included both the Columbia and Kicking Horse Rivers.

- In 1989 the BC Ministry of Environment and Parks conducted an assessment of the flood protection dikes using the HEC-2 model and additional data surveyed in 1987. However, the flood risk mapping for the Town was not updated.

- In 1999, in response to the need to justify further channel excavations, Hydroconsult prepared the report "*Hydraulic Modeling of the Kicking Horse River to Determine Channel Capacity*" for the Town of Golden. The report assessed the channel's capacity to pass the 200-year flood event and assessed alternatives to ensure that the flood protection dikes would not be overtopped. The alternatives consisted of either increasing the dike heights in low areas or excavating out bar deposition areas to increase the channel capacity or both.

- Since 1997, the Town of Golden has repeated surveys of several sections of the river once or twice a year. Hydroconsult recently reviewed the degree of deposition based on the historical cross-section surveys in the report "*Assessment of Sedimentation on the Kicking Horse River, Town of Golden*" February, 2003.

The floodplain mapping currently applicable to the Town under the Canada-BC Flood Damage Reduction Program is based upon the study and mapping prepared in the original February 1979 report. This mapping assumes no dikes are in place and places a large portion of the left overbank area of the Kicking Horse River within the Town of Golden under a building restriction. The restriction states "*that the flood construction level be three (3) feet above the level of the crown of the adjacent municipal road at the point nearest the building*". A review and updating of this mapping is warranted because of the dike system upgrades undertaken by the Town, particularly recently from 1996 to present as summarized in Figure 2, and in view of the more recent survey and hydraulic modeling studies conducted.

1.2 OBJECTIVES

The Town of Golden, with funding and technical support provided by the BC Ministry of Water, Land and Air Protection (MWLAP), retained Hydroconsult EN3 Services Limited (Hydroconsult) to:

- Review and assess the risk of dike overtopping and/or breaching and the expected extent of downstream flooding that could develop based upon the current dike conditions and the hydraulic capacity of the river considering factors such as debris, ice and sedimentation.

- Review and, where appropriate, refine the current (1979) flood risk mapping for the Town of Golden and propose supporting development restriction guidelines, if necessary.

This assessment is limited to the south side (left bank as viewed looking downstream) of the Kicking Horse River because the current flood risk mapping and development restrictions only apply in this area of the Town. The assessment considers the combined effect of flooding from the Columbia River by assuming coincident flood peaks.

1.3 STUDY BASIS

Hydroconsult's analyses and assessment for this Flood Risk Mapping in Golden are based on the following:

- site visits and site photographs by Hydroconsult staff (Dave Cooper, P.Eng. and Greg Standen, E.I.T.) in July and September 2003 and numerous previous visits for prior work by Hydroconsult,
- discussions and review meetings with Ron Buss, P.Eng., Manager of Operations for the Town of Golden and Dwain Boyer, P. Eng. (MWLAP),
- historical river cross-section data from 1987 to 2002 provided by BC Ministry of Environment, Land & Parks (MELP) and the Town of Golden
- September 2003 top of dike and spot check surveys conducted by Alpine Surveys Ltd. for this study,
- historical Water Survey of Canada (WSC) streamflow data for the Kicking Horse River at Golden (Station 08NA006) the Columbia River at Nicholson (Station 08NA002) and other regional stations, as published by WSC,
- stage-discharge rating curve and historic ice data for the Kicking Horse River at the Golden station, as provided by WSC,

- historical spot water level data in the Kicking Horse River at the Highway 95 bridge recorded from 1991 to 1997, as provided by the Town,
- historical airphotos from 1953 and 1996 and river photos;
- bridge plans and design drawings for the Highway 95 bridge provided by BC Ministry of Transportation and Highways, and for the CPR bridge provided by Canadian Pacific Limited,
- the floodplain mapping prepared by the BC Ministry of Environment in February 1979,
- base mapping from the 1990 BC Ministry of Transportation "Golden to Yoho National Park Corridor Study" mapping at 1:5000 scale with 5 and 10 m contours and spot elevations,
- Town of Golden street plan and profile as-built drawings and 1987 downtown revitalization drawings, and
- review of previous hydrologic and hydraulic study reports for the Town of Golden and region.

2.0 APPROACH

A review and updating of the current flood risk mapping for the Town of Golden requires more than simply a re-mapping of the flood limits with the dike system in place. An understanding of the dynamic river conditions and the related potential risks of dike breaching or overtopping as well as the potential duration of a flood event is required to appropriately assess the potential flood risks in the developed areas of the Town.

The original (1979) flood risk mapping is based upon no protective dike system in place. With the dike system in place the development guideline that *the flood construction level be three (3) feet above the level of the crown of the adjacent municipal road* may be excessive. To evaluate this, the following questions may be asked:

What happens if the dike overtops or is breached?
What is the risk of this occurring?

The following study components presented in the subsequent chapters in this report are applied to address these questions:

1. Hydrologic Review and Analysis: The 200-year return period peak discharge value is applied for flood risk mapping in the province of BC under the Canada-BC Flood Damage Reduction Program. A review of the 200-year peak flood value is conducted based upon current data, previous study results and regional data to assess the magnitude of the 200-year flood event and identify the risk of greater events. The probable shape(s) of the 200-year event flood hydrograph is also important to identify the potential magnitude and duration of any breakout or overtopping flow. An assessment of the probable duration of a specific discharge being equaled or exceeded provides input to subsequent hydraulic analyses and the volumes of water that can break out.

2. Surveys and Base Mapping: An updated 1 m contour interval map is developed for the Town. This mapping includes spot elevation data checks in the Town and a re-survey of the top of dike level. The mapping, contours and elevation data are provided as digital layers on an AutoCAD drawing with the 1996 orthophoto as one layer. The mapping is not spatially corrected but provides adequate detail and accuracy for hydraulic analysis and flood risk mapping purposes.

3. Dike Assessment and Failure Mode Evaluation: An assessment of the existing dike system is presented considering the dike side-slopes and stability, erosion protection material in place, top of dike conditions and width, land-side conditions and slopes and local river flow conditions. An evaluation of potential dike failure modes is discussed. The higher risk locations for potential failure are then identified based upon the dike system assessment. These locations are evaluated in subsequent hydraulic analyses.

4. River Hydraulic Analyses: A re-analysis of the hydraulic modeling of the river channel is conducted using the most recent channel cross-section surveys dated November 2002. This re-analysis provides a comparison from Hydroconsult's 1999 study to more precisely assess the probable impact of recent sedimentation. Other more extreme scenarios are investigated to evaluate the risk of dike overtopping. These include assessing a much greater than the 200-year flood event, applying maximum feasible increases in channel roughness and severe debris jam conditions.

5. Overbank Hydraulic Analyses: A series of scenarios and sites are evaluated to identify the magnitude and scope of potential dike breaches and overflows. These scenarios consider: site-specific dike conditions/levels, overland conditions/grades and probable directions of flow, potential flood level with corresponding duration required at that level for breaching or a breakout flow, and dike breach mechanisms/size and timing. Hydraulic models are used as tools to conduct backwater calculations, evaluate hydrograph attenuation effects, define probable flood levels and related risks of the various scenarios and identify the most probable flow routes according to defined breakout scenarios. Various assumptions are tested to identify sensitivities and develop flood level estimates and associated risks.

6. Flood Risk Mapping: The above components are considered together to develop a flood risk mapping guideline for the left (south) side of the Town.

3.0 HYDROLOGIC ANALYSIS

3.1 SCOPE

A hydrologic review of the results from the 1999 Hydroconsult study was conducted to:

- assess / confirm the magnitude of the 200-year flood event and the level of confidence in the value, and
- define the probable shape(s) of the 200-year event flood hydrograph to assist in defining the length of time potential overtopping may occur.

This review included assessing rainfall/snowmelt runoff producing events to further assess the probable duration and magnitude of flood events.

3.2 PREVIOUS STUDIES

Estimates of the 200-year peak discharge on the Kicking Horse River at Golden have ranged from 475 m^3/s to over 750 m^3/s based on previous site specific and various regional studies as follows:

- The original 1979 floodplain mapping was based on a 200-year peak discharge of 538 m^3/s (basis is not known).

- From a previous regional study (B. C. Environment, 1987), the Interior Mountains Region Envelope Curve of extreme floods estimates an extreme (no return period) maximum daily discharge of 970 m^3/s for the Kicking Horse River.

- The dike assessment by the Ministry of Environment and Parks in 1987 used a peak discharge of 645 m^3/s in the Kicking Horse River and 800 m^3/s in the Columbia River (basis is not known).

- A regional flood study conducted for the Elk River valley (Hydrocon, 1982), that included the Kicking Horse River basin, predicted a maximum daily discharge of 584 m^3/s on the Kicking Horse River. The single station frequency analysis of the Kicking Horse River data at that time estimated a maximum daily discharge of 534 m^3/s (the same as the 1999 study indicated above).

- A preliminary report prepared in 1996 (Ward & Associates Ltd.) estimated a 200-year peak discharge of "about 500 m^3/s" based on a frequency analysis of data up to 1993 on the Kicking Horse River at Golden.

- Hydroconsult's 1999 study estimated the 200-year maximum instantaneous discharge at 570 m^3/s. This was based upon a frequency analysis of 34 years of maximum daily data on the Kicking Horse River at Golden (data up to 1997) and multiplying the maximum daily value by the average instantaneous/daily ratio for the three largest recorded flood events. A value of 782 m^3/s was determined for the Columbia River at Golden (91 years of record) in a similar manner. The hydraulic analyses conducted in the 1999 study assumed coincident flood peaks at Golden on the Kicking Horse River and the Columbia River.

- Another regional study (not as comprehensive as the other regional studies) by Hydroconsult (1999a) developed for the Elk River valley region would estimate a 200-year maximum daily discharge of 757 m^3/s on the Kicking Horse River.

- The Kicking Horse River is in hydrologic subzone 14x in the East Kootenay Region, as defined in a recently completed regional streamflow study (B. C. Ministry of Sustainable Resource Management, 2002). The 10-year peak flow is 320 m^3/s applying the envelope curve developed for this subzone. The resulting 200-year peak is computed as 475 m^3/s applying the recurrence interval ratio in the frequency relationship in this study.

The regional studies noted above show that the frequency analysis results based upon the Kicking Horse River station data alone predict lower extreme flood values than those indicated by other regional streams. This suggests that extreme flood events may have simply missed this basin by chance or they have not been recorded on the Kicking Horse River or the Kicking Horse River flood peaks are lower. Regardless, relying upon a flood frequency analysis of data from a single station requires significant extrapolation to predict a 200-year flood event and the confidence limits are very wide.

3.3 REVIEW OF HISTORIC FLOOD EVENTS

Up to 2002 the WSC station 08NA006 - Kicking Horse River at Golden, with a drainage area of 1850 km^2, has 39 years of annual maximum daily discharge data and 17 years of maximum instantaneous values from 1912 to 2002, as shown in Figure 3. A review of the data indicate:

- Annual maximum daily peaks have occurred between May 13 and August 25 with 7 events occurring in May, 25 in June, 6 in July and 1in August. The highest events occurred in June.

- The ratio of maximum instantaneous to daily flow ranges from 1.041 for the average of the three highest events (This ratio is typically applied to extrapolate daily data to estimate extreme return period events.) to 1.064 as an overall average ratio and up to 1.158 as the highest ratio. The highest ratio occurred during one of the lowest peak events in 1987.

Annual maximum flows on the Kicking Horse River primarily occur due to snowmelt. However, the highest peak events are a result of rainfall, with rainfall on snowmelt causing the most extreme events. A detailed review of historic flood events is complicated by the size of this basin, the variable physiographic conditions and the sparse climatic station network within the basin.

The hydrographs of the seven largest historic flood events on record are plotted in Figure 3 (1916, 1918, 1920, 1974, 1986, 1988 and 1999). A graphical plotting method was used in this figure to convert the daily data to 3 hour data and thereby estimate instantaneous peaks. The baseflow at the start of these events averaged 78 m^3/s. For runoff comparison purposes, the runoff depths for the events were all computed by subtracting an assumed average baseflow of 100 m^3/s over the duration of the events. A brief summary of the available climatic data and runoff conditions from these events follows.

<u>June 19, 1916 Event (450 m^3/s)</u>– No climatic data are readily available from posted sources to describe conditions leading up to or during this event. It is the largest event on record at Golden with an estimated peak of 450 m^3/s and it has the shortest duration at about 6 days. This suggests that it was primarily due to rainfall with a snowmelt baseflow rate of about 100 m^3/s. The runoff

depth over this 6 day event was 46 mm not counting the baseflow. Appendix A, Photo A1 is a post card photo of this flood event.

June 14, 1918 Event (410 m³/s) – This is the second highest event recorded. It was a double peak event with 22 mm of rain recorded in Field the day prior to the second higher peak. Total precipitation for the preceding six months (January to June peak) was 266 mm. Runoff above the baseflow was 57 mm over 9 days.

July 2, 1920 Event (352 m³/s) - No climatic data were available for review. The runoff depth above the baseflow was estimated at 39 mm over 6 days.

June, 1974 Event (322 m³/s) – This was a long duration snowmelt event extending over 18 days and peaking on June 23. The recorded snow pillow data at Kicking Horse Pass indicated the water equivalent depth was 432 mm on April 1 and still at 221 mm by May 31. Total recorded precipitation at the Boulder Creek station for the preceding six months was 290 mm including 11 mm rainfall over the three days prior to the peak. Runoff depth above the baseflow was 124 mm over 20 days.

May 30, 1986 Event (367 m³/s) – This was an early snowmelt peak event. The snow pillow water equivalent was 349 mm on both April 1 and May 1 at the Pass. No rainfall was recorded at the Boulder Creek station for the 7 days prior to the peak. Total recorded precipitation at Boulder Creek from January to May was 260 mm. Runoff above the baseflow was 86 mm over 10 days.

June 8, 1988 Event (351 m³/s) – This was clearly a rainfall event with 98 mm rain recorded over the previous 12 days including 32 mm in the 2 days prior to the peak at the Boulder Creek station. The total recorded precipitation from January to the peak in June was 296 mm. The recorded snow pillow data at Kicking Horse Pass indicated the water equivalent depth dropped from 274 mm on April 1 to 0 mm by May 1. Runoff above the baseflow was 36 mm over 6 days.

June 19, 1999 Event (316 m³/s) – This was a slow rising extended snowmelt peak event. Total recorded precipitation from January to the peak in June was the highest of the events reviewed at 461 mm, not counting missing data for entire the month of May. The recorded snow pillow data at Kicking Horse Pass indicated the water equivalent depth was 394 mm on April 1 and still at 354 mm by May 15. Only 3 mm rainfall was recorded on the day prior to the peak at the Emerald Lake station. Runoff above the baseflow was 46.5 mm over a period of 7 days.

The time to peak for the above events ranged from 2.5 days for the rainfall event in 1988 to over 12 days for the long duration snowmelt in 1974. Runoff depth (above the baseflow) ranged from 36 mm in 6 days to 124 mm in 20 days and the average daily runoff rate was fairly consistent ranging from 6 to 8.6 mm/day. The above flood events were primarily due to snowmelt with no extreme rainfall amounts occurring compared to those recorded at other stations in the East Kootenay Region A major storm event, greater than the 1988 event and in the order of 100 mm in a day, combined with one of the other snowmelt events is expected to produce the more extreme 200-year type of flood event.

3.4 200-YEAR FLOOD PEAK ESTIMATE

3.4.1 Single Station Flood Frequency Analysis

A flood-frequency analysis of the maximum daily data is presented in Figure 3 applying the U.S. flood frequency analysis procedure (USACE, 1992). The expected probability curve in this frequency analysis procedure estimates the 200-year maximum daily discharge at a comparatively low value of 474 m^3/s. Other frequency distribution analysis procedures (Environment Canada, 1993) result in similar to slightly lower estimates. Applying the average maximum instantaneous to daily ratio results in a maximum instantaneous discharge estimate of 504 m^3/s. The frequency analysis in Figure 3 shows the upper 95% confidence limit at the 200-year return period is a maximum daily flow of 536 m^3/s – this value is nearly identical to the previous frequency analysis results conducted using less available data in 1984 and 1999.

3.4.2 Two-Station Comparison Frequency Analysis

The above results show the range of values possible when relying upon a statistical single station frequency analysis of observed flows having a moderately short period of record - even considering the moderately good fit of the data to the frequency distribution. A problem with the statistical frequency analysis procedure for this station is that the annual events are the result of a mix of different hydrologic events (e.g. rain, snowmelt, ice effected or combinations), they may or may not be representative of extreme events, and the lower events can significantly affect the distribution fit and prediction of the more extreme events.

As noted earlier, the regional flood studies indicate that the Kicking Horse River flood frequency results are low compared to other station data in the same region. Reasons for this may be simply due to: chance that major floods have physically missed the basin, differences in rainfall runoff characteristics (e.g. physical rain-shadow effects), or the period of record did not cover major events (e.g. the highest flood of record on the Columbia River station was in 1972 when the Kicking Horse River station was not in operation). An analysis to extend the Kicking Horse River period of record based upon correlation with the Columbia River station and other regional stations was therefore conducted. The two-station comparison (USWRC, 1981) analysis with the Columbia River station provided the best results with a correlation coefficient (R^2) value of 0.87 and an extension of the period of record from 39 years to a 50 year systematic period of record. Applying a regionally computed skew coefficient of 0.32, the resulting 200-year maximum daily discharge estimate is 538 m^3/s – or nearly identical to the upper 95% confidence limit for the single station data alone.

3.4.3 Runoff Depth Approach

A runoff depth approach, as developed by Alberta Transportation for use in Alberta, was used to assess the magnitude of flood peaks that may be expected to occur based upon maximum runoff depths historically observed in the region. The regional data indicates that a 40 to 60 mm runoff depth may be an extreme maximum event in the order of a 200-year event. The estimated time to peak with this approach ranges from 40 to 60 hours (much shorter than the observed historic events). Sub-dividing the watershed into six sub-basins and routing of the 60 mm maximum runoff depth over the sub-basins and along the 90 km of main river channel results in a peak discharge estimate of 595 m^3/s assuming a time to peak of 50 hours. Sensitivity analyses conducted by varying the runoff depth from 40 mm to 60 mm and the time to peak from 40 hours to 60 hours result in peak discharge estimates from 395 m^3/s to 740 m^3/s. These are all peak events without baseflow. The baseflow, based on the historic flood events, may add another 75 to 100 m^3/s to the above peaks.

3.4.4 Recommended 200-Year Flood Peak

Other variables over time can significantly increase flood peaks. These include climate change impacts and the dramatic loss of tree cover in a basin due to major forest fires, as has recently occurred in large portions of the province. Recognizing the potential impact of these variables, caution should be applied when predicting extreme flood events. For this reason a reasonably conservative value should be applied as a standard base value. However, the impact of much higher but possible events should also be assessed.

Based upon the above review, the recommended base 200-year flood peak value is 570 m^3/s, as previously used in the 1999 study. This value is consistent when compared with the above results. The value is equal to a maximum daily discharge of 536 m^3/s times the average maximum instantaneous to daily ratio of 1.064. This maximum daily value is equal to the upper 95% confidence limit of the frequency analysis shown in Figure 3 and comparable to the two-station comparison analysis results using the Columbia River records to extend the period of record.

Because much higher peak events are identified as possible, ranging from 740 m^3/s to a regional extreme envelope value approaching 1,000 m^3/s, an upper limit discharge of 1,000 m^3/s is presented in the hydraulic analyses in the following sections for evaluation/illustration purposes.

3.5 FLOOD EVENT DURATION

The shape of the flood hydrograph is important to identify the potential magnitude and duration of any breakout or overtopping flow. The historic flood events, shown in Figure 3, give an indication of the possible hydrograph shapes and probable duration of a specific discharge being equaled or exceeded.

An average dimensionless hydrograph shape was developed by converting the historic hydrographs to dimensionless hydrographs by dividing the discharge at any time by the peak discharge. Based on the historic hydrographs shown in Figure 3, the average hydrograph duration is about 9 days, with an average time to peak of 5.5 days.

The average dimensionless hydrograph can be scaled up to any peak event accordingly. With 570 m^3/s as the 200-year peak value, the flood event hydrograph indicates that the discharge may:

- exceed 560 m^3/s for about 3.5 hours,
- exceed 540 m^3/s for about 10 hours, and
- exceed 500 m^3/s for nearly 24 hours.

The historic hydrographs suggest that much longer duration events of over 10 days may be possible. However, these longer events are typically due to snowmelt and have lower peaks and would not lead to overtopping. The shorter duration events that are a combination of rainfall and snowmelt are the more extreme type of event that may be expected to produce discharges exceeding 500 m^3/s. These are expected to be of shorter duration than the average hydrograph shape presented here.

4.0 FLOODPLAIN AND DIKE MAPPING

4.1 MAPPING AND SURVEYS

The available survey and mapping data used to assist in the preparation of a base map for the flood risk mapping and evaluation included:

- The 1979 BC MELP Floodplain Mapping at a scale of 1:5,000 with a 1 meter contour interval. The indicated accuracy of spot elevations is ±0.3 m. The mapping is based on a September 1975 hard copy orthophoto map.

- Digital 1990 BC MOT "Golden to Yoho National Park Corridor Study" mapping at 1:5,000 scale with 5 and 10 m contours and spot elevations shown on the Town streets at all intersections and at intermediate points (spacing at 100 to 170 m).

- Available street plan and profile drawings (e.g. 9^{th} Street South 1990-91 as-builts) and 1987 downtown revitalization drawings.

- Top of dike and river cross-section survey data from 1997 to 2002 compiled by Hydroconsult for the Town.

- September 2003 top of dike and spot check surveys conducted by Alpine Surveys Ltd. specifically for this study.

Figure 4 shows the resulting digital 1 m contour interval base map for the Town. This figure includes in layers: the detailed top of dike and spot elevation data in key areas for hydraulic assessment purposes, the 1996 air photo, the street plan from the 1990 BC MOT map, 1m contours, and spot elevation data throughout the Town.

5.0 DIKE ASSESSMENT AND FAILURE MODE EVALUATION

5.1 LEFT DIKE CONDITIONS

A visual inspection of the left (south) bank dike system, the land and river sideslopes and potential overland flow drainage paths was conducted by Hydroconsult on July 31, 2003. Figures 5 and 6 show photos taken along the dike and areas of potential overland flow. Historic site photos of flood and ice event conditions are provided in Appendix A. A re-survey of the dike crest was conducted in September 2003 by Alpine Surveys Ltd.

A summary of observations from the inspection is provided in Table 1.

Based upon the site inspection and hydraulic conditions along the river, the following four dike reaches are identified as key areas to evaluate for dike breaching:

1. At the west end of 9th Street South at the Campground (Photo 1, Figure 5) near the upstream end of Town. Any overflow will run down 9th Street South.
2. At the Skateboard Park upstream of Highway 95 (see Photos 3 and 4, Figure 5) with the flow running south along 10th and 11th Ave S as well as west down Park Drive.
3. Off the sub-channel (see Photos 5 and 6, Figure 6) where the greatest dike height above natural ground occurs (1.8 m) within the main developed portion of Town.
4. Downstream of section K7 (Photo 8, Figure 6) where the dike height is 3.4 m above natural ground – presently building development downstream of this section is minor consisting of the airport facilities and the road adjacent to the dike.

5.2 DIKE FAILURE MODES

A discussion of the potential failure mechanisms and associated relative risks of these types of failures occurring along the dike system follows.

5.2.1 Bank Erosion

The existing riprap material along the dike system is considered to be of adequate gradation, quality and thickness and at a sufficiently stable slope as to be stable under 200-year flood conditions. However, failure of the riprap slope protection could occur as a result of ice or debris impacts and/or local high velocities causing undermining scour and/or mobilization of the dike slope riprap material. These conditions are not expected for the design flood. It is possible that a gradual loss of riprap over an extended period of time combined with inadequate follow-up maintenance may be another reason for this form of failure.

Any bank erosion failure will be a visible event that may be expected to develop over a period of several hours before any dramatic dike breaching may occur. Emergency remedial measures may minimize the risk of a dramatic breach due to this form of failure. Due to the relatively short duration of the extreme flood events, the peak may have passed by the time a serious breach could develop as a result of erosion. Provided appropriate monitoring and maintenance is conducted along the dike system, the risk of a serious failure leading to a complete breach due to bank erosion is considered to be minor.

Table 1: Summary of Left (South) Dike Observations – July 31, 2003

Location	Dyke Crest Width (m)	Crest Level above Natural Ground (m)	River Sideslope (%)	Land Sideslope (%)	Overland Downstream Slope (%)	Riverside Comments	Landside Comments
Campground, near K52	5	1.25	50	30	1	Stable riprap slope/straight river reach	Open grassed area with mature trees draining down to 9th St S. Old sub-channel along valley wall densely overgrown now.
150 m downstream of K52	4	1.4	55	25	1	Slightly steep riprap slope/straight river reach.	Open grassed area with mature trees draining down to 9th St S.
West end of 9th St S (Photo 1, Figure 2)	4	1.6	50	20	0.7	Stable riprap slope/straight river reach	Open grassed and paved area, draining to 9th St. S.
K2	4	1.33	50	20	0.7	On the inside bend of river.	Open grassed and paved area with some gravel, draining to 9th St. S.
50 m downstream of K2	4	1.35	50	20	-0.9	Stable riprap slope with shrubs growing on the top of the dyke. Inside bend of river where velocities are low as indicated by the gravel bar near the toe of the riprap slope.	Open grassed and paved area, draining to 9th St. S. Local rise in 9th St. S would create ponding here.
College of the Rockies (Photo 2, Figure 2)	4	0.6	50	10	0.26	Outside bend of river. Stable riprap slope. Channel splits around a small island.	Open grassed/ paved area, drains to 9th St. S. through College lot. Low breach potential due to low dyke above natural ground.
K51	4	0.9	60	15	0.3	Oversteep riprap slope with juvenile trees. High potential for erosion on nose of bend in river	Open grassed area with shrubs, low potential for breach due to low dyke above natural ground.
K4, Skateboard park area (Photo 3 Figure 2)	3	0.96	55	30	0.3	Slightly steep but stable riprap slope. Straight reach upstream of Highway 95 bridge.	Open paved area. Narrow dyke. Shrubs on the dyke. Draining to 10th and 11th Ave. S and Park Dr.
Sub-channel downstream of Highway 95 bridge (Photos 5 and 6 Figure 3)	3	1.8	65	62	0.6	Oversteep riprap slope located on the outside of a bend in the sub-channel. Upgrading planned for October 2003.	Open grassed area. Narrow dyke, Mature trees and shrubs on dyke. Maximum dyke height in developed portion of Town
Near K6 (Photo 7, Figure 3)	6	1.7	50	40	0.2	Stable riprap slope with shrubs/outside bend of river	Residential area with shrubs and mature trees. Minor local depression
K7 (Photo 8, Figure 3)	6	3.4	55	55	0.2	Slightly steep but stable riprap slope covered by deposition from river. Straight river reach.	Maximum dyke height above natural ground here.

5.2.2 Geotechnical

Seepage through the dike may cause a blowout failure on the downstream face of the dike. This is considered a low risk event because of:

- the low differential head across the dike versus its' width (the maximum dike height in the main Town section is 1.8 m versus a base width of more than 8 m);
- the relatively short duration of a major event with maximum head differential; and
- the competent compacted granular material in the dike (a suspected low proportion of fine silt material).

A local slip failure on the river side of the dike may be a subsequent and supporting failure mechanism associated initially with bank erosion. This might occur locally if unsuitable fill material was used.

5.2.3 Dike Overtopping

Reasons for dike overtopping may include:

- a greater than a 200-year flood event and corresponding higher water level;
- sedimentation in the channel over time and/or during the event;
- a higher than design channel roughness value occurs;
- debris pile-up and blockage such as at the Highway 95 bridges and islands; and
- ice jam blockage primarily during spring breakup.

One or a combination of the above events that may be feasible to occur may result in overtopping, subsequent downslope erosion and eventual breaching of the dike.

5.2.4 Summary

Of the modes discussed above, overtopping is expected to be the most probable mode of potential dike failure. This mode with the potential reasons indicated above and impacts are evaluated further in the following sections.

6.0 RIVER HYDRAULIC MODELING

6.1 INPUT DATA

A re-analysis of the HEC-RAS hydraulic modeling of the river was conducted using the most recent channel cross-section surveys dated November 2002. This re-analysis provides a comparison from Hydroconsult's 1999 report to more precisely assess the probable impact of recent sedimentation. The 1999 study was based upon cross-section survey data up to 1997. Sedimentation effects were recently reviewed in a cursory hydraulic manner (manual calculations rather than detailed modeling) in the report "Assessment of Sedimentation on the Kicking Horse River, Town of Golden" (Hydroconsult, Feb. 2003). The 2003 review was based upon the river survey data up to November 2002. All the previous historic section surveys were presented and compared in this February 2003 report and are therefore not repeated here.

Because not all the sections used in the 1999 hydraulic model were re-surveyed in November 2002, a few section's and the stationing previously used in the 1999 study are used as required to complete the hydraulic model. All other hydraulic model parameters (e.g. roughness, expansion/contraction coefficients, bridge data, starting downstream water level, and calibrations) used in the 1999 study were repeated in the present model. This includes assuming coincident flood peaks on the Columbia and Kicking Horse rivers to define maximum backwater effects. Backwater effects from the Columbia River and the timing of the peaks were discussed in the 1999 report. Assuming coincident peaks with the Columbia River is not significant because this only effects the lower 1 km of the Kicking Horse River. There is minimal impact from overtopping here because of the limited development here and the Columbia will backflood the other side of the dike to a comparable flood level.

The HEC-RAS model Version 3.0 (May 2002) was used in the present study. This is updated from Version 2.2, dated September 1998, as used in the 1999 study. Comparison runs of the same physical model input data with the two versions indicated no differences in the computed water level results beyond minor starting water level computational differences.

The input data used in the present analysis are provided in Appendix B.

6.2 MODEL ASSESSMENT RESULTS

6.2.1 Current Condition Comparisons

Table 2 compares predicted 200-year flood levels with the 1999 study and the present study. The table indicates the sections that are re-surveyed and used in the current model. The left bank freeboard included in Table 2 is based upon the recent September 2003 detailed top of dike surveys. This table comparison shows that increases in potential flood levels due to increased sedimentation, since the last channel excavation in 1997, are minor and less than 10 cm. The major increase of 0.326 m at section K6 is due to an assumed 1 m data reduction error in the 1997 survey data at this section. Comparisons with the 1979 and 1989 computed flood profiles and the historic record of subsequent surveys at this section support this assumption.

Figure 7 shows the channel profile complete with the existing thalweg (minimum bed level), the 200-year flood level at 570 m^3/s, the left and right top of dike levels and the natural land level directly adjacent to the left dike. The flood level at 1,000 m^3/s is also shown in Figure 7 to help illustrate where dike overtopping may occur first and assist in identifying possible areas of greatest risk along the dike system.

Table 2: Kicking Horse River Hydraulic Sections Compared with 1999 Hydraulic Study Results

Section	Date of Section Used in Present Model	Present Study 200 Year Flood Level (m)	Difference in 200 Year Level from 1999 Study(1) (m)	Present Left Bank Freeboard (m)
K1	1987	794.74	0.003	1.66
K52	1987	794.85	0.004	0.62
K2	1987	792.67	-0.005	1.12
K50	Nov / 02	790.67	0.028	0.90
K3	1987	790.23	0.084	1.07
K51	Nov / 02	788.98	-0.156	1.60
K4	Nov / 02	788.80	0.011	1.19
K10	Apr / 02	788.50	-0.016	1.46
K11	1987	788.13	-0.041	1.98
K11B	1987	788.14	-0.037	0.61
K5	Nov / 02	787.80	-0.017	0.76
K6	Nov / 02	785.75	0.007	1.14
K6A	Nov / 02	785.27	0.326(2)	1.05
K7	Nov / 02	785.04	-0.087	1.44
K7A	Nov / 02	784.87	0.062	1.86
K7B	Nov / 02	784.65	0.052	0.69
K8	Nov / 02	784.49	-0.017	0.55
K53	Apr / 02	784.13	0.053	1.75
K54	Nov / 02	784.22	0.015	2.82
K9	Nov / 02	784.16	0.011	2.93
K55	Nov / 02	784.05	0.004	

(1) A negative value means current results are lower than the 1999 study results. The 1999 study results are based upon April and October 1997 cross-section surveys and the common 1987 sections as indicated above.

(2) This difference is due to a 1 m error in the bed level in downstream section K7 in October 1997. Prior and subsequent surveys support this assumed data error.

Figure 8 shows the freeboard along the dike system based upon the computed 200-year flood level. A minimum freeboard of 0.6 m above the 200-year flood level is a suggested guideline by MWLAP. This minimum does not occur in three short locations along the left dike:

- at the campground at the upstream end representing the first area where a breakout flow could occur,
- at the sub-channel downstream of the Highway 95 bridge (grading and top surfacing following the upgrading work just completed in October 2003 is expected to raise the dike grade to or above this minimum freeboard), and
- at a couple of sections at the downstream end about 175 and 375 upstream of the CPR bridge.

The top of the right bank is important to define where overtopping may occur first. Low sections along the right bank occur at:

- Section K50 approximately 550 m upstream of Highway 95 bridge where a 10 m wide overflow section adjacent to the railway may permit a breakout flow. This overbank section widens out progressing downstream. The opposite left dike is also low in this area with less than 1 m freeboard near the College of the Rockies but it is higher than the right bank – meaning a breakout would occur on the north side first.

- A section from 100 to 250 m upstream of Highway 95 bridge. Any overbank flow occurring here and in the upstream bank section will break out and flow north along the railway. This potential right side breakout will occur before the opposite left side overtops – in effect, protecting the south side of Town.

- A 200 m long reach downstream of Highway 95 where the Town has opted to have a reduced freeboard to permit local surface drainage from this commercially developed area. Nearby barriers are stored with plans in place for temporary diking to be placed in this area if/when required.

- The same reach upstream of the CPR bridge and at a similar level as on the left side.

6.2.2 Extreme Case Scenario Evaluations

The 1999 hydraulic study investigated the effect on flood levels due to 0.3 m, 0.5 m and 1 m of sediment deposition and dredging of 0.5 m, 1 m and 2 m below the thalweg in selected reaches. Sensitivities of the flood level to variations of up to 20% in the assumed hydraulic channel roughness values were also computed. Other additional more extreme case scenarios are therefore investigated with the present model to further evaluate the risk of dike overtopping. These more extreme case scenarios, assessed independently, are as follows:

- <u>A greater than the 200-year event</u> discharge equal to 660 m^3/s or possibly in the order of a 500-year event. As previously noted, the flood profile at an extreme envelope discharge value of 1,000 m^3/s (no associated return period) is also plotted in Figure 7.

- <u>An extreme increase in the channel roughness</u> coefficient by a maximum practical limit of 80% reflecting possible extreme impacts of debris, sediment and turbulent flow. This represents an increase from 0.032 to 0.058 in the calibrated roughness value upstream of Highway No. 95 and an increase from a calibrated value of 0.025 to 0.045 downstream of Highway No. 95.

- <u>A debris jam blockage in the south sub-channel</u> at the Highway 95 bridge because of the moderately narrow 18 m wide opening at this location. A complete flow blockage at the bridge is assumed. The computed flow in this sub-channel is approximately 80 m^3/s or 14% of the total river discharge during the 200-year flood peak.

The results from these extreme case scenarios plus the previous results from the assumed 1 m of sediment deposition in the 1999 study are summarized in Table 3.

Table 3: Impact On River Flood Levels For Assumed Extreme Case Scenarios

Section	Increase in Flood Level over Computed 200-Year Flood Level (m)			
	A Peak Discharge of 660 m^3/s	Hydraulic Roughness Value Increases by 80%	Full Blockage of Sub-channel at Highway 95	1 m of Sediment Deposition (K6A to K9)
K1	0.16	1.27	0.01	0
K52	0.30	1.02	0.01	0
K2	0.12	0.74	-0.01	0
K50	0.27	0.95	0.01	0
K3	0.36	0.87	0.03	0
K51	0.22	1.45	0.34	0.02
K4	0.27	1.26	0.48	0.03
K10	0.28	1.23	0.22	0.05
HIGHWAY 95 Br.				
K11	0.26	1.15	0.18	.12
K11b	0.26	0.88	0.15	.12
K5	0.26	0.83	-0.09	.21
K6	0.24	1.14	0	1.00
K6a	0.21	1.14		0.97
K7	0.23	1.19		0.95
K7a	0.24	1.22		0.92
K7b	0.29	1.24		0.84
K8	0.30	1.28		0.78
K53	0.29	1.41		0.50
K54	0.31	1.34		0.52
CPR Bridge				
K54b	0.30	1.37		0.40
K9	0.32	1.36		-0.22
K55	0.32	1.38		0.0
AVERAGE	0.26	1.16		

These extreme case scenarios in Table 3 show:

- The left dike system does not overtop at the flood event of 660 m^3/s. In fact, Figure 7 shows that overtopping of the left dike may not occur until a discharge of about 740 m^3/s. Increasing the discharge from 570 to 660 m^3/s increases the flood level by 0.26 m on average to a maximum of 0.36 m at section K3. The minimum estimated freeboard at any point along the left dike is reduced to nearly 0.2 m at this discharge.

- An 80% increase in the assumed hydraulic roughness increases the predicted flood level by an average of 1.16 m. Under this assumption, extensive areas of overtopping may be expected to begin to occur with significant overtopping in the identified low areas.

- Full blockage of the sub-channel at Highway 95 bridge does not significantly increase upstream flood levels. The maximum local increase is about 0.5 m. This effect reduces to negligible about 280 m upstream. Overtopping of the left dike would not occur with this assumption alone. However, the clearance at the main Highway 95 bridge at the 200-year flood level is only 0.32 m and mean channel velocities in the main channel are estimated to increase from 3.53 to 3.86 m/s on average. This increases the risk of erosion at the main bridge.

- The 1m of sediment deposition in the lower reach downstream of the island increases the flood level by up to 1 m at section K6. In this case the freeboard is reduced to negligible at section K6a and overtopping of the low areas downstream of section K7b would occur.

Modeling of higher flows up to 1,000 m^3/s extends the stage-discharge relations along the river to allow estimation of where the dike may overtop first and at what possible discharge. As seen in Figure 7, six locations are identified where the left dike overtops with assumed discharges ranging from 740 m^3/s to 910 m^3/s.

7.0 ICE EVENT CONSIDERATIONS

7.1 HISTORIC ICE INFORMATION

The risk of dike overtopping due to ice conditions as opposed to an open water flood event is discussed in this section.

Site specific recorded data on ice levels are limited to anecdotal information and the historical observations and photos presented in Appendix A. The WSC gauging station does not specifically record ice levels other than during the spot winter flow measurements. However, the WSC station records do give an indication of start and end dates of ice conditions and corresponding river flows. Based on 32 years of record, these are as follows:

- Start of ice conditions is from October 30 to December 19 and averages November 23rd. Discharges at this time average 8.7 m^3/s with 16.8 m^3/s the maximum recorded.
- Ice free conditions or breakup is from February 21 to April 27 and averages March 27th. Discharges at this time average 6.4 m^3/s with 13.6 m^3/s the maximum recorded.

The photos in Appendix A show that recent maximum recorded ice levels have typically been well below dike crest levels other than where ice is locally shoved up on the banks. Although this ice shove may effect the dike armouring to some extent, actual observed water levels tend to be much lower than the top of dike level.

The severe high ice conditions shown in the December 1955 photos in Appendix A may have been exacerbated by the constriction created by the former railroad bridge. This former bridge was located on a skew approximately 150 m upstream of the present Highway 95 bridge crossing. It is difficult to estimate the peak ice and water level in 1955 but it is believed to be adequately below the present dike level based on the photo documentation. (Photo A1 of the 1916 flood and A2 of the 1955 ice level show the ice level was not much higher than the 1916 flood level.) With the railroad bridge constriction now removed the risk of a similar ice event may be reduced. The former sub-channel in the left overbank starting at the Kinsmen Park (Photo A6) has now been filled in.

7.2 ICE JAM PROCESSES AND RISKS

Severe ice jamming or build-up is possible in this reach of the Kicking Horse River within the Town and upstream based upon the river's confined nature and gradient and breakup nature. High ice conditions may occur due to frazil ice during freeze up, as a mid-winter breakup (at low flow with a sudden melt) and as an ice jam at breakup.

The numerous ice event photos in December, shown in Appendix A, suggest extensive frazil ice growth is common. Frazil ice growth tends to start in the lower gradient segments and builds progressing upstream.

The high elevation of the basin and low winter flow means that sudden melt events with significant flow increases are not likely to occur in this basin. The historic flow data indicate that the maximum daily peak discharge from January to March for the period of record is 16.4 m^3/s.

Extreme ice jams occur when thick, competent ice builds during breakup combined with a sudden increase in flow. The flatter reaches with islands and bars such as upstream of section K50 near the College of the Rockies and to a lesser extent the reach downstream of the pedestrian bridge may be reaches most susceptible to ice jamming. However, the channel splits provide flow relief routes and the straight channel alignment does not promote jamming at any specific location. The most severe ice jams with the greatest risk of dike overtopping may be expected to occur when the ice becomes grounded forming a dam leading to an upstream increase in water level. In order to develop a significant dike overtopping flow an ice jam would need to develop upstream in the canyon and suddenly release combined with a subsequent jam downstream. However, the steep channel gradient and narrow conditions upstream in the canyon limits the potential for any significant storage volume to be created.

wrong

water

It is considered possible that an extreme ice event (either frazil or breakup jamming) could create some dike overtopping flow. However, the flow records show that the river flow during ice conditions rarely exceeds 20 m³/s and is typically less than 10 m³/s. Further, the potential for any significant storage releases is low. Complete flow blockage in the channel is considered highly unlikely, particularly with staging to the top of dike level. Therefore, the magnitude of any potential overtopping flow is expected to be less than 10 m³/s. This magnitude is expected to be lower than that which may be possible during open water flood conditions.

Any potential overbank flow depths due to ice conditions are therefore expected be quite minor and, although problematic during winter conditions, they are expected to be manageable and pose less risk of damage than an open water flood event.

8.0 OVERBANK HYDRAULIC ANALYSES

8.1 DIKE BREACH SCENARIOS

Assuming dike overtopping does occur, it will probably be due to a combination of some of the extreme case scenarios discussed above. The likelihood of these scenarios occurring in combination and of them being adequate to cause a breach is summarized in Table 4.

Table 4: Risk Assessment of Overtopping Scenarios

Overtopping Scenarios	Likelihood of Scenarios Occurring in Combination					Likelihood of Causing		Potential Consequences of Scenario
	1.	2.	3.	4.	5.	Overtopping	Breaching	
1. >200 Year Flood Peak Discharge	A	D	C	C	E	D	D	I – II
2. Increased Channel Roughness		A	D	B	E	D	D	I – II
3. Sedimentation			A	B	E	D-E	E	III
4. Debris Blockage				A	E	E	E	III
5. Ice Conditions					A	D	E	IV

Probability:
A = Is occurring / will occur
B = Very likely to occur
C = Likely to occur
D = not likely to occur
E = practically impossible

Potential Consequences:
I – Public Health and Safety
II – Property Damage
III – Minor Public Damage / Repairs
IV – Public disruption / cleanup

The most likely locations where overtopping will occur are based upon the dike level and channel conditions previously identified in Section 5.1.

The magnitude of a breakout flow when overtopping and breaching occurs may be hydraulically estimated based upon the head difference across the dike. However, defining the occurrence, timing and size of a breach is highly speculative. From historical data including anecdotal records of various observed breaches in roads, dikes and gravel fill embankments, it is expected that overtopping by a depth of as little as 20 to 30 cm may be adequate to cause erosion of the downstream slope and then the crest of the dike. Overtopping for a period of one to two hours may be sufficient for the downstream (land) side of the dike to start to erode. Once this erosion starts, development of a full breach in the dike may then occur in a matter of hours and the actual subsequent time to develop a significant breach and a breakout flow can be a matter of minutes. Dam breaches are commonly assumed to occur instantaneously in model analyses (DAMBRK, 1993). However, on the Kicking Horse River, the moderate gradient across the dike combined with the riprap facing on the dike might be expected to slow the development rate, size and depth of the breach. Considering the above, a period of two to six or more hours of sustained high water level that overtops the dike may be required to cause a significant breach in the dike. The average river flood hydrograph shape, discussed earlier, concluded that discharges and corresponding flood water levels might be expected to remain within 98% of their peak for 3.5 hours and within 95% of the peak for 10 hours.

Rectangular, trapezoidal or triangular shaped breaches are typically assumed to develop with the bottom of the cut approaching the natural ground level adjacent to the dike. Estimates of breach widths and timing have been developed from review of historical data on dam failures (MacDonald and Langridge-Monopolis, 1984 and Froelich 1987). An equation developed by Froelich, primarily based on dams from 4 to 30 m high, is based upon reservoir volume and dam height. Although not directly applicable to a river dike situation, this equation suggests that a 20 to 30 m wide breach may develop for a 1.2 m high dike.

For assessment purposes, a maximum 30 m wide, rectangular shaped breach was evaluated at the most upstream site at the campground, as illustrated in Figure 9. This is about the widest breach width that might be expected to develop given that the maximum length of initial overtopping is about 100 m. Initial hydraulic analyses applying a weir flow equation compute a peak flow in the order of 75 m^3/s in such a breach without considering any downstream backwater conditions to limit the breakout flow. Incorporating backwater conditions with a minimum downstream flood depth of 0.6 m in the campground area and assuming the river flood level remains at 0.2 m above the dike crest, the maximum breakout flow is estimated at 45 m^3/s at the campground. As an increasing breakout flow occurs, the river level and upstream head will begin to drop by approximately 0.25 m, meaning that the peak breakout flow will also rapidly drop.

8.2 OVERLAND FLOW MODELING

Modeling and analysis of overland flow conditions to estimate potential flood levels is also highly speculative because of the complexity of possible flow paths and variable roughness conditions. Recognizing these limitations, several simplifying assumptions were made to compute potential overland flood levels.

The assumed breakout flow hydrograph resulting from the breach condition described above starts as an overtopping discharge of approximately 5 m^3/s for 2 hours. At this time the breach occurs and the flow almost immediately increases to 45 m^3/s and remains at this flow rate for at least 2 hours, then drops to zero almost immediately as the river water level drops below the level of the breach. Hydrologic routing scenarios of this hydrograph using the program FLDWAV (Fread and Lewis, 1998) does not predict any appreciable decrease in the peak overland flow due to storage and attenuation effects – a drop of less than 5% in the peak flow may occur. Hydraulic analyses were therefore conducted using HEC-RAS and assuming a constant steady state overland flow of 45 m^3/s.

The average computed flow depth from hydraulically modeling 45 m^3/s down 9th Street is approximately 0.9 m (depths vary locally depending upon high/low points along the street). This flow depth reduces progressing downstream and away from the dike as the breakout flow spreads out. The flow is assumed to split down the various streets according to the width and grade of the various streets. All the flow is assumed to occur along the streets. Based upon these assumptions, hydraulic estimates suggest that the average depth of flow is approximately 0.3 m by the time the peak flow reaches Highway 95. Deeper ponding will occur in site specific low points along the various roads.

Similar hydraulic modeling for breakout flow rates of 15 m^3/s and 5 m^3/s indicate average flow depths in the order of 0.2 m and 0.1 m, respectively, along the main overland flow routes (i.e. 9th Street). These depths are assumed as representative of maximum overland flow depths that may develop due to overtopping without dike breaching.

8.3 ASSESSMENT OF ALTERNATIVE BREACHES

Evaluations were conducted at the other downstream sites along the left dike assuming similar breach and overland flooding conditions. The magnitude of potential breakout flows and risks of overland flooding are reduced at these locations, as follows:

- Any breakout flow in the vicinity of the College of the Rockies is limited to an overtopping flow because natural ground level is at most only 0.6 m below the top of dyke level in this area. The opposite bank level is also about 0.3 m lower here.

- The left dike level upstream of Highway 95 is more than 0.5 m higher than the right dike, suggesting that any breakout flow will occur on the right (north) side first. In addition, the flood level would have to be 1.2 m higher than the estimated 200-year flood level here before the left dyke overtops. Therefore, the risk of overtopping and breaching of the left dike in this location is considered quite low.

- Bank protection has recently (fall 2003) been upgraded along the left dike of the sub-channel downstream of Highway 95 and the final dike grade is expected to be raised slightly. The potential for a significant breach at this location is limited by the small portion of the total river flow in this sub-channel and the current level of the dike protection along here.

- Dike breaching downstream of section K7 may result in a significant breach and breakout flow because of the much lower ground level behind the dike in this location and overtopping is the main dike failure risk. However, the current freeboard is greater than 1 m for a distance of over 200 downstream of K7. Further downstream of here the flood risk levels from the Columbia River apply rather than any dike overtopping scenario.

A summary of the likely order of occurrence of an overtopping breakout flow is provided below based strictly on the level of freeboard at the computed 200-year flood level (This assumes that the existing low right bank area downstream of Highway 95 is appropriately blocked.):

1. Right side 550 m upstream of Highway 95 (Section K50), width of overflow here is limited by the railway
2. Left side at campground (downstream of Section K52)
3. Left side in sub-channel downstream of Highway 95 (downstream of K11b) – to be raised
4. Left and right side in lower reach 200 –300 m upstream of CPR bridge (Sections K7b to K8)
5. Right side 250 m upstream of Highway 95 bridge (Section K51)
6. Left side at College of the Rockies (upstream of Section K50)

9.0 FLOOD RISK MAPPING DISCUSSION

A flood risk mapping for the Town of Golden on the south (left) side of the Kicking Horse River that is protected by the current dike system must consider a number of factors, as previously discussed. For significant overland flooding to occur, the following will have to occur:

- a water level must occur that is at least 0.7 m higher than the estimated 200-year flood level for the computed discharge of 570 m^3/s (i.e. a flood of over 800 m^3/s at the campground) ;
- this water level must be sustained for a period of at least 2 hours;
- no remedial protection measures are taken;
- erosion occurs on the downstream side of the dike leading to a significant breach; and
- the water level in the river would need to remain at or near the top of dike level for a further period of at least 2 hours for adequate water to break out and create significant downstream flooding.

Applying the provincial guidelines, it may be stated that the present dike system is appropriately protected and stable, with adequate freeboard (at least 0.6 m in most all locations) above the designated 200-year flood level equivalent to a peak discharge of 570 m^3/s. Therefore, no flood risk and no development restriction guidelines should be applied to the area protected by this dike system. This assumes that the dike system is maintained.

On the other hand, with the extreme case scenarios presented, it is shown that some risk of flooding exists. A depth of flooding of up to 0.9 m may be possible in areas. This depth of flooding reduces progressing downstream from the breach and further away from the dike.

The possible range of flood risk mapping options that might be considered are as follows:

1. Apply no specific development restrictions, but indicate that the Town is in a flood risk zone and property owners are to be aware of potential overland flooding risks. The Town would be responsible for dike monitoring and maintenance, and having an Emergency Preparedness Plan (EPP) in place.

2. Apply a graduated flood risk level depending upon proximity to the river. For example, define the potential flood level as 0.9 m deep within 100 meters of the left dyke. Reduce this to 0.6 m deep in the area from 100 to 300 m from the dyke and to 0.3 m within the remaining area extending to the Columbia River flood risk limit (as previously defined in the 1979 mapping).

The second option is considered excessive and would be difficult to manage and implement because of the graduated zones. Normal sound building practices where buildings are located higher than the roads may be adequate for the low risk and extent of potential overland flooding that has been identified.

10.0 CONCLUSIONS AND RECOMMENDATIONS

The recommended value for the 200-year return period peak discharge is 570 m^3/s. Higher peak events are possible to an upper envelope value approaching 1,000 m^3/s (no return period).

Of the modes of potential dike failure evaluated, dike overtopping and subsequent breaching is expected to be the most probable mechanism causing overland flooding. River hydraulic modeling based on surveyed sections from 1997 to 2002 show that the differences in predicted water levels during the 200-year flood peak of 570 m^3/s are minor and less than 10 cm – indicating that any sedimentation impacts over this period have been minor. The existing south (left) dike freeboard at the computed 200-year flood level is greater than MWLAP's recommended minimum guideline of 0.6 m in all but three short locations where it drops to as low as 0.5 m in these areas.

Overtopping of the left dike does not occur until a river discharge of 740 m^3/s is exceeded, assuming the model inputs and assumptions are correct. Assumed extreme case scenario evaluations show that dike overtopping can occur, likely as a result of a combination of extreme conditions (local sediment deposition, debris and increased values in assumed channel roughness as well as a flood peak exceeding 570 m^3/s). Ice conditions are not expected to pose as great a risk as an open water flood event because the magnitude of any potential breakout flow will be low, will not threaten the integrity of the dike and the depth of overland flow will be minor.

Analysis of potential dike breaching as a result of overtopping indicates that the campground area has the greatest risk for a breach and a significant breakout flow. Overtopping flows without dike breaching may be in the order of 5 to 15 m^3/s. Downstream overland flow depths are estimated to be in the order of 0.1 to 0.2 m under these breakout flows. An assumed breach 30 m wide by 1.2 m deep may have a breakout flow of up to 45 m^3/s and result in an average flow depth of approximately 0.9 m along the upper end of 9th Street. This depth of flow reduces as it spreads out downstream and may have an average depth of 0.3 m by the time it reaches Highway 95 with greater ponding in local low points. For this extreme case scenario to occur, a number of conditions would have to be met - meaning the risk of this occurring is low.

Because the extreme case scenarios show some risk of flooding exists, the Town is in a flood risk zone and property owners are to be aware of potential overland flooding risks in this area. Applying the provincial guidelines, the dike system is appropriately protected and stable, with adequate freeboard (at least 0.6 m in most all locations) above the designated 200-year flood level equivalent to a peak discharge of 570 m^3/s. Therefore, no flood risk development restriction guidelines are recommended to be applied to the area protected by this dike system, provided that appropriate ongoing dike monitoring and maintenance is conducted. This includes:

- locally raising the dike by about 0.1 m in the few identified low locations,
- annually inspecting the dike sideslopes, the protective riprap and the dike crest,
- replacing /stabilizing the riprap, as required, and
- continuing to conduct and evaluate river cross-section surveys at least every two or three years to assess sedimentation conditions.

In addition, an Emergency Preparedness Plan (EPP) should be in place, updated and tested on a regular basis.

11.0 REFERENCES AND DATA SOURCES

BC Ministry of Environment, 1979. *Floodplain Mapping Columbia River at Golden,* 4 Sheets, 1:5000 scale maps produced by Environmental and Engineering Service Floodplain Mapping Program.

BC Ministry of Environment and Parks, February 1987. *A Procedure for Regionalization of Peak Flows in B. C.,* D. E. Reksten, Water Management Branch, Hydrology Section.

BC Ministry of Environment and Parks, August 1988. Kicking Horse River dike upgrading Plans and Sections, Water Management Branch. Surveys May 1987. Project No. 87-RPP-3.

BC Ministry of Sustainable Resource Management, January, 2002. *Streamflow in the Kootenay Region.* W. Obedkoff, P. Eng., Water Information Section, Aquatic Information Branch.

BC Ministry of Transport, 1990. *Golden to Yoho National Park Corridor Study,* 1:5000 scale, 5 metre contour interval map.

BOSS International, 1993. *DAMBRK Hydrodymanic Flood Routing, User's Manual.*

Environment Canada, March 1993. *Consolidated Frequency Analysis (CFA) Version 3.1,* Paul J. Pilon and K. David Harvey, Interpretations and Applications Division, Ecosystem Sciences and Evaluation Directorate.

Environment Canada, 2003. Historical streamflow data, various stations, HYDAT CD-ROM data to 1999 and preliminary data to 2003 and Kicking Horse River stage discharge data. Water Survey of Canada. Historical precipitation and snow course data at http:/climate.weatheroffice.ec.gc.ca/climateData/.

Fread, D. L. and J. M. Lewis, 1998. *The NWS FLDWAV Model: Theoretical Description and User Documentation,* FLDWAV, Version 1.0. Hydrologic Research Laboratory, U.S. National Weather Service, Silver Spring, MD.

Fread D. L., 1988. *The NWS DAMBRK Model: Theoretical Background and User Documentation,* Hydrologic Research Laboratory, National Weather Service, Silver Spring, MD.

Froehlich, D. C., 1987. *Embankment-Dam Breach Parameters*, Proceedings of the 1987 National Conference on Hydraulic Engineering, ASCE, New York, New York.

Hydrocon Engineering, December, 1982. *Elk River Project Hydrology Summary 1977-1981,* Prepared for Elco Mining Ltd.

Hydroconsult, April 1999. *Hydraulic Modeling of the Kicking Horse River to Determine Channel Capacity,* prepared for Town of Golden.

Hydroconsult May 1999a. *Elkview Mine Hydrologic Assessment.* Prepared for Elkview Coal Corporation.

Hydroconsult, February 2003. *Assessment of Sedimentation on the Kicking Horse River, Town of Golden,* prepared for Town of Golden.

MacDonald, T. C., and J. Langridge-Monopolis, 1984. *Breaching Characteristics of Dam Failures.* Journ. Hydraulics Div., ASCE, 110, No. 5.

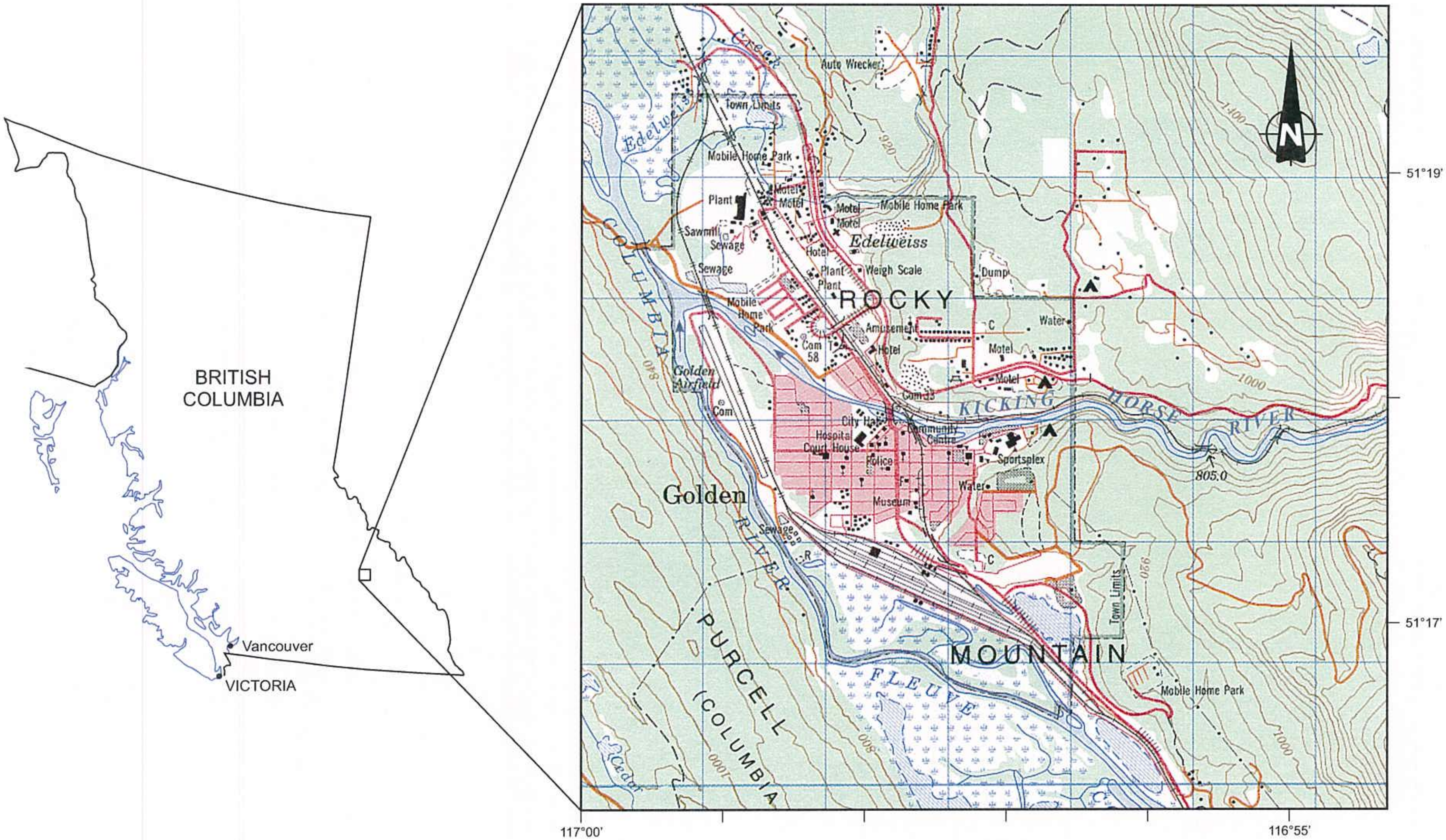
Ward & Associates Ltd., April 1996. *Flood Potential from Kicking Horse River Overflows at Golden: Preliminary Report,* prepared for Town of Golden.

US Army Corps of Engineers (USACE), May 1992. *HEC-FFA Users Manual,* Hydrologic Engineering Centre. Davis, CA., CPD-13.

US Army Corps of Engineers, May, 2002. *HEC-RAS River Analysis System,* Version 3.0. Hydrologic Engineering Centre. Davis, CA., CPD-68.

United States Water Resources Council, Revised September 1981. *Guidelines for Determining Flood Flow Frequency.* Bulletin #17B of the Hydrology Committee.

FIGURES



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TOWN OF OPPORTUNITY
GOLDEN
BRITISH COLUMBIA

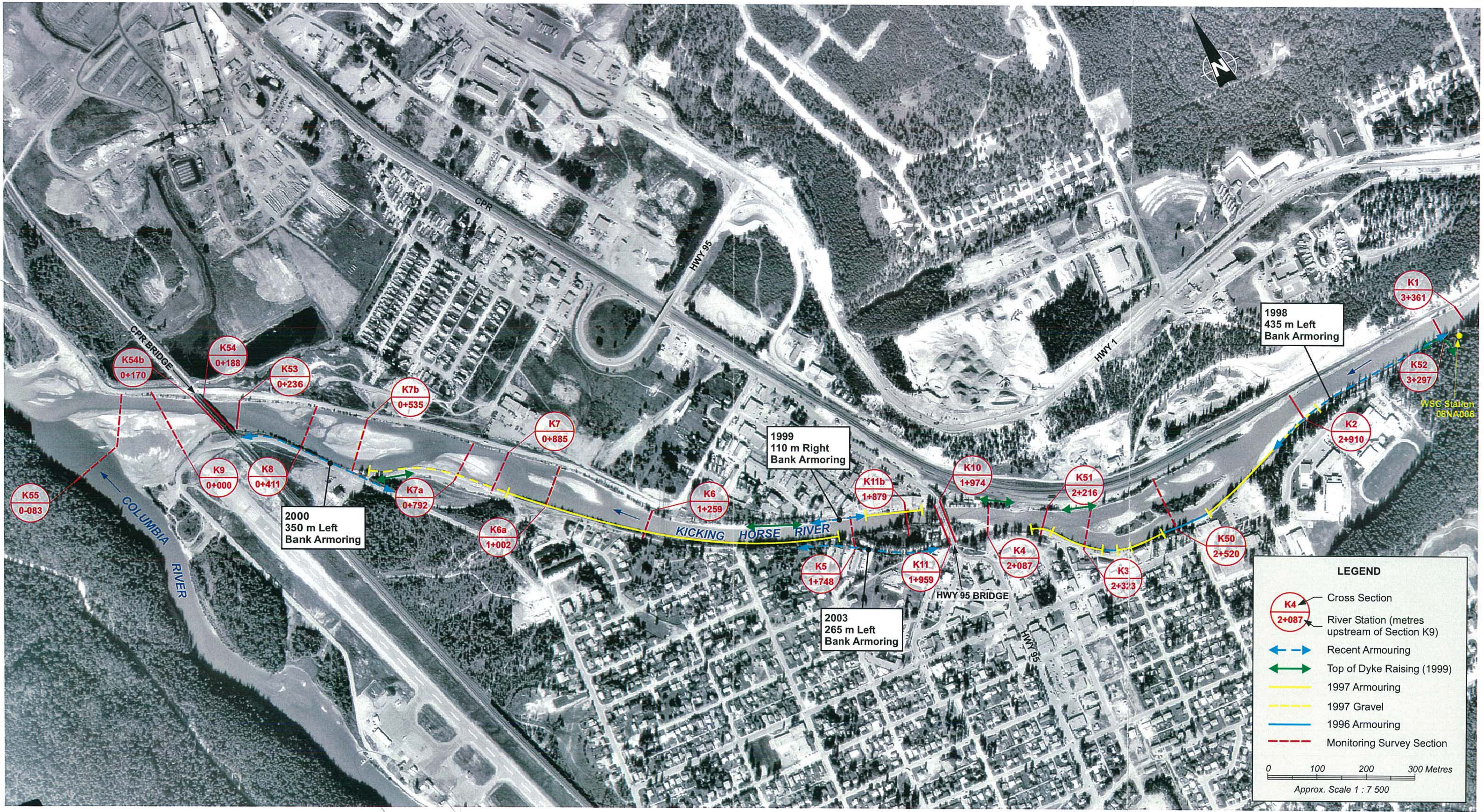
DRAFT FOR REVIEW

DRAWN	Gary Evenson
DESIGN	Greg Standen
APP	Wim Veldman
SCALE	As shown
DATE	03/10/22

FLOOD RISK MAPPING ASSESSMENT

LOCATION PLAN

FILE (CDR) 785-03-12-22-A1 | JOB NO. **785** | **FIGURE 1**



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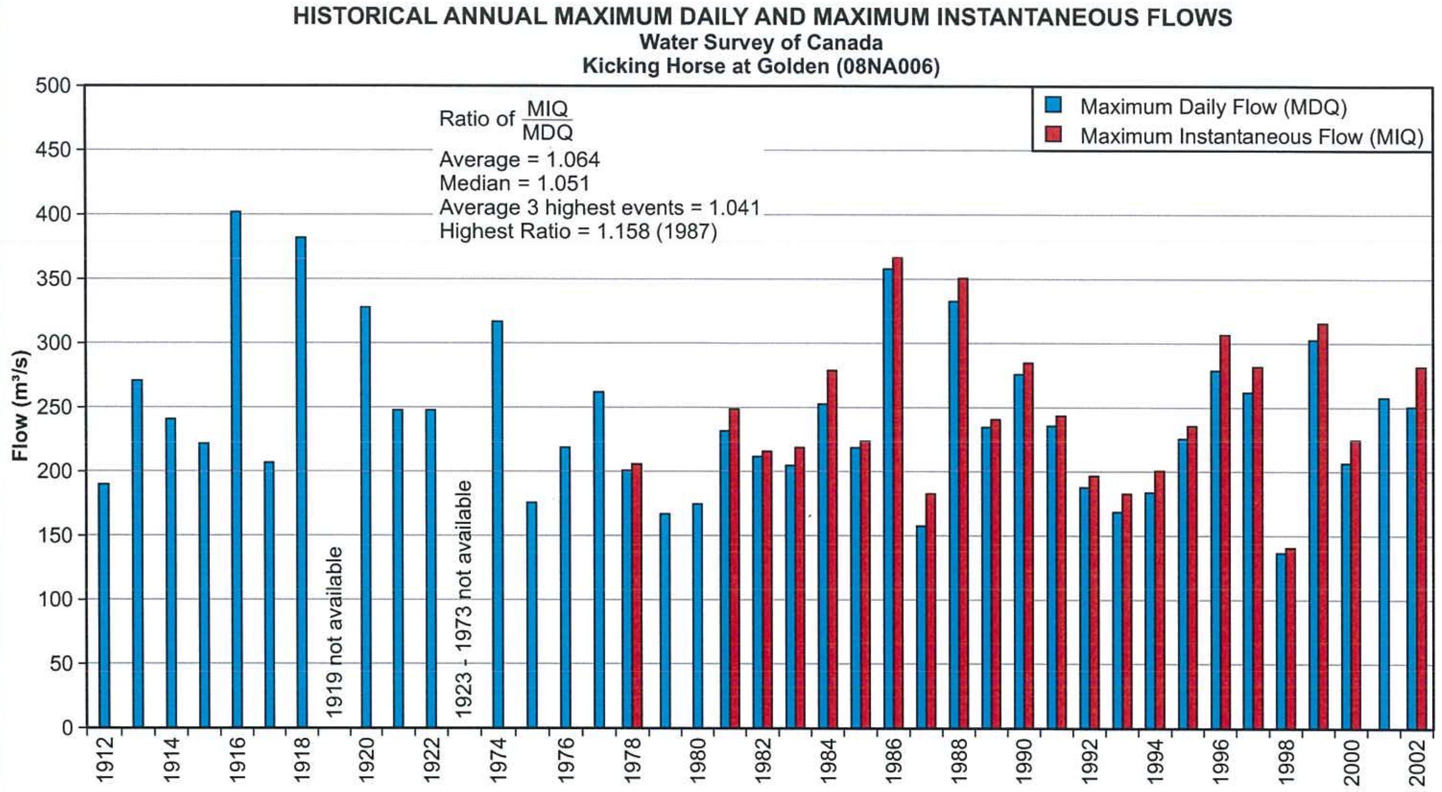
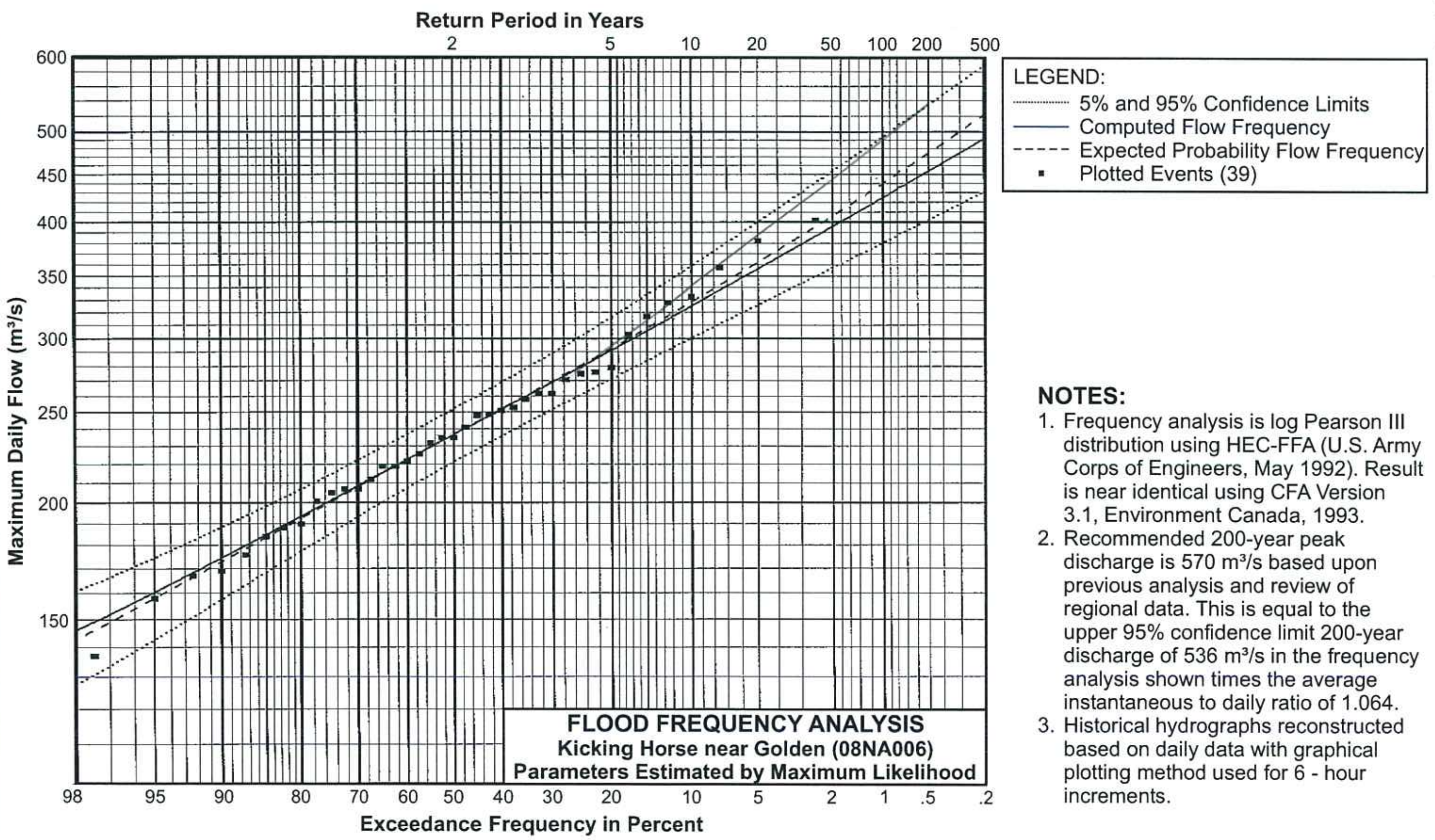
SCALE As shown
DATE 03/08/19

FLOOD RISK MAPPING ASSESSMENT

KICKING HORSE RIVER CROSS SECTION LOCATIONS AND HISTORY OF RECENT DYKE UPGRADES

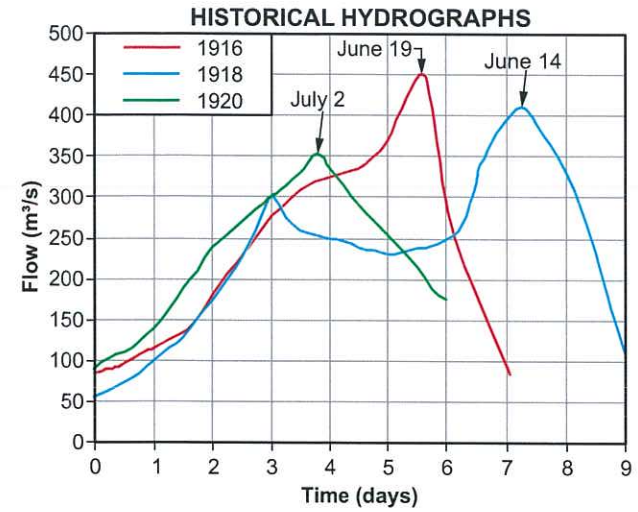
FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | FIGURE 2

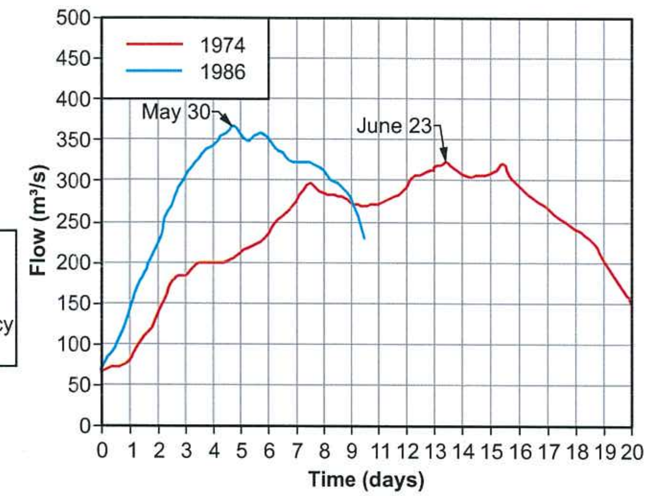


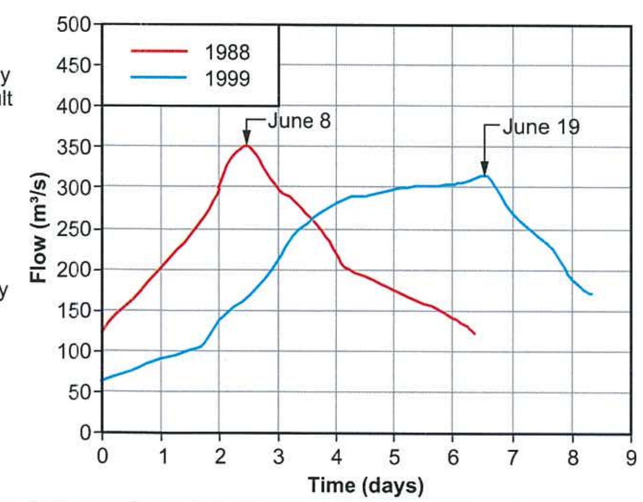


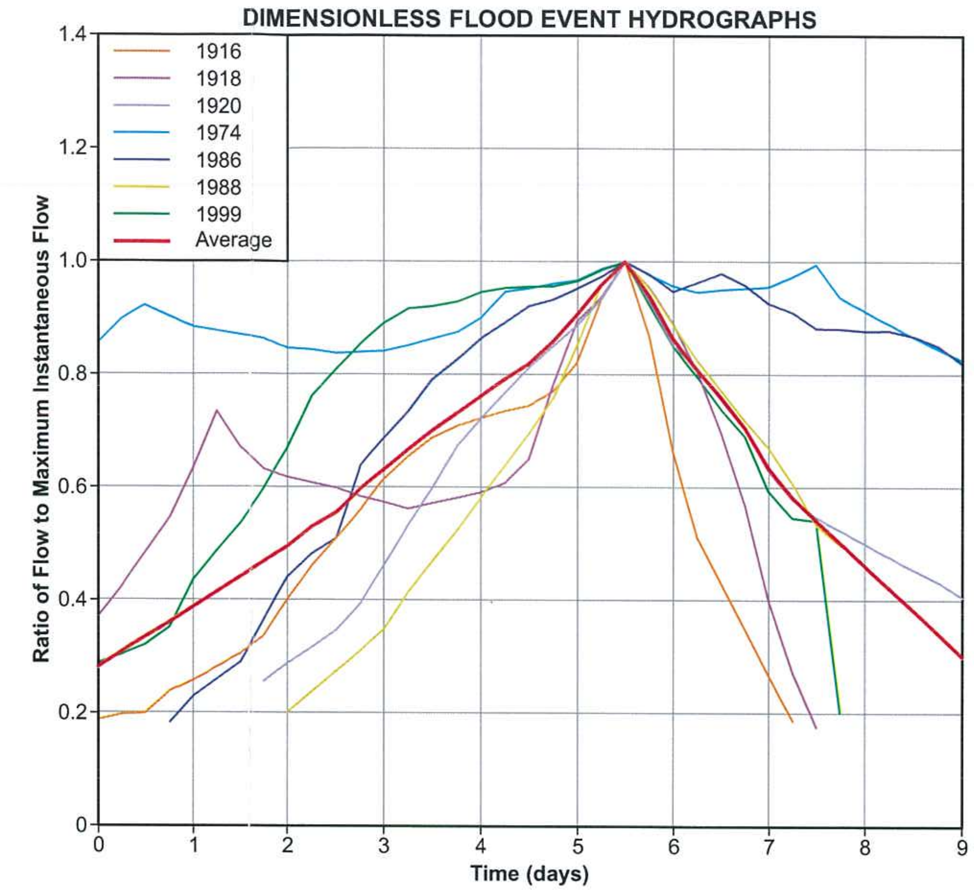
NOTES:

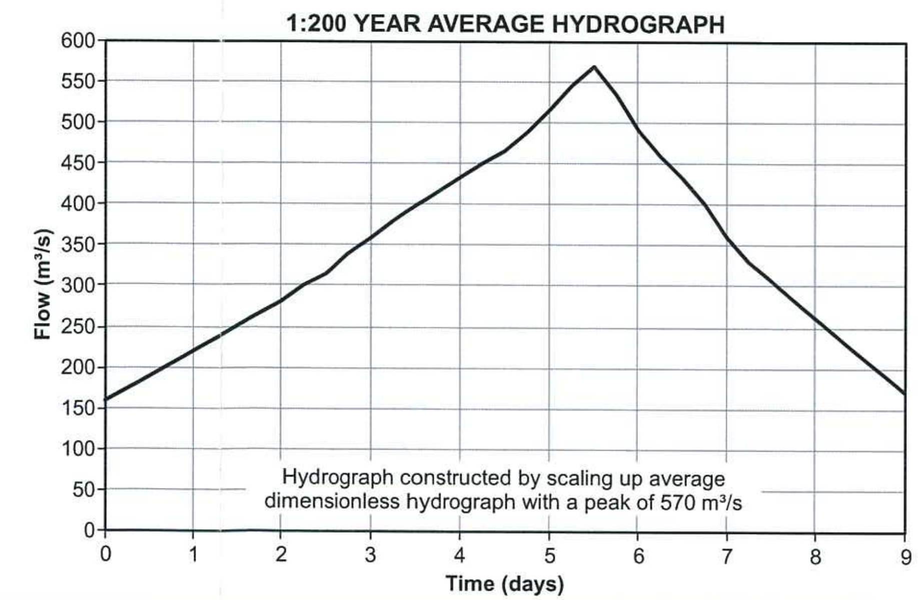
1. Frequency analysis is log Pearson III distribution using HEC-FFA (U.S. Army Corps of Engineers, May 1992). Result is near identical using CFA Version 3.1, Environment Canada, 1993.
2. Recommended 200-year peak discharge is 570 m³/s based upon previous analysis and review of regional data. This is equal to the upper 95% confidence limit 200-year discharge of 536 m³/s in the frequency analysis shown times the average instantaneous to daily ratio of 1.064.
3. Historical hydrographs reconstructed based on daily data with graphical plotting method used for 6 - hour increments.











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DESIGN Juan Sarmiento
APP Wim Veldman
SCALE As shown
DATE 03/08/19

FLOOD RISK MAPPING ASSESSMENT

HYDROLOGIC ANALYSIS

FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | **FIGURE 3**

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TOWN OF OPPORTUNITY
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DATE 03/10/09

FLOOD RISK MAPPING ASSESSMENT

BASE CONTOUR MAP

FILE (DWG) 785-03-10-09-A1
JOB NO. 785
FIGURE 4

1. Entrance to campground looking downstream. The dyke crest is 4 m wide at the top with a 1.6 m drop to the ground level beside the road. Potential breach area to evaluate.

2. Looking downstream at College of the Rockies property. The dyke crest is 0.66 m above the natural ground and 4 m wide at the crest. Minimal risk of major dyke breaching here, although on outside bend of river and freeboard is low.

3. Looking downstream on south side of the Highway 95 bridge. This area has a significant elevation drop of 1.5 m from the dyke to the road level at the intersection with 11th Avenue South (just left of the photo).

4. Skateboard Park looking upstream. The dyke crest is 0.96 m above the ground in the park area and 1.4 m above the road with a crest width of 3 m. The river side of the dyke is slightly steep at 55%. Potential breach area to evaluate.

Photos taken by Dave Cooper of Hydroconsult on July 31, 2003

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FLOOD RISK MAPPING ASSESSMENT

DYKE CREST SITE PHOTOS
1 TO 4

FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | FIGURE 5

5. Looking upstream along dyke beside sub-channel downstream of Highway 95 bridge. Significant drop to natural ground of approximately 1.5 m.

6. Looking downstream from location of Photo 5. The narrow 3 m wide dyke crest is 1.8 m above the natural ground level. The landward slope of the dyke is oversteep at 62%. Dyke upgrading is planned along this sub-channel reach in September - October, 2003.

7. Photo taken from near Section K6 looking downstream. The 6 m wide dyke crest is 1.7 m above the natural ground on the left. The natural ground on the left is a local depression. Residential area ground level is about 1 m below the dyke crest.

8. Photo taken from near Section K7 looking downstream. The 6 m wide dyke crest is 3.4 m above the natural ground on the left.

9. Photo taken halfway between Section K7a and K7b looking downstream. The 5 m wide dyke crest is approximately 3 m above natural ground level.

Photos taken by Dave Cooper of Hydroconsult on July 31, 2003

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TOWN OF OPPORTUNITY GOLDEN BRITISH COLUMBIA

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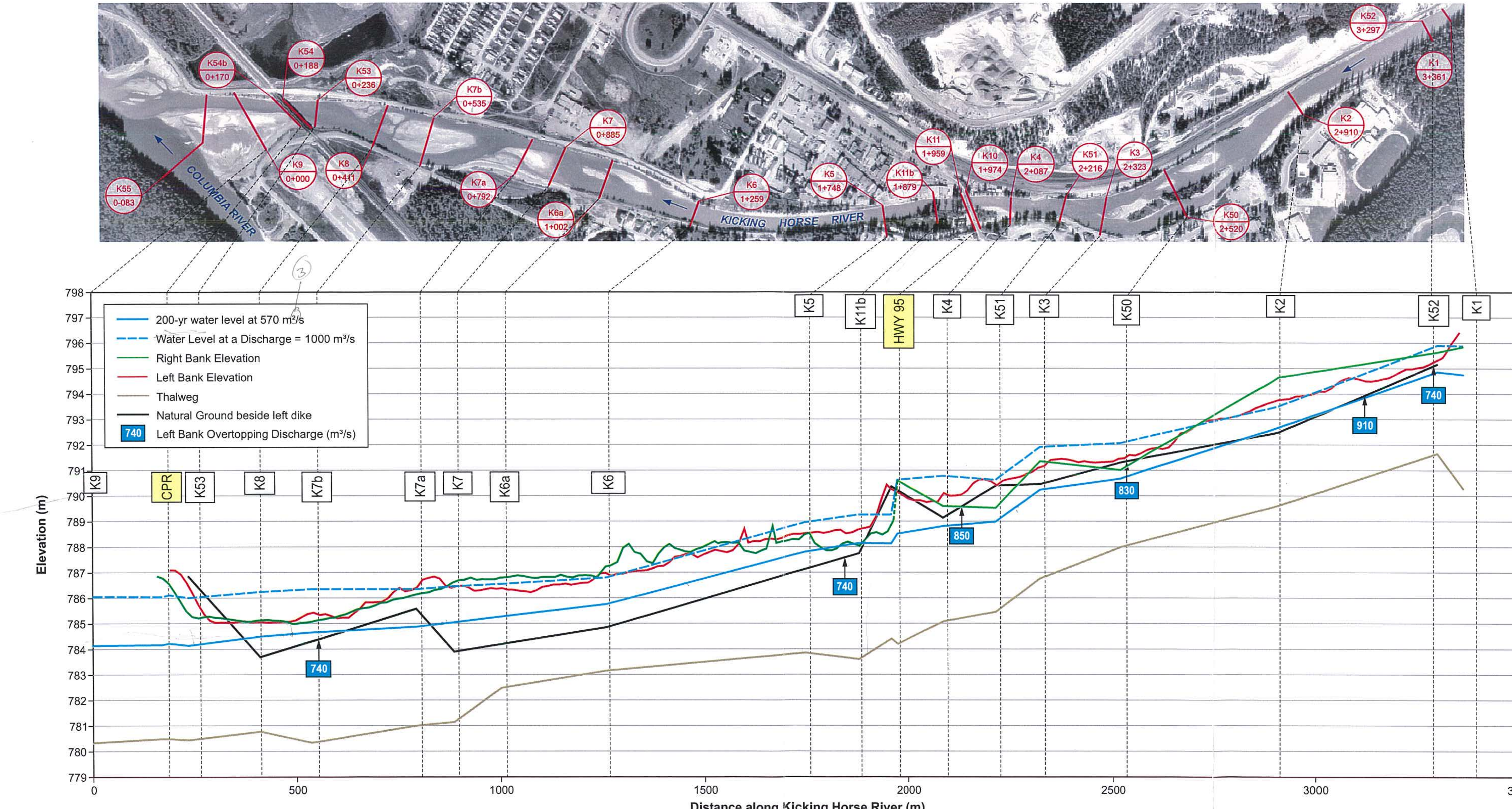
FLOOD RISK MAPPING ASSESSMENT

DYKE CREST SITE PHOTOS 5 TO 9

FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | **FIGURE 6**



NOTE:

1. Right bank and left bank are defined looking downstream.
2. Right and left bank elevations based on surveys by Alpine Surveys conducted September 3, 2003. Right bank elevations from Hwy 95 upstream based on October 1999 bank surveys.
3. Water Levels and thalweg based on latest available cross-section surveys conducted November, 2002.

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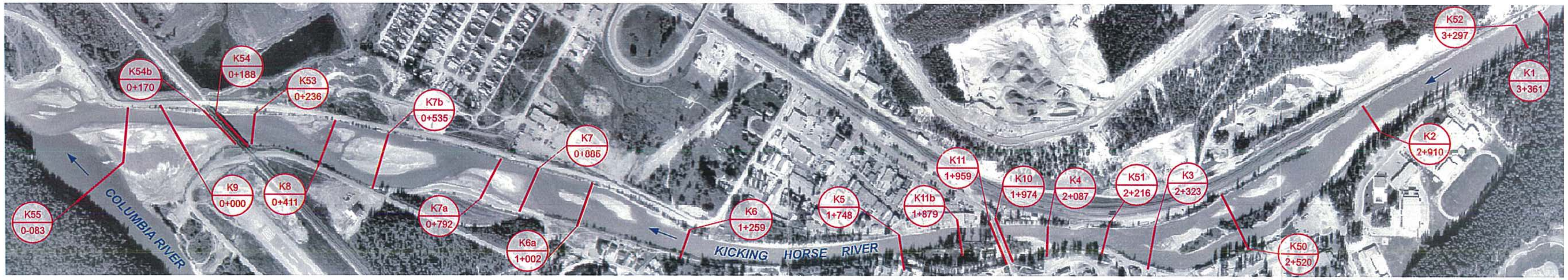
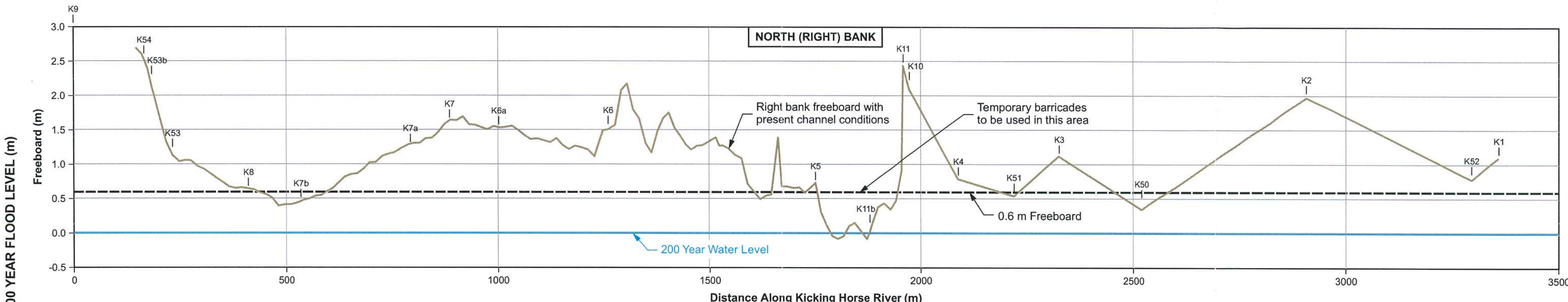
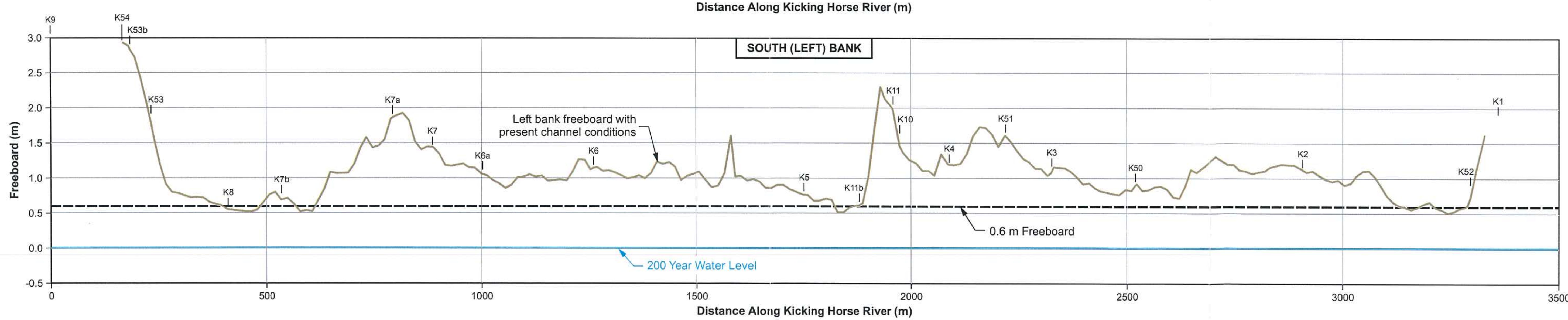
FLOOD RISK MAPPING ASSESSMENT

BANK LEVELS VERSUS FLOOD DISCHARGE LEVELS

FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | **FIGURE 7**







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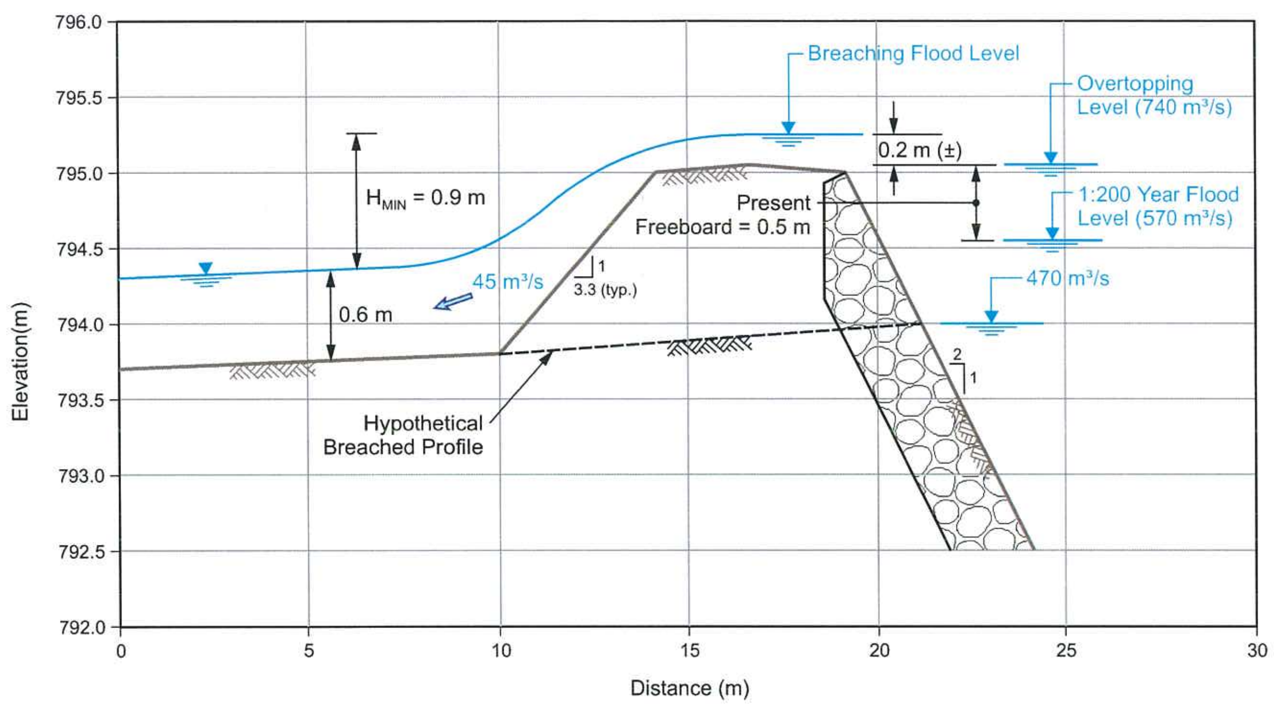
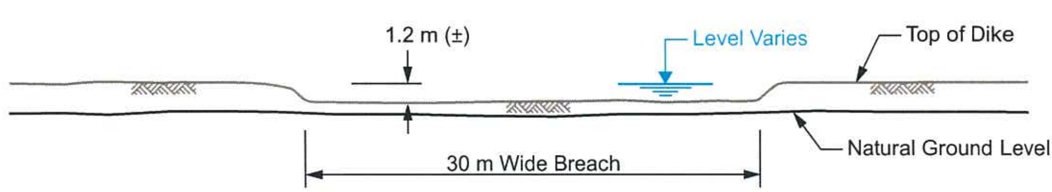
FLOOD RISK MAPPING ASSESSMENT

FREEBOARD OF LEFT AND RIGHT DYKES

FILE (CDR) 785-03-08-19-A1 | JOB NO. 785 | **FIGURE 8**

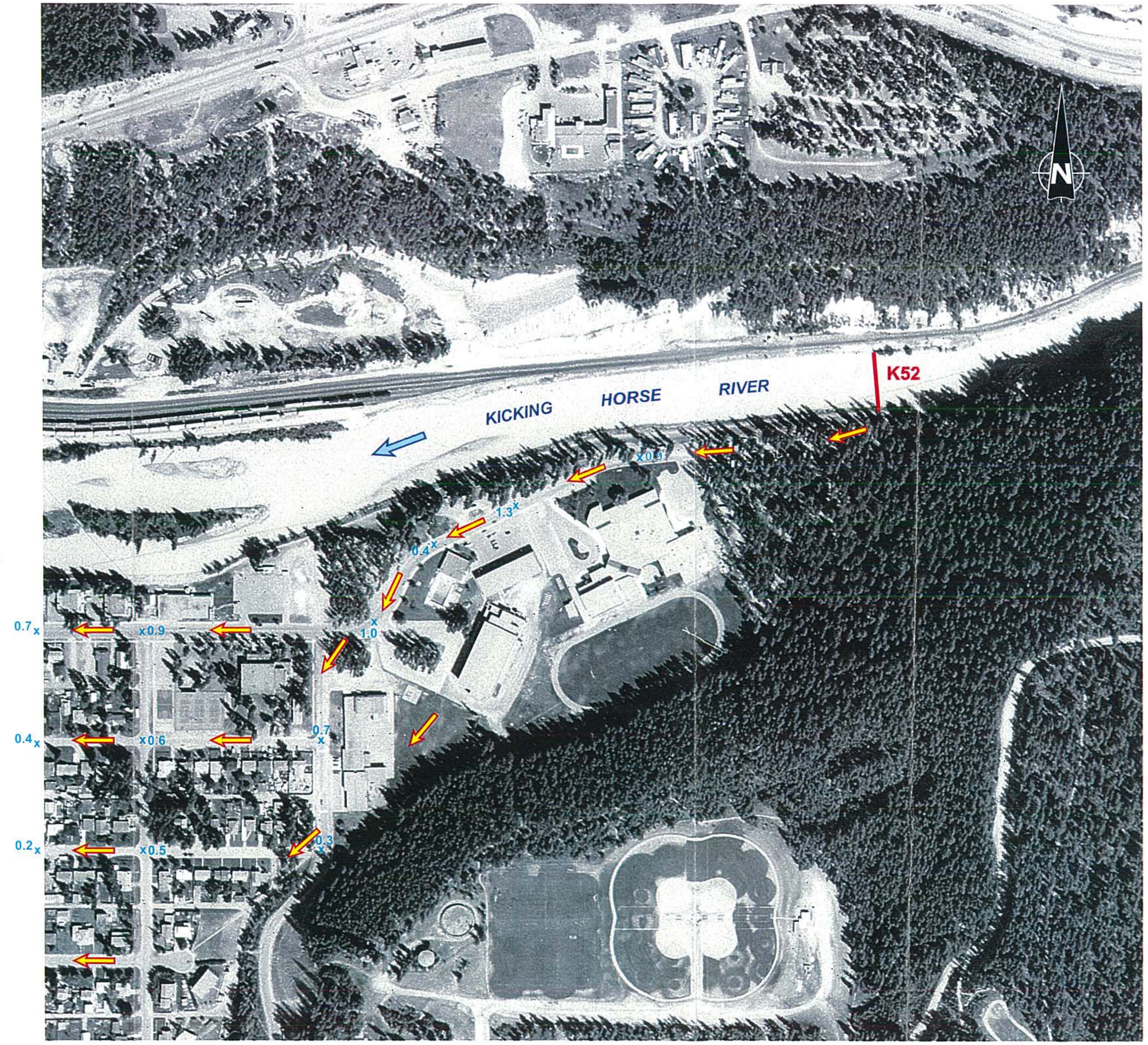


Dyke Section (K52) at Campground



Typical Breach Shape in Dike Profile

N.T.S.



1996 Airphoto

Scale 1 : 5000

Overland Flood Route

0.6 x Typical Flow Depth (m)

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DATE 03/08/19

FLOOD RISK MAPPING ASSESSMENT

HYPOTHETICAL DIKE BREACH SCENARIO AT CAMPGROUND

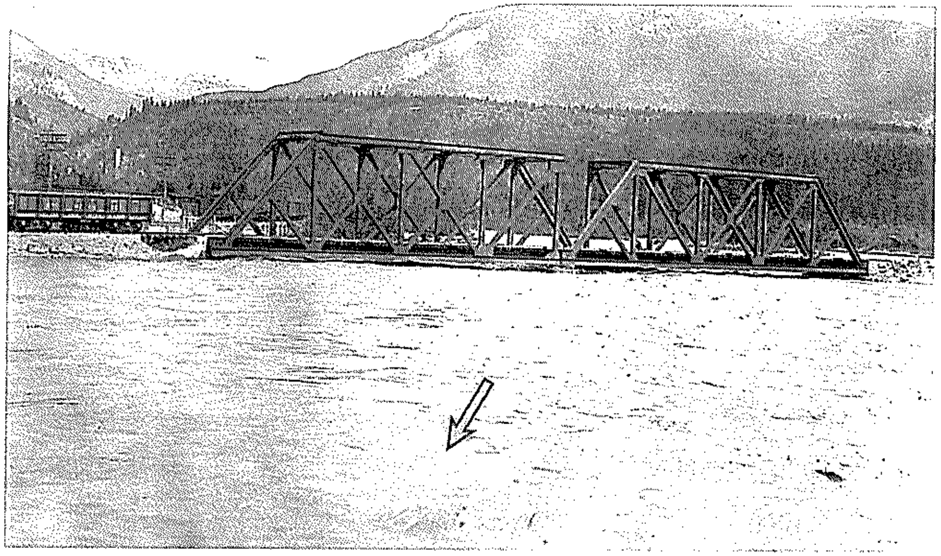
FILE (CDR) 785-03-08-19-A1

JOB NO. 785

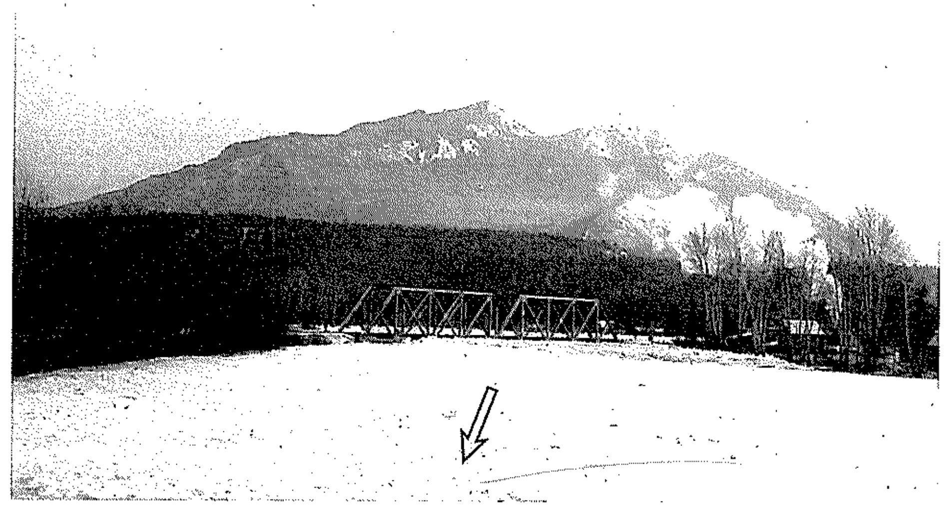
FIGURE 9

APPENDIX A

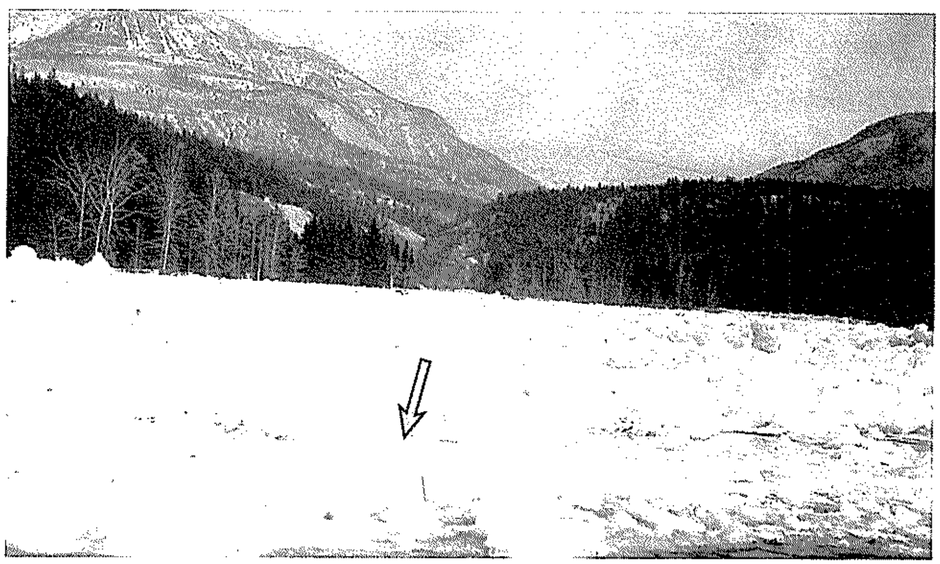
Historic Flood and Ice Event Photos



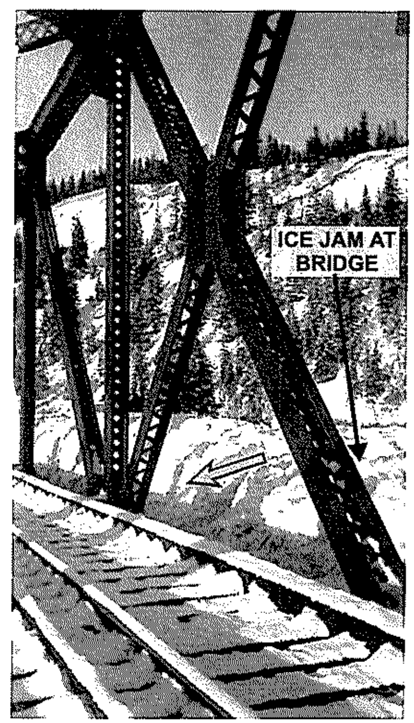
A1. Flood of 1916 (highest on record), peak estimated at 450 m³/s on June 19th. Looking upstream at former Kootenay Central Railroad bridge - Section K4 to K51.



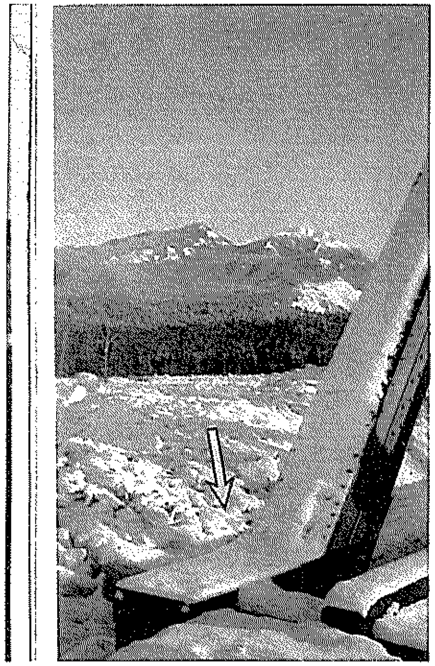
A2. Looking upstream at former Kootenay Central Railroad bridge - Section K4 to K51. High ice level - December, 1955.



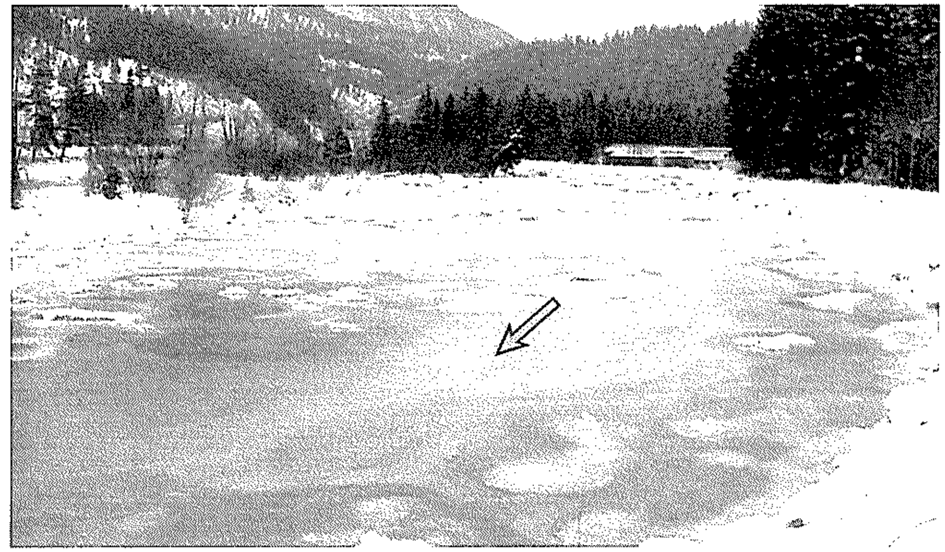
A3. December 1955. Looking upstream towards canyon in vicinity of Section K2.



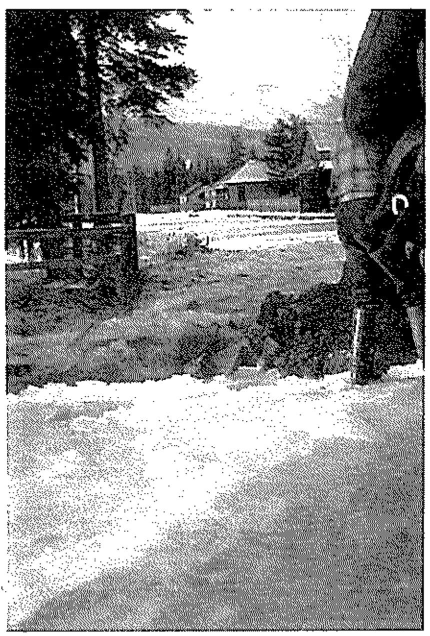
A4. December, 1955. Close-up of former railroad bridge upstream of current Highway 95.



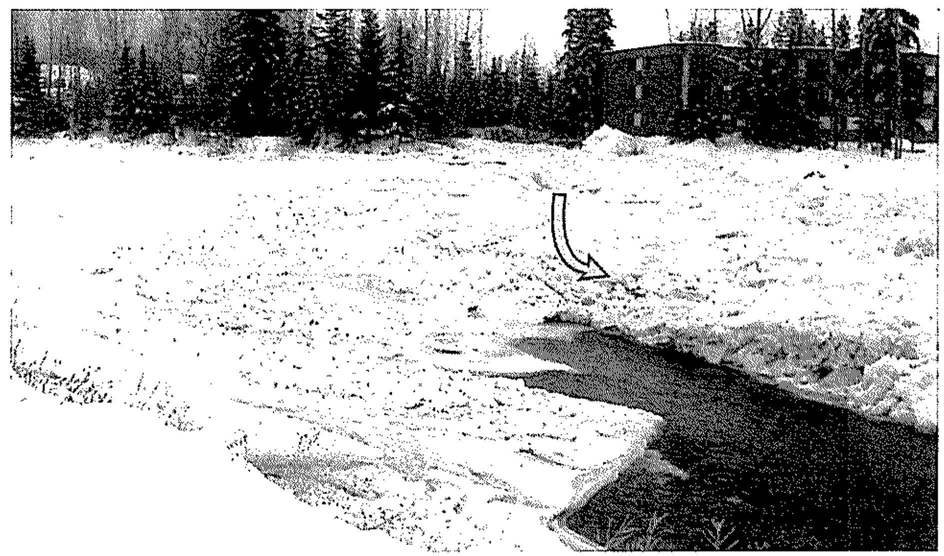
A5. December, 1955. Looking upstream from former railroad bridge between Sections K4 and K51.



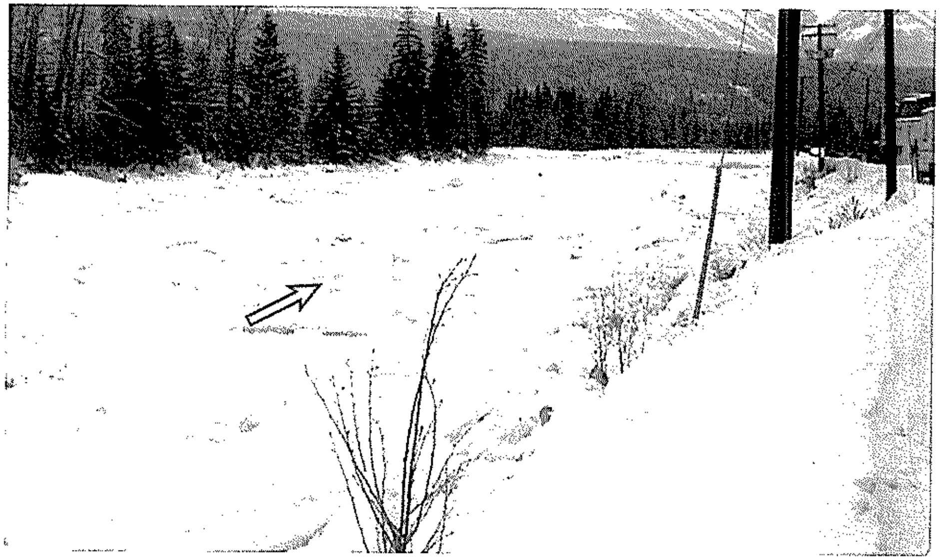
A7. December, 1988. Looking upstream towards Section K50. Potential jam location at head of island as shown in background



A6. December, 1955. Ice in channel by Kinsmen Park. Channel is now filled in.



A8. December, 1988. looking south (upstream) at Crichton Apartments on S8th Avenue. River discharge was 3.5 to 8 m^3/s at the time. Ice shove within 1.5 m of top of bank.



A9. December, 1988. Looking downstream from Section K11b. Gould's Island on the left.

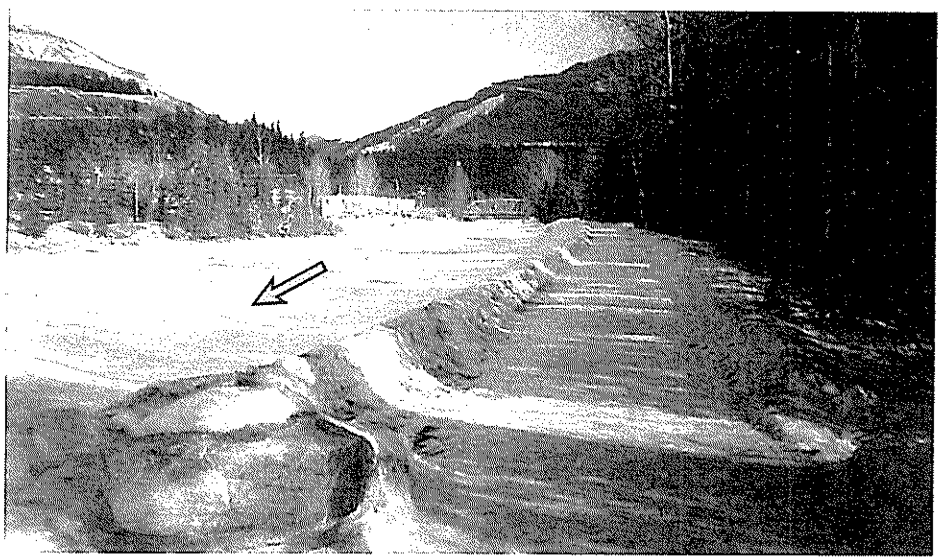
A10. 1995, looking upstream in vicinity of Section K6.

A11. 1995, looking at River Glen (Section K6 to K6a) from north bank. Ice shove approaching top of dike level in places.

A12. December, 1997. Looking downstream from right bank at planned pedestrian bridge site.

A13. December 1997. Looking upstream towards Highway 95 bridge. Ice shove within 2m of top of dike. Water level more than 1 m lower. River discharge only 8 to 13 m³/s at the time.

A14. December, 1998. Looking downstream from south bank at the campground. River flow 3 to 13 m³/s at the time.



A15. March, 2001. Looking upstream between Sections K5 and K6 at planned pedestrian bridge site. Highway 95 bridge in background.

A16. March, 2001. Looking downstream at planned pedestrian bridge site from left bank. Typical river ice level more than 2 m below top of dike.

APPENDIX B

HEC-RAS Input and Output Files

```
                    HEC-RAS Version 3.0.1  May 2002
                    U.S. Army Corp of Engineers
                    Hydrologic Engineering Center
                     609 Second Street, Suite D
                    Davis, California 95616-4687
                           (916) 756-1104

     X     X  XXXXXX   XXXX        XXXX      XX      XXXX
     X     X  X       X    X       X   X    X  X    X
     X     X  X       X            X   X   X    X   X
     XXXXXXX  XXXX    X       XXX  XXXX    XXXXXX    XXXX
     X     X  X       X            X  X    X    X        X
     X     X  X       X    X       X   X   X    X        X
     X     X  XXXXXX   XXXX        X    X  X    X   XXXXX
```

```
********************************************************************************
```

PROJECT DATA
Project Title: Kicking Horse River (metric)
Project File : khrmet.prj
Run Date and Time: 18/12/03 10:55:58 AM

Project in SI units

Project Description:
Kicking Horse River at Golden - Cross sections from 1987 and Nov 2002 surveys.

```
********************************************************************************
```

GEOMETRY DATA

Geometry Title: Golden Geometry (2003)
Geometry File : C:\Jobs\785 Golden\Data from Job 345\hec model\khrmet.g23

Reach Connection Table

```
*********************************************************************************
* River              Reach               *  Upstream Boundary  * Downstream Boundary *
*********************************************************************************
* Kicking Horse Ri Town of Golden     *                     *   Golden            *
* Columbia River   South of Golden    *                     *   Golden            *
* Columbia River   SW of Golden       *   Golden            *                     *
*********************************************************************************
```

JUNCTION INFORMATION

Name: Golden
Description: Confluence of the Kicking Horse and Columbia Rivers
Energy computation Method

```
    Length across Junction             Tributary
     River            Reach              River          Reach          Length   Angle
Kicking Horse RiTown of Golden   to Columbia River   SW of Golden        83
Columbia River   South of Golden  to Columbia River   SW of Golden       270
```

```
 3080.33 794.641

Manning's n Values         num=        3
     Sta    n Val       Sta    n Val      Sta    n Val
************************************************
    3000     .032 3010.01     .032 3080.33     .032

Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
       3010.01 3080.33                 495      390      385                .1        .3
Left Levee       Station= 3010.01       Elevation=  793.87

CROSS SECTION            RIVER: Kicking Horse Ri
REACH: Town of Golden      RS: 13

INPUT
Description: X-Section K50 (Nov/02)
Station Elevation Data     num=       65
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
       0 790.944    3.428 789.261    4.473 788.749    5.127 788.474    5.296 788.403
   5.423 788.398     7.84  788.31    9.062 788.366   12.215 788.512   12.436  788.52
   15.72  788.64   19.092 788.795   19.933 788.834   28.074  788.99   28.141 788.992
  28.153 788.991   28.303 788.983   34.856 788.631   35.359 788.604   37.473 789.995
  37.601  790.08   49.707 790.183   52.147 790.204   60.702 790.012   61.905 789.985
  62.224 789.196   62.339 788.911   64.149 788.743   69.189 788.277   72.461 788.367
  72.963 788.381   75.949 788.604    76.16  788.62   80.021 788.905   80.059 788.908
  80.465 788.928   83.329  789.07   86.686 789.156   88.215 789.195   94.643 789.212
  94.915 789.213   100.72 789.033  100.782 789.031  101.266 788.893  102.981 788.407
 105.085 788.105  105.567 788.036  105.629 788.035  108.308 787.988  110.507  788.01
 110.828 788.013  114.115 788.444  114.186 788.454  114.447 788.529   117.55 789.423
 120.725 790.884  121.042  791.03  121.148 791.029  124.706 791.005  125.687 790.999
 125.716 790.999  125.727 790.992   127.33 790.086   131.53 791.926  132.129 792.189

Manning's n Values         num=        3
     Sta    n Val       Sta    n Val      Sta    n Val
************************************************
       0     .032        0     .032 121.148     .032

Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
             0 121.148                 215      197      180                .1        .3
Right Levee      Station= 121.148       Elevation= 791.029

CROSS SECTION            RIVER: Kicking Horse Ri
REACH: Town of Golden      RS: 12

INPUT
Description: X-Section K3 - May 1987
Station Elevation Data     num=       48
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    3000  790.343004.499 790.4713006.002 790.8193007.599 790.8193009.199  790.81
3012.421 788.3593012.969  787.85 3014.25 787.622 3015.99   787.63016.499 787.509
3018.389 787.5093020.611 787.509 3022.86 787.5093024.969 787.509 3027.24 787.411
3029.179 787.3113031.072 787.3113033.041 787.311    3035 787.0613038.259 786.759
3041.651 786.9113044.339 787.3113047.979 787.6493049.622 787.8593050.569 788.262
3051.582 788.6093060.201 788.889 3072.78  788.393075.301 788.2013079.129 788.079
3082.312  787.71 3084.25 787.5613090.129 787.6313090.331 787.631 3093.54  787.53
```

```
 ************************************************
     .575     .032     .575     .032   64.402     .032

 Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
            .575   64.402                113      113      113                .1        .3

 CROSS SECTION            RIVER: Kicking Horse Ri
 REACH: Town of Golden      RS: 9

 INPUT
 Description: X-Section K10 - May 1987 - Upstream of Bridge
 Station Elevation Data     num=       52
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ********************************************************************************
     3000 790.7093020.989 790.7093023.272 790.7093023.299 790.7093023.659 790.709
 3023.671 789.1393023.991  789.133024.521 787.1613026.801 786.4393027.591 786.241
  3030.23 786.271 3033.62 786.2413038.009 786.3413038.469   786.5 3041.41  787.53
  3041.73 789.1213042.099  789.173042.139 790.6093042.669 790.2793042.669  790.59
 3045.629 790.5813057.812 790.749 3070.61 790.441 3072.11 790.5813072.509 790.581
 3072.561 788.399 3073.64 788.3993073.802 787.4813075.969 787.3113077.249   786.5
 3078.739 785.9513079.711 785.799 3082.01   785.53083.049   785.43085.122 785.299
 3087.149 785.101 3089.05     785 3091.09  784.753094.391   784.53097.429  784.25
 3099.719 784.1993103.541 784.2993107.421    784.9 3109.53 785.299 3113.87 785.439
 3115.891  786.183117.089 786.4113118.089 786.6713118.101 788.3713119.311 788.441
 3119.369 790.6213119.781 790.621

 Manning's n Values         num=        3
      Sta    n Val      Sta    n Val      Sta    n Val
 ************************************************
     3000     .0323072.509     .0323119.781     .032

 Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
       3072.5093119.781                 15       15       15                .1        .3

 BRIDGE                    RIVER: Kicking Horse Ri
 REACH: Town of Golden      RS: 8.5

 INPUT
 Description: Kicking Horse River Bridge - East of Golden
 Distance from Upstream XS =    1.329
 Deck/Roadway Width         =   8.199
 Weir Coefficient           =    1.44
 Upstream  Deck/Roadway Coordinates
     num=        8
      Sta Hi Cord Lo Cord      Sta Hi Cord Lo Cord      Sta Hi Cord Lo Cord
 ************************************************************************
     3020 790.581     786     3026 790.581  789.11 3040.63 790.581  789.11
  3041.73 790.581     786  3073.8 790.569     786  3073.8 790.569  788.82
 3117.089 790.569  788.82     3121 790.569 786.671

 Upstream Bridge Cross Section Data
 Station Elevation Data     num=       52
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ********************************************************************************
     3000 790.7093020.989 790.7093023.272 790.7093023.299 790.7093023.659 790.709
 3023.671 789.1393023.991  789.133024.521 787.1613026.801 786.4393027.591 786.241
  3030.23 786.271 3033.62 786.2413038.009 786.3413038.469   786.5 3041.41  787.53
```

```
 Number of Bridge Coefficient Sets =  1

 Low Flow Methods and Data
        Energy
        Momentum                 Cd    =       2
 Selected Low Flow Methods = Highest Energy Answer

 High Flow Method
        Pressure and Weir flow
            Submerged Inlet Cd              =
            Submerged Inlet + Outlet Cd =        .8
            Max Low Cord                    =

 Additional Bridge Parameters
        Add Friction component to Momentum
        Do not add Weight component to Momentum
        Class B flow critical depth computations use critical depth
            inside the bridge at the upstream end
        Criteria to check for pressure flow = Upstream water surface

CROSS SECTION           RIVER: Kicking Horse Ri
REACH: Town of Golden      RS: 8

INPUT
Description: X-Station K11b- Downstream copy of K11
Station Elevation Data    num=       51
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    3000 790.5813039.569 790.5813065.752 790.5813081.641 790.5813081.641 790.441
3082.189 790.5813082.189 789.191   3082.5  789.133082.589  788.38 3085.25  787.46
3087.959  786.28 3089.23  786.143091.721  786.113095.671 786.1313096.579 786.299
3099.499 787.469 3099.99 789.121 3100.41 789.149 3100.49  790.563101.011  790.56
 3102.02 790.541 3113.73 790.301 3129.04 790.5113130.589 790.5693130.939 790.569
3130.991  788.78 3132.14 788.6493134.621 787.701 3139.05 787.2413139.681 786.539
  3140.9 785.9393142.049  785.753143.189 785.7993143.701 785.701 3145.63   785.5
 3147.88  785.25  3150.9 784.951 3155.32 784.601 3158.85   784.43162.099   784.4
 3166.22   784.53168.381   784.5  3172.7  785.253175.199 785.369 3176.54 785.689
3176.751  788.38 3177.79 788.4413177.869 790.5813178.079  790.593179.951  790.59
 3191.04  790.59

Manning's n Values          num=        3
     Sta    n Val      Sta    n Val      Sta    n Val
***********************************************
    3000     .0323130.939     .0323177.869     .032

Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
      3130.9393177.869                    80      80       80                .1        .3

CROSS SECTION           RIVER: Kicking Horse Ri
REACH: Town of Golden      RS: 7.1

INPUT
Description: X-Station K11- Downstream of Bridge East of Golden
Station Elevation Data    num=       51
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    3000 789.7813039.569 789.7813065.752 789.7813081.641 789.7813081.641 789.641
```

58.065	785.454	62.323	787.773

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
2.572	.025	2.572	.025	62.323	.025

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
2.572	62.323	257	257	257	.1	.3

CROSS SECTION RIVER: Kicking Horse Ri
REACH: Town of Golden RS: 5.8

INPUT
Description: X-Section K6a (Nov/02)
Station Elevation Data num= 45

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
0	786.251	4.497	784.099	5.711	783.519	6.031	783.364	6.66	783.061
8.421	782.975	8.808	782.956	8.868	782.956	13.352	782.938	13.39	782.937
17.28	782.9	17.624	782.915	21.28	783.072	24.454	783.154	25.581	783.184
29.151	783.362	30.231	783.416	32.02	783.505	32.239	783.516	38.447	783.771
38.736	783.783	44.222	783.606	47.128	783.513	47.307	783.506	55.01	783.226
55.113	783.258	57.326	783.948	57.887	783.932	64.377	783.753	64.487	783.747
70.079	783.459	70.132	783.456	74.205	783.226	76.082	783.12	76.106	783.119
76.114	783.117	77.856	782.651	82.668	782.491	83.066	782.478	89.524	782.594
89.912	782.601	93.148	782.964	93.156	782.966	97.681	786.543	97.897	786.714

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
0	.025	0	.025	97.897	.025

Bank Sta: Left	Right	Lengths: Left	Channel	Right	Coeff Contr.	Expan.
0	97.897	117	117	117	.1	.3

CROSS SECTION RIVER: Kicking Horse Ri
REACH: Town of Golden RS: 5

INPUT
Description: X-Section K7 (Nov/02)
Station Elevation Data num= 32

Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
1.995	786.815	4.59	785.308	7.87	783.402	11.663	783.432	17.372	783.478
22.252	783.519	28.219	783.569	33.202	783.586	38.386	783.604	42.65	783.526
48.538	783.417	52.463	783.328	57.988	783.202	61.95	783.027	67.338	782.789
70.88	782.617	75.079	782.412	76.274	782.341	77.201	782.286	79.905	782.1
81.017	782.024	82.596	781.724	83.426	781.535	86.635	781.15	93.091	781.138
93.145	781.156	94.955	781.757	96.067	782.359	96.092	782.373	96.399	782.584
101.966	786.423	102.737	786.479						

Manning's n Values num= 3

Sta	n Val	Sta	n Val	Sta	n Val
1.995	.025	1.995	.025	101.966	.025

```
 Description: X-Section K8 (Nov/02)
 Station Elevation Data    num=       47
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ******************************************************************************************
     .987   784.82    5.538 783.178    7.033 782.638    9.281 782.126   10.575 781.831
   11.035    781.8   11.217 781.787   14.067 781.048   15.155 780.766   18.345 780.781
   18.899  780.784   22.789 781.084   23.515  781.14   27.832 780.986    28.37 780.967
   31.836  780.908   32.659 780.894   36.053  780.86   36.922 780.851    39.88 780.969
   40.644      781   43.175 781.565     43.9 781.727   45.096 781.947   45.448 782.011
   50.425  782.023   51.524 782.026   56.022 782.297   57.095 782.362   61.579 782.627
   62.858  782.702    67.27 782.743   68.854 782.758   73.385 782.789   75.062 782.801
   79.794  782.846    81.63 782.863   86.102 782.897   88.036 782.912   92.363  782.68
   94.432  782.569   98.643 782.465  100.638 782.416  105.089 782.304  106.957 782.257
  110.245  784.263  111.286 784.898

 Manning's n Values         num=        3
      Sta    n Val      Sta    n Val      Sta    n Val
 ************************************************
     .987     .025     .987     .025  111.286     .025

 Bank Sta: Left   Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
             .987 111.286               175      175      175                .1        .3

 CROSS SECTION           RIVER: Kicking Horse Ri
 REACH: Town of Golden       RS: 3

 INPUT
 Description: X-Section K53 (Nov/02 Extended with 1987 Data)
 Station Elevation Data    num=       28
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ******************************************************************************************
 2996.198 787.0212996.316 787.0252996.462 787.0042997.938   786.852998.967 786.575
 3005.728 784.7923008.734 783.9053017.137 781.4363022.145 781.7393025.196 781.921
 3028.567 781.8563037.463 781.6843042.146 781.4493051.688 780.9713060.975 780.654
  3063.73   780.563065.733 780.5213070.122 780.434   3073.1 780.6573078.257 781.046
 3081.503 782.1583089.073 784.732  3089.12 784.7343089.203 784.7363091.309  784.79
   3096.6 785.3513101.011 785.351  3106.01 785.351

 Manning's n Values         num=        3
      Sta    n Val      Sta    n Val      Sta    n Val
 ************************************************
 2996.198    .0252996.198     .025   3096.6     .025

 Bank Sta: Left   Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
       2996.198  3096.6                  14       50       80                .1        .3

 CROSS SECTION           RIVER: Kicking Horse Ri
 REACH: Town of Golden       RS: 2

 INPUT
 Description: X-Section K54 (Nov/02 Extended with 1987 Data)
 Station Elevation Data    num=       44
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ******************************************************************************************
     3000  787.189     3014 786.859   3015.2 786.8593020.352   786.92  3020.51 785.171
 3021.659 785.1713021.671   784.68  3028.42 783.3353033.066 782.4123035.822 782.158
 3041.262 781.6573041.711 781.6793043.399 781.7663046.512 781.9243047.339 781.967
```

```
 3151.58  788.45  784.54

Downstream Bridge Cross Section Data
Station Elevation Data    num=      44
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
*******************************************************************************
    3000 787.189    3014 786.859  3015.2 786.8593020.352  786.92 3020.51 785.171
3021.659 785.1713021.671  784.68 3028.42 783.3353033.066 782.4123035.822 782.158
3041.262 781.6573041.711 781.6793043.399 781.7663046.512 781.9243047.339 781.967
3051.419 782.1453053.301 782.2283056.423  782.243059.448 782.2533061.352 782.227
3065.401 782.1723066.115 782.1483068.555 782.0673071.557 781.9523077.142  781.75
3082.212 781.5143087.513 781.2083092.647 780.9943111.277 780.482 3119.86  780.98
3121.579  781.093123.319  781.193128.479 781.6693130.601 781.6693132.271 781.971
 3134.46  782.443141.409 784.2113150.431  784.54 3150.44  785.153151.501 785.171
 3151.58  786.923153.851 786.8593156.509 786.8813163.309 787.161

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
*************************************************
    3000   .0253020.352    .025 3151.58    .025

Bank Sta: Left   Right    Coeff Contr.   Expan.
      3020.352 3151.58             .1        .3

Upstream Embankment side slope               =       2 horiz. to 1.0 vertical
Downstream Embankment side slope             =       2 horiz. to 1.0 vertical
Maximum allowable submergence for weir flow =     .95
Elevation at which weir flow begins          =
Energy head used in spillway design          =
Spillway height used in design               =
Weir crest shape                             = Broad Crested

Number of Piers =  3

Pier Data
Pier Station     Upstream=3055.151    Downstream=3055.151
Upstream      num=        4
    Width   Elev    Width   Elev    Width   Elev    Width   Elev
*****************************************************************
   2.624 776.118   2.624 778.249   2.624 780.099   2.624     786
Downstream      num=        4
    Width   Elev    Width   Elev    Width   Elev    Width   Elev
*****************************************************************
   2.624 776.118   2.624 778.249   2.624 780.099   2.624     786

Pier Data
Pier Station     Upstream= 3087.42    Downstream= 3087.42
Upstream      num=        4
    Width   Elev    Width   Elev    Width   Elev    Width   Elev
*****************************************************************
   2.624 776.118   2.624 778.231   2.624 779.901   2.624     786
Downstream      num=        4
    Width   Elev    Width   Elev    Width   Elev    Width   Elev
*****************************************************************
   2.624 776.118   2.624 778.231   2.624 779.901   2.624     786

Pier Data
```

```
CROSS SECTION           RIVER: Kicking Horse Ri
REACH: Town of Golden      RS: 1

INPUT
Description: X-Section K9 (Nov/02 Extended with Apr/02 Data)- Downstream of
             Railway Bridge
Station Elevation Data    num=      61
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
       0  784.181     1.09  783.611    1.968  783.165    3.736  782.651    6.039  781.632
   6.345  781.638    6.349  781.638    6.354  781.639    7.257  781.614    8.537  781.619
  18.037  781.659   22.148   781.43   22.702  781.399   22.881  781.403   25.914   781.46
  26.578  781.637   29.971  782.542   32.103  782.565   44.563  782.706   46.987  782.714
  57.611  782.749   64.382  782.566   65.296  782.542   67.671  782.192   67.983  782.145
  68.675  782.176   72.902  782.362   80.912   782.25    83.22  782.218   94.727  782.038
  95.585  782.024  104.196   781.97  108.127  781.945  108.331  781.925   112.17   781.56
 114.688  781.249  116.145  781.182  121.738  781.097  121.861  781.096  127.758  781.074
 127.909  781.073  134.582  780.961  134.644  780.959  137.897  780.957  141.309  780.954
 141.368  780.953  147.453  780.845  147.528  780.839  151.396  780.571  151.551  780.565
 156.744  780.328  156.807  780.339    158.1  780.562  158.788  780.825  160.506  781.554
 160.672  781.642  164.875  783.901  165.074  784.007  165.127  784.008  165.128  784.008
 165.506   784.01

Manning's n Values         num=       3
     Sta    n Val      Sta    n Val      Sta    n Val
***********************************************
       0     .025        0     .025  165.506     .025

Bank Sta: Left    Right     Lengths: Left Channel    Right       Coeff Contr.     Expan.
             0  165.506                  0       0        0                .1         .3

CROSS SECTION           RIVER: Columbia River
REACH: South of Golden     RS: 2

INPUT
Description: X-Section C6b - South of Golden
Station Elevation Data    num=      29
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
********************************************************************************
    3000   783.763000.079    781.9 3001.25   780.77 3001.71  780.9313003.249   780.23
3005.371   779.233008.361  778.4293009.281  778.3313011.522  778.331 3015.24   778.13
 3020.01   777.883021.601   777.88 3024.46  777.831 3027.13  778.029 3035.01  778.231
 3038.92  778.4293043.458   778.38  3047.4   778.383049.311  778.4813050.569  778.529
3052.389   778.733053.849  778.9293060.021  779.9193062.981    780.73064.542  781.309
3065.752  781.9493065.971  782.2113067.879    782.83072.963  782.781

Manning's n Values         num=       3
     Sta    n Val      Sta    n Val      Sta    n Val
***********************************************
    3000     .022     3000     .0223067.879     .022

Bank Sta: Left    Right     Lengths: Left Channel    Right       Coeff Contr.     Expan.
          30003067.879            109.999     120  124.998                 .1         .3

CROSS SECTION           RIVER: Columbia River
REACH: South of Golden     RS: 1
```

```
              0 265.462                 354       354       354                .1         .3

 CROSS SECTION            RIVER: Columbia River
 REACH: SW of Golden        RS: 3

 INPUT
 Description: X-Section C7
 Station Elevation Data     num=       40
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ********************************************************************************
     3000  783.1013001.341 781.449 3003.06 780.069 3004.24 779.959 3008.05   778.56
 3011.241   777.563012.439 777.661 3020.15  778.063026.329 778.4593028.819 778.261
 3031.041 778.161 3037.92 778.761  3040.1 778.959 3043.87 778.9593045.851 778.859
 3052.691 778.4593053.959 778.3593056.339  778.663057.979  778.66 3061.04  778.56
 3063.722 778.761 3066.45 778.9593073.481 780.3893076.691 780.971 3080.58 781.111
 3082.369  781.48 3082.89  781.65 3086.99 781.7513089.111 781.2913091.321 781.169
 3094.202  781.263096.411   781.93098.691 781.8793104.979 783.2293109.938 782.059
 3115.669 782.291 3117.72  783.473126.501 783.5743126.949  783.583129.031 782.729

 Manning's n Values          num=        3
      Sta    n Val      Sta    n Val      Sta    n Val
 ************************************************
     3000     .022     3000     .0223126.501     .022

 Bank Sta: Left    Right     Lengths: Left Channel    Right     Coeff Contr.    Expan.
          30003126.501                   156      156      156                .1         .3

 CROSS SECTION            RIVER: Columbia River
 REACH: SW of Golden        RS: 2

 INPUT
 Description: X-Section C56 - Upstream of Golden
 Station Elevation Data     num=       49
      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
 ********************************************************************************
     3000  787.173022.001 785.4213023.991  785.143024.241  783.793024.841  783.79
 3024.841 783.4493027.898 781.5013030.051 780.6293032.839  779.243035.951 778.289
  3039.38   777.49 3040.81 777.289 3042.16 777.2893046.961 777.7093050.899  777.74
 3054.541 777.9413059.061  778.09  3061.6 778.191 3063.63 778.2893066.831 778.849
 3068.751 778.889 3072.43  778.74 3075.56 778.7913079.458 778.791 3082.29  778.74
 3083.781 778.7913084.101 778.8893087.271  779.413090.041   779.3 3093.58 779.599
  3097.42 779.8613102.139 780.0693104.141 780.1513106.951  780.123110.061 780.291
  3114.55 780.389 3116.37  780.483119.601 780.4713121.481 781.0293122.841  781.48
  3124.49 782.839 3126.05  783.19 3126.23  783.793127.059 783.839 3127.15  784.97
  3128.47 784.869 3143.07 784.641 3151.47  784.043179.469 783.909

 Manning's n Values          num=        3
      Sta    n Val      Sta    n Val      Sta    n Val
 ************************************************
     3000    .0223023.991     .022 3128.47     .022

 Bank Sta: Left    Right     Lengths: Left Channel    Right     Coeff Contr.    Expan.
      3023.991 3128.47                    6        6        6                .1         .3
 Left Levee       Station=3023.991      Elevation=  785.14
 Right Levee      Station= 3128.47      Elevation= 784.869

 BRIDGE                   RIVER: Columbia River
```

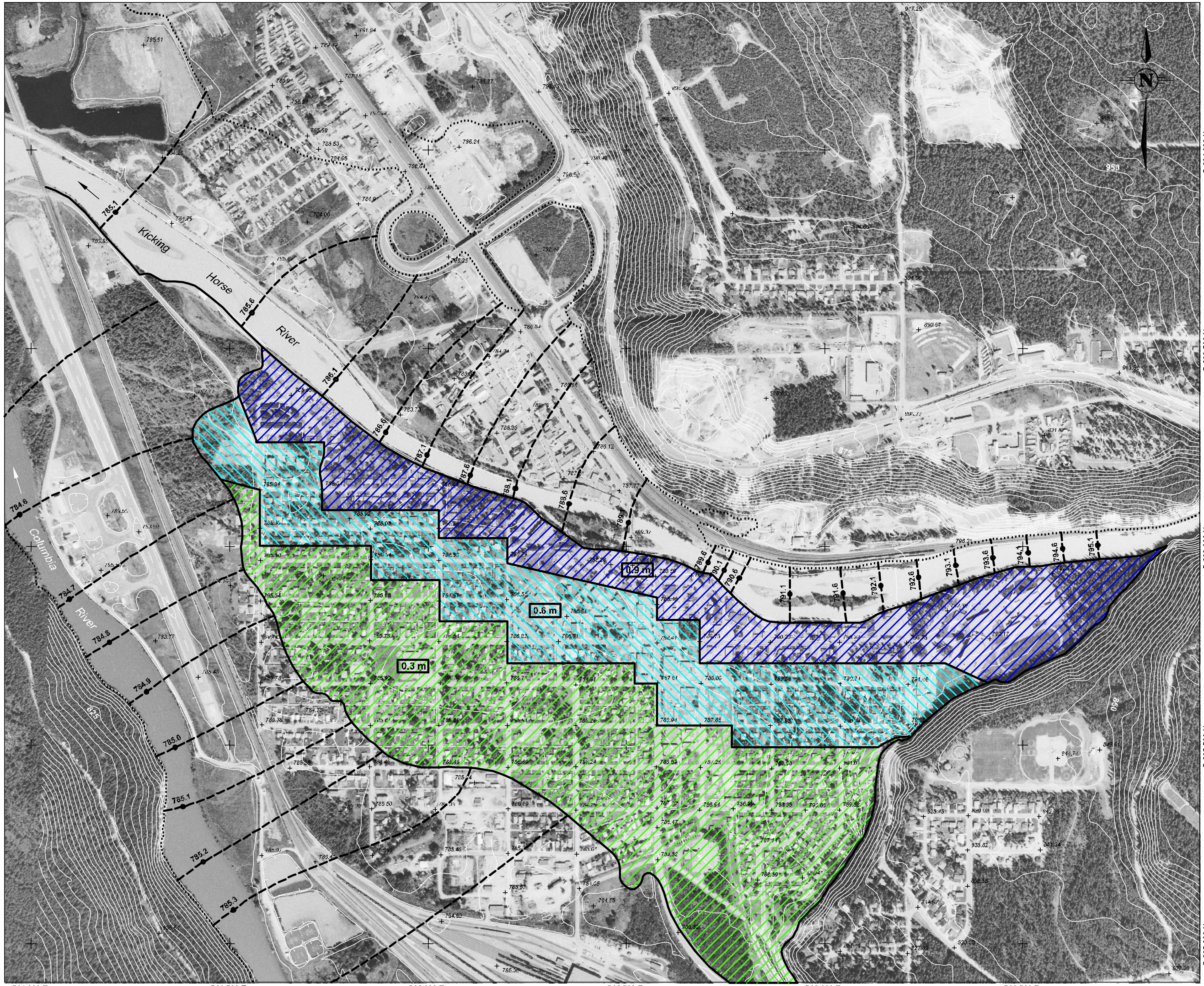
```
3121.161  780.453122.719 780.8493123.849 781.4893125.861 782.839 3127.73 783.049
3127.769  783.793128.659 783.7993128.699     7853130.198    784.93144.701 784.491
  3170.2 783.879

Manning's n Values        num=        3
     Sta    n Val     Sta    n Val     Sta    n Val
************************************************
    3000    .0223025.701    .0223130.198    .022

Bank Sta: Left   Right    Coeff Contr.   Expan.
      3025.7013130.198             .1        .3
Left Levee      Station=3025.701      Elevation=  785.15
Right Levee     Station=3130.198      Elevation=   784.9

Upstream Embankment side slope                 =    3.73 horiz. to 1.0 vertical
Downstream Embankment side slope               =    3.73 horiz. to 1.0 vertical
Maximum allowable submergence for weir flow =      .95
Elevation at which weir flow begins            =
Energy head used in spillway design            =
Spillway height used in design                 =
Weir crest shape                               = Broad Crested

Number of Piers =  4

Pier Data
Pier Station     Upstream=    3040    Downstream=    3040
Upstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21
Downstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21

Pier Data
Pier Station     Upstream=    3065    Downstream=    3065
Upstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21
Downstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21

Pier Data
Pier Station     Upstream=    3090    Downstream=    3090
Upstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21
Downstream      num=        2
    Width   Elev    Width   Elev
********************************
      1 780.919       1  789.21
```

```
Left Levee       Station=3025.701      Elevation=  785.15
Right Levee      Station=3130.198      Elevation=   784.9

CROSS SECTION           RIVER: Columbia River
REACH: SW of Golden        RS: 0

INPUT
Description: X-Section C57.0 : Downstream  X-Section C57
Station Elevation Data    num=       56
     Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev      Sta     Elev
*********************************************************************************
3000.601 786.881 3028.24 785.381 3030.84  785.153031.138 783.7813031.641 783.812
3031.641  782.54 3033.51  781.483035.189  780.513036.921  779.733037.149 780.181
3038.201 779.1113040.959 778.4813045.601 777.4813047.451 777.1793050.539 777.231
3051.749  777.283052.209 777.0813053.709 777.0393054.888  777.28 3055.91 777.481
3059.131 777.429 3065.63 777.981 3069.33 778.481 3070.57 778.0813073.552 777.911
  3076.2 778.2793081.641 778.6793083.491 778.6793086.691  778.983087.819 779.081
3090.081 779.029 3091.16  778.98 3091.27  778.983093.641  778.953096.811 779.029
 3101.31 779.6213104.931 779.8893105.659  779.84 3107.61 780.1113109.771 780.181
3113.041  780.193114.361  780.133118.881  780.163122.341 780.3313123.301 780.501
  3126.3  780.453127.861 780.8493128.989 781.489    3131 782.8393132.872 783.049
3132.911  783.793133.801 783.7993133.841     785 3135.34   784.93152.739 784.491
3183.341 783.879

Manning's n Values          num=       3
     Sta    n Val      Sta    n Val      Sta   n Val
************************************************
3000.601     .022 3030.84     .022 3135.34     .022

Bank Sta: Left    Right     Lengths: Left Channel    Right      Coeff Contr.    Expan.
       3030.84 3135.34                  0       0        0                .1        .3
Left Levee       Station= 3030.84      Elevation=  785.15
Right Levee      Station= 3135.34      Elevation=   784.9
```

APPENDIX C

Town of Golden South Overbank Flood Risk Map

(to be provided, if required, depending upon Draft review)



LEGEND

+ 913.56	Ground Spot Elevation
788.0	Flood Level (with freeboard)
― ― ―●― ― ―	200 Year Frequency *
··················	200 Year Floodplain Limit **
0.6 m	Potential Flood Depth

* Columbia River levels are from 1979 Floodplain Mapping (BC MOE&P). Where Columbia River and flood depth lines merge the higher of the two levels will dictate. Kicking Horse River levels are based upon Hydroconsult 2004 Report.

** Limits shown are from the 1979 Floodplain Mapping (BC MOE&P).

NOTES

GENERAL

Prepared by: Hydroconsult EN3 Services Ltd.

Survey control supplied by: BC Ministry of Transportation and Highways

Compiled by The ORTHOSHOP, Calgary, December 1996, based on Photography dated 1996/08/01 at a scale of 1:40 000.
a) Contour interval - 5 m; Spot elevations shown to 0.01 metres, with accuracy to ± 0.03 metres.
b) Coordinates: LSR, WGS84

FLOODPLAIN DATA

a) Floodplain limits and flood levels include allowance for freeboard equal to 0.6 m.

b) Position of floodplain boundary not established on ground by legal survey.

c) 200 year floodplain limit shown on north side of Kicking Horse River are based upon contour data from 1976 Floodplain Mapping (BC MOE&P).

DEVELOPMENT WITHIN POTENTIAL FLOOD DEPTH AREAS

1. This mapping recognizes the protection provided by the south side dyke along the Kicking Horse River.

2. Recognizing the risk of dyke overtopping and/or breaching all new development within the hatched area shall be constructed considering flooding to at least a depth of 0.3 m above the crown of the adjacent municipal road.

3. The areas shown with 0.6 m and 0.9 m are at risk of flooding to depths of 0.6 m and 0.9 m respectively. Therefore, new development in the 0.6 m and 0.9 m designated areas should recognize the additional flood depth risk in these areas.

100 0 100 200 300 400 500 Metres

DRAFT FOR REVIEW

TOWN OF OPPORTUNITY GOLDEN

Hydroconsult

FLOOD RISK MAPPING ASSESSMENT

FLOODPLAIN MAPPING

DRAWN	DESIGN	APP	SCALE
Gary Evenson	Dave Cooper	Dave Cooper	As shown
DATE 04/05/06	FILE (DWG) 785-04-05-06-A1	JOB NO 785	FIGURE 1