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Revised Design Flood Profile for Lower Fraser River

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Abstract: The Lower Fraser River has undergone significant change since the major floods of 1894 and 1948. Over 250 km of dikes have been constructed, side-channels and sloughs have been blocked off, river training structures have been installed to narrow and deepen the channel and the lower 40 km reach has been dredged to improve navigation. Most flood control works were designed on the basis of studies carried out in 1969, which relied mainly on historic high water marks and staff gauge rating curve extensions. An updated design flood profile was produced based on detailed surveys of the river channel and floodplain collected in 2005. A MIKE11 hydrodynamic model was calibrated to known recorded floods and then used to calculate water levels that would result from a re-occurrence of the 1894 flood of record, estimated to have had a flow of 17,000 m^3/s at Hope. Results of the modelling show there is a serious flood threat along the lower 120 km reach, with the potential for several dikes to be overtopped.

1. Introduction

The Fraser River, with a drainage area of 233,000 km^2, is the largest river in British Columbia and flows from the Rocky Mountains to the Pacific Ocean. At Hope, roughly 180 km from the ocean, the river enters the Fraser Valley. From Hope, downstream to near Mission, the river has a gravel bed and anabranching channel pattern. Just above Mission, the channel slope reduces and the river abruptly transitions to a sand bed river with a single, irregularly meandering channel containing several major islands (Figure 1). The Fraser Valley is largely developed and protected by a system of dikes with a total length of over 250 km. The diking standard varies, but most dikes were built to a design profile developed in 1969, which was based on historic high water marks, extensions to staff gauge rating curves and some limited hydraulic computations. Fraser River floods are snowmelt generated and typically occur in May or June. However, near the ocean, the most severe flood conditions occur in the winter from a combination of high tide levels and storm surges. The freshet condition governs throughout most of the river and only in the lower 28 km of the river is the ocean condition higher.

There have been two major floods since non-native settlement in the valley, in 1894 and 1948. The magnitude of the 1894 flood was greater than that of 1948. However, there was substantially less development on the floodplain in 1894 and damages were less extensive. Since 1948, over $300 million have been spent by federal, provincial and municipal governments to construct and maintain dikes and flood control structures to protect communities within the 55,000 ha floodplain of the Lower Fraser Valley. An estimated 300,000 people presently live on the floodplain and the direct damage from another major flood could exceed $2 billion if dikes failed. Indirect economic losses from disruption of commerce would likely far exceed direct costs.

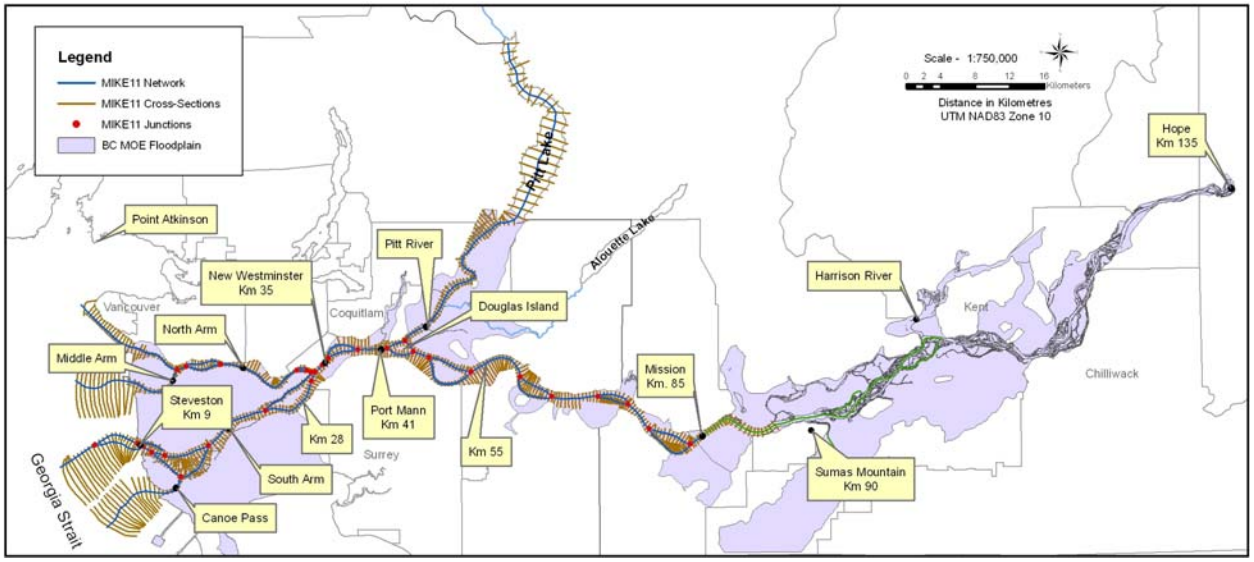

Figure 1: Location map and model extents.

In 2003, the Fraser Basin Council and BC Ministry of Environment initiated a multi-year project to develop a hydraulic model of the river. The main purpose of the project was to develop a design flood profile for the lower Fraser River corresponding to current channel and floodplain conditions. Specific project objectives included:

Updating the dike design profile and assessing the adequacy of existing diking;

Improving the understanding of sedimentation and dredging effects on flood levels;

Developing a flood level forecasting model for spring freshet floods; and,

Providing a tool for land use planning and flood-proofing practices.

The one-dimensional hydraulic modelling software, MIKE 11, developed by Danish Hydraulic Institute was used for the project. Two flood conditions were assessed:

A Fraser River spring freshet flood with the same magnitude as the 1894 flood of record combined with spring high tide conditions.

A winter storm surge and high tide condition combined with a Fraser River winter flow.

Design profiles were developed for both conditions and the two profiles then overlaid and the higher of the profiles used to develop the overall design flood profile for the river.

The original study area covered a 95 km reach of the lower Fraser River from Sumas Mountain to the Strait of Georgia, encompassing the North, Middle and South Arms, including Canoe Pass, as well as Pitt River to Pitt Lake inlet. These investigations complemented earlier hydraulic studies in the upstream gravel-bed reach between Hope and Sumas Mountain (Shumuk et. Al. 2000). As a result of model findings, the study area was later extended upstream another 25 km to Harrison River confluence.

This paper describes some of the challenges in developing, calibrating and verifying the numerical model of the Lower Fraser River. It also discusses the major changes that have occurred along the river over the last century and assesses how these changes have affected channel hydraulics and the flood hazard.

2. History of Floods

First Nation legends describe a devastating flood in the Fraser Valley in historic times (Keller 1976). However, the 1894 flood is the largest flood since European settlement and a water level of 7.92 m was recorded at Mission. Discharge measurements were not taken in 1894 at either Hope or Mission, locations of present gauging stations. Based on high water marks and extension of the Water Survey of Canada stage-discharge rating curve, the Fraser Basin Board (1958) estimated the magnitude of the flood at Hope to be 17,000 m^3/s. The magnitude of the discharge 130 km downstream, at New Westminster, was reported (Morton 1949) to be substantially lower (13,900 m^3/s), possibly reflecting overbank spills and storage on the floodplain between Hope and New Westminster. A recent assessment of floodplain conditions in 1894 and analysis of stage-discharge rating curves by Water Survey of Canada suggests that the actual flow reaching Mission in 1894 was approximately 16,500 m^3/s. These recent assessments suggest that considerable attenuation occurred over the 150 km long floodplain downstream of Hope in 1894. Some low berms and dikes had been built by 1894 but they failed during the flood and essentially the entire floodplain was inundated.

The 1948 flood had a published discharge of 15,200 m^3/s at the Hope gauge and a water level of 7.61m at Mission. McNaughton (1951) estimated the peak discharge in 1948 at Mission to be 15,800 m^3/s, or virtually the same as the flow at Hope. Based on a comparison of the recorded discharges at Hope and Mission during the common period of record (1965 to 2005), the expected discharge at Mission in 1948 would have been around 17,200 m^3/s. The difference between McNaughton's estimate and the expected value represents the effects of spills and overbank storage in 1948 as well as uncertainties in McNaughton's estimate and the year-to-year variability of inflows between Hope and Mission.

The flood of 1950 produced the fourth highest recorded water level at Mission of 7.45 m and was exceeded only in 1882, 1894 and 1948. The discharge was not measured at Mission, although it was estimated by McNaughton (1951) to have reached 14,500 m^3/s. The published discharge at Hope in 1950 was reported by Water Survey of Canada to be 12,600 m^3/s.

The flood of 1972 is the largest event in recent times, with a maximum daily discharge at Hope and Mission of 12,900 m^3/s and 13,700 m^3/s respectively. However, based on the historic water level data, 1972 with a Mission water level of 7.15 m, was probably only the fifth largest event over the period of record.

Within the past ten years, the three largest floods occurred in 1997, 1999 and 2002. These floods were considerably smaller but since they correspond to present channel conditions they were used for model calibration and verification. Recorded and estimated flow and water level records are summarized in Table 1.

Table 1: Flow and Water Level Records

Year	**Mission Water Level** (m GSC)	**Mission Flow** (m^3/s)	**Hope Flow** (m^3/s)
1894	7.92	16,500 estim.	17,000 estim.
1948	7.61	15,800 estim.	15,200
1950	7.45	14,500 estim.	12,600
1972	7.15	13,700 measured	12,900
1997	6.39	12,200	11,400
1999	6.30	11,800	11,100
2002	6.09	11,300	10,800

The adopted freshet design discharge for modelling is based on the 1894 flood of record, estimated to have had a peak discharge of 17,000 m^3/s at Hope. To account for inflow from tributaries, flow is estimated to increase to 18,900 m^3/s at Mission and 19,650 m^3/s at New Westminster. The adopted design discharge assumes containment of the river by the existing dike system downstream from Hope under current and future floodplain conditions. Due to variations in tributary flows and flow attenuation from overbank spilling and floodplain storage the actual 1894 flows at Mission and New Westminster may have been considerably less. For this reason, the 1894 historic flood profile is not directly comparable to the computed design flood profile.

3. Modelling Challenges

Some of the key physical characteristics of the river channel in the study reach that affect model formulation and schematization are summarized in Table 2.

Table 2: Fraser River Characteristics

Features	Hydraulic Effects
Low gradient	Backwater effects extend long distances upstream.
Large tidal influence	Unsteady tidal influence extends 85 km upstream to District of Mission.
Flow stratification	Saltwater wedge present in estuary, shifts downstream during high flows.
Effect of dikes	Overbank flow component is now small. Large spills in 1894 and 1948.
Varying roughness	Complex changes in channel resistance due to growth of bed forms.
River training structures/islands	Trifurcation and other structures induce complex head losses and alter flow splits in distributary channels.
Effects of dredging	Long-term changes in bed levels from dredging over the last 50 years affect flood levels; make it difficult to calibrate models with historic data.

To properly represent the tidal nature of the river and unsteady component of flow the high order fully dynamic version of MIKE11 was used. The branched nature of the channel and frequent large islands required that junctions be included at all islands, outlet arms and at Pitt River. Figure 1 shows the adopted schematic representation of the channel/floodplain network. The model is highly sensitive to where junctions are located within the network and careful adjustment of the junctions was required to eliminate or minimize water level instabilities. For optimum performance, it was found that the computational time step had to be kept at a maximum of two seconds.

Large dunes are known to form in the Lower Fraser River when flows exceed approximately 6,000 m^3/s. Since channel roughness varies with dune height and wave length, the roughness is expected to vary throughout the flood hydrograph in a complicated fashion. Under these conditions a model should ideally be calibrated for a flow approaching the design discharge. However the 1894 channel was very different from today's and any calibration to known historic high water marks would be meaningless. Similarly, the river has substantially changed since 1948 as a result of ongoing diking, dredging and construction of river training structures. These channel changes made it impossible to compare historic flood levels with recent values. To assess what the channel roughness may have been during the 1948 flood, a separate historic model was developed using bathymetry from 1950 for the Mission to Douglas Island reach.

4. Model Development and Calibration

Up-to-date channel bathymetry from Mission to Georgia Strait was provided by Public Works and Government Services Canada and floodplain topography was based on recent LiDAR surveys. A cross-section spacing of 400 m was selected and over 600 cross-sections were represented in the model. The spacing was reduced to 200 m in the narrower side channels. Cross-sectional data was manipulated through a GIS interface. A total of 27 bridges were modelled and detailed control structure designs were incorporated as necessary. Model boundary conditions were flow at the upstream end at Sumas Mountain, assumed to equal flow at Mission, flow at the upstream end of Pitt River and tidal ocean levels at the downstream end of each outlet. Tributary inflows for Stave, Alouette and Coquitlam Rivers were included as point source flows.

Water levels are recorded along the river at multiple locations with both continuous recorders and staff gauges during recent high flows. The model was initially calibrated to 2002 recorded water levels and flows. This flood was selected because of the high quality continuous water level records that were available during the freshet. The model was then verified using the freshets of 1999 and 1997. These three floods had peak discharges in the 11,300 to 12,200 m^3/s range at Mission. Manning roughness values were found to vary between 0.025 and 0.033 and average 0.03 in the channel over the flow conditions observed in 1997 through 2002.

A second model was developed using bathymetric surveys from 1950 to assess hydraulic characteristics and channel roughness during some of the large historic flood events (1948,1950 and 1972). The overall best-fit channel roughness was estimated to be 0.027 during the 1948 flood, 0.028 in 1950 and 0.029 in 1972 for the reach between Douglas Island and Mission. Approximate error bounds were estimated for the roughness in 1948, based on the uncertainty in the estimated discharge at Mission. This analysis indicated the actual Manning roughness could have ranged between a low of 0.026 and a high of 0.029. These results suggest a weak trend of lowering resistance with discharge (Figure 2).

Channel roughness along sand bed rivers is expected to vary with changing flow conditions due to the formation of sand dunes on the river bed. During very high flows the roughness may decrease substantially if the dunes wash out and flat bed conditions develop. The highest flow observations of dunes in the Fraser River were made during the 1950 freshet, (Pretious and Blench 1951). Their surveys in the estuary showed dunes of up to 4.5 m in height and up to 150 m in wave length near the peak of the flood. Measurements were also made at Mission in 1984 and 1986 at flows of up to 12,300 m^3/s (McLean unpub). During the highest flows the dunes had a maximum height of 2.8 m and a wave length of 44 m. There was no evidence from these studies that the dunes would wash out during high flows, which would lead to a rapid decline in roughness.

A number of methods have been developed to estimate alluvial channel roughness (Einstein-Barbarossa, Engelund-Hansen and van Rijn). The equations were tested against the measured hydraulic geometry data collected by Water Survey of Canada at the Mission gauge (approximate channel width 555 m, average flow depth 14 to 15 m). All relations were found to overestimate velocity and underestimate channel roughness, particularly at flows less than 10,000 m^3/s, with the Engelund-Hansen and van Rijn method providing reasonably good agreement with the measurements. However, the equations are quite sensitive to the assumed sediment size and energy slope. Figure 2 compares estimated Manning roughness values from the model calibration runs and predicted channel roughness from the Engelund-Hansen equation. The results provide additional support that the roughness decreases gradually when discharge exceeds approximately 12,000 m^3/s. Based on these results, it was decided to specify a variable roughness (from 0.03 to 0.027) over the range of flow between 12,000 m^3/s and 15,500 m^3/s, and to maintain a value of 0.027 for flows greater than 15,500 m^3/s.

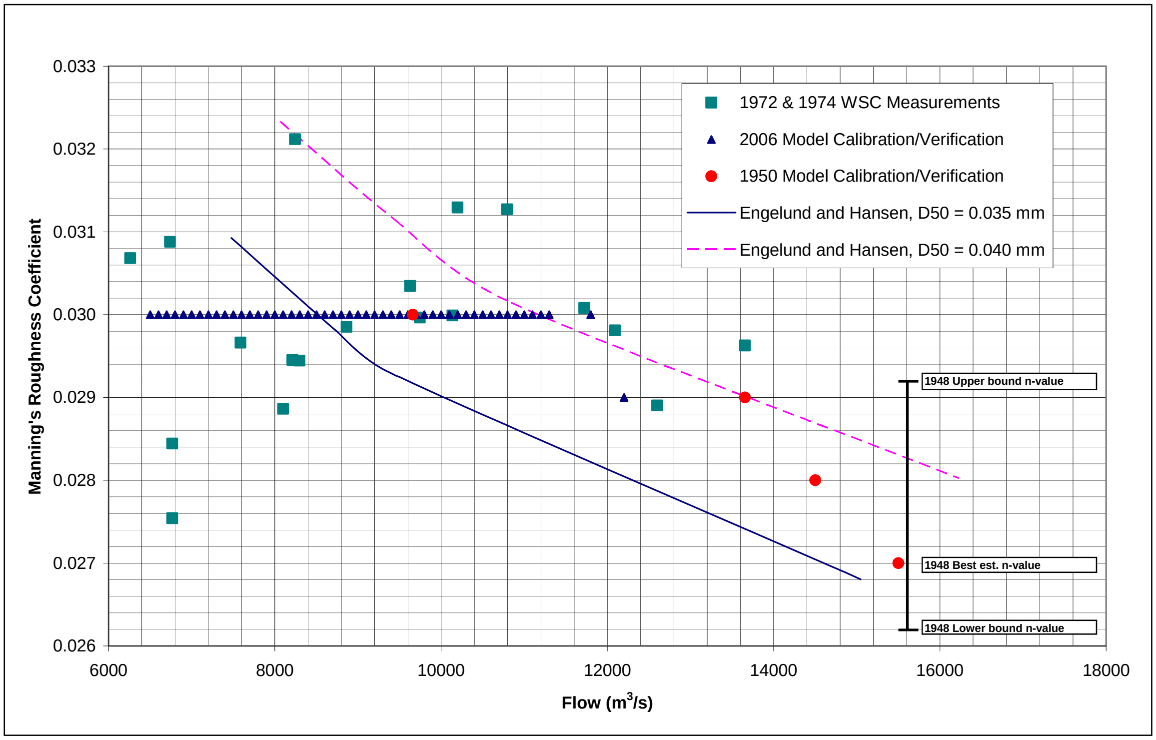

Figure 2: Relation between roughness and discharge at Mission.

5. Freshet Design Flood Profile

Following successful model calibration and verification, the model boundary conditions were set to design values. A design inflow of 18,900 m^3/s was used at the upstream end of the model, corresponding to the 1894 flood at Hope, plus local inflows between Hope and Mission. During the Fraser freshet, high tide levels are common (since Large tides occur in June around the time of the peak freshet) but storm surges are minimal. At the four outlet arms, the 2002 calibration tide levels were used as the downstream boundary condition (maximum tide at Point Atkinson of 1.84 m GSC). The levels roughly correspond to a two-year return period summer high tide.

Some adjustments had to be made to the calibrated model to accommodate the design flow. All standard, non-standard and other types of dikes, including railroad and highway embankments, were extended vertically in the model to stop flow spillage onto the floodplain. This was based on the assumption that dikes presently not sufficiently high will be raised to prevent flooding in the future and is in keeping with BC Ministry of Environment guidelines for floodplain mapping studies. However, unprotected floodplain areas were included as flow conveying. Only one of the 27 bridges modelled was subject to pressure flow, with water touching the bridge deck but not overflowing it. The design profile is plotted in Figure 3. The water level at Mission was found to be 8.9 m or 1.0 m higher than the observed 1894 water level.

Model results are quite sensitive to variations in channel roughness. A 10% increase in roughness would, for example, increase design water levels by a further 0.6 m at Mission. A similar decrease in roughness would reduce the water level by roughly the same amount. The model results are not highly sensitive to local topographic changes and it is anticipated the cross sections will not need to be updated for at least five to ten years unless an extreme flood occurs.

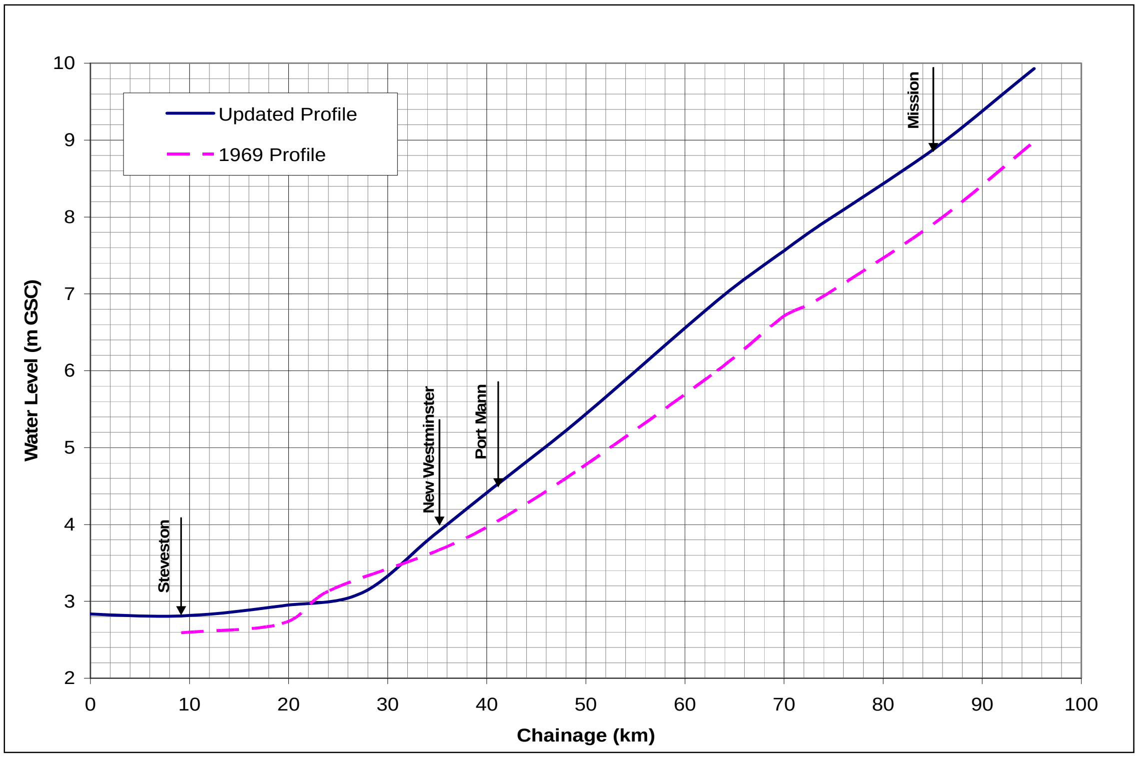

Figure 3: Updated (2006) and original (1969) design profiles.

6. Winter Design Profile

The 200 year tide/surge level in combination with an appropriate Fraser River winter flood flow was specified as the winter design boundary conditions. The 200 year ocean level at Point Atkinson was estimated to be 2.89 m GSC at the 95% upper confidence limit by Triton Consultants. Using a harmonic tidal model of Georgia Strait, Triton translated this elevation to an ocean levels of 2.84 m at Fraser South Arm (2.78 m at Canoe Pass), 2.88 m at North Arm and 2.87 m at Middle Arm. The design event was incorporated into a two-week time series of ocean water levels for simulation in the model.

Ocean storm surges and high Fraser River winter discharges are not statistically independent events and conceivably a 200 year Fraser River winter flow and the design surge could coincide. This condition was initially assumed for the winter base profile. The 200 year Fraser River winter flow at Mission, based on flows recorded between September and March, was estimated to be 9,130 m^3/s. The 200 year winter tributary flows combined with the Fraser 200 year winter flow resulted in a discharge of 12,690 m^3/s at New Westminster. However, in the lower reach where the winter profile exceeds the freshet profile, the computed water level was very insensitive to discharge and nearly completely dependent on the ocean level. Therefore, a more sophisticated joint frequency analysis of coinciding storm surge/ high flow events was not warranted.

7. Assessment of Results

The freshet and winter profiles were combined and the higher of the two profiles adopted as the design profile. The point where the winter profile exceeds the freshet profile is roughly at Km 28. Figure 3 shows the adopted design flood profile. For comparison, the previous design profile, corresponding to the 1894 profile developed in 1969 is also shown. The winter design profile is about 0.3 m higher than the previous profile. In the transition from the winter to freshet profile, the updated profile is slightly lower than the previous profile. However, upstream of New Westminster the updated profile becomes increasingly higher. From about Km 55 to the upstream end of the study reach the two profiles are roughly parallel, with the updated profile being nearly 1 m higher.

Previous modelling of the gravel-bed reach from Mission to Hope (Shumuk 2000), used the Mission 1894 water level as the downstream boundary. Since the new design level was found to be considerably higher, the design profile for the upper reach had to be revised. The new starting condition was found to affect the profile from Mission to Harrison River confluence.

The Fraser River channel and floodplain have undergone significant change over the last century due to the effects of diking, blockage of side channels, river training and dredging. These factors have significantly affected flood levels along the river although determining their exact magnitude is difficult. Flood flow attenuation due to flood storage and overbank spilling between Hope and Mission during the historic 1894 flood event appears to have considerably attenuated the magnitude of the discharge at Mission. The available hydrometric data and a simplified floodplain routing analysis indicated that the peak discharge downstream of Mission in 1894 reached approximately 16,500 m^3/s. The adopted design discharge for the flood model assuming all flow remains contained within the existing dikes is 18,900 m^3/s at Mission and 19,650 m^3/s at New Westminster. The difference in discharge is one of the prime reasons why the 1894 historic flood profile is lower than the computed flood profile for present-day conditions. The loss of floodplain storage between Hope and Mission, assuming dikes can contain the design discharge, would result in up to 0.7 m higher water levels at Mission.

Confinement effects of diking between Mission and New Westminster were investigated using the historic model (1950 cross section geometry) and 1948 flood condition to simulate overbank flow. This analysis indicated that the dikes in the Mission to New Westminster reach would have raised water levels at Mission by approximately 0.4 m compared to the largely un-diked condition that existed in 1894.

A common perception in the media is that the river has “silted-in” over the last several decades and that dredging is needed to restore the river’s conveyence. A detailed comparison of bathymetric surveys from 1952, 1990 and 2005 was made to assess the channel changes that have occurred and the relation to past dredging activity (McLean et al. 2006). The analysis showed that the main channel has not aggraded but has actually lowered significantly downstream of Mission, mainly due to dredging and river training works below New Westminster. This bed degradation has lowered the water level at Mission by 0.2 to 0.3 m. Aggradation has occurred further upstream in the gravel bed reach (Ham, 2005).

By adding and subtracting these variations it can be seen that, with present channel conditions, it is no longer possible to pass the design flood with a Mission water level of 7.92 m, as recorded in 1894. The break-down in Table 3 illustrates the overall effect of these various influences on the present-day design flood level at Mission:

Table 3: Approximate Water Level Changes

1894 historic flood level	El. 7.92 m
Loss of flood storage downstream of Hope:	+ 0.7 m
Confinement from dikes, Mission to New Westminster	+ 0.4 m
Bed degradation Mission to New Westminster	- 0.2 m
Minimum design level for present river conditions:	El. 8.82 m

This heuristic analysis lends support to the model result that showed a water level elevation of 8.9 m at Mission for the design event.

8. Implications for Flood Protection

Based on the available dike information it was found that widespread dike overtopping and failures would occur throughout the region in the event of a re-occurrence of the 1894 flood of record. Dikes from

Chilliwack to Surrey on the south bank and from Kent to Coquitlam on the north bank would be overtopped at one or more locations. None of the dikes provided the BC Ministry of Environment specified standard freeboard allowance of 0.6 m.

An initial evaluation of the flood protection capacity of the present diking systems was made by computing a series of water surface profiles for a range of flood discharges at Mission. These results were then compared to the 1894-profile published in 1969. Without compromising freeboard, the present capacity in the upstream reach of the study area is approximately 16,500 m^3/s, increasing to roughly 17,500 m^3/s at New Westminster. Additional detailed analysis using dike surveys showed that the freeboard of some dikes would be compromised at a flow of roughly 14,500 m^3/s (equivalent to the 1950 flood). At a flow just exceeding 16,000 m^3/s the dikes would be over-topped. The dike evaluation did not include a geotechnical assessment and it is possible that some dikes would fail due to seepage or piping at lower discharges.

For the winter storm surge flood, with the specified 200-year frequency, overtopping of main dikes would not occur but freeboard would be compromised at a number of locations. As a result of future sea level rise and delta subsidence, the adequacy of the dikes will be reduced. According to published reports, sea level has risen on average around 2 mm/year during the last century in the vicinity of the Fraser delta (Church 2002). Most recent studies have concluded that the sea level will rise at a faster rate than in the last century due to the effects of climate change. BC Ministry of Water Land and Air Protection (2002) provide a range of global sea-level rise scenarios between 9 to 88 cm by 2100 (corresponding to rates of 0.9 – 8.8 mm/year). The figures listed above represent only eustatic changes in sea level and do not include effects of local or relative sea-level change induced by factors such as ground subsidence. Estimates of subsidence in portions of the Fraser delta and Boundary Bay have ranged from 1.2 to 1.7 mm/year (Mathews et al 1970). For the purposes of this study we assumed a potential net rise of 0.6 m over the next century, which is on the higher side of the rates in Church (2002) but well within the range of scenarios provided by IGPCC (2001). Assuming a sea level rise of 0.6 m, the winter starting level was raised to 3.38 m. The rise is nearly horizontal over the lower reaches and shifts the location where the winter and freshet profiles cross roughly 5 km upstream up to near the Trifurcation at New Westminster. When applied to the freshet design profile, a 0.6 m rise in ocean level increased the starting level at the downstream boundaries to 2.04 m. With this assumption, the water levels at the winter/freshet profile transition point (at Km 28) were 0.33 m higher. Consequently, adjustment of the freshet profile due to sea level rise will be required over time.

9. Conclusions

The updated design profile, corresponding to a re-occurrence of the 1894 flood discharge is significantly higher than the actual observed 1894 flood profile in the 60 Km reach between Sumas Mountain and New Westminster. The increase in water level is believed to mainly reflect the effects of constructing the extensive system of dikes throughout the Fraser Valley, which have eliminated spills and overbank storage on the floodplain and has reduced floodplain conveyance. Widespread channel aggradation has not occurred in the main river downstream of Sumas Mountain and has not significantly contributed to the increased flood level. In fact, persistent channel degradation has occurred in the lower reach (downstream of Mission) over the last several decades.

Despite the improvements in analytical tools and data collection methods since the original design flood profile was established in the 1950's and 1960's, the current studies remain subject to uncertainty. The hydraulic model should be re-calibrated and verified if another large flood occurs (equal or greater than the 1972 flood event). This could confirm the channel roughness coefficients used in the model.

The model is a useful tool for assessing the effect of various floodcontrol measures and alternatives, for determining the impact of floodplain development and as an aid for predicting flood levels based on forecasted inflows.

10. Acknowledgements

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11. References

Church, M. 2002. Potential Changes in Relative Sea Level and Implications for Sustainability in the Fraser River Delta. *The Changing Face of the Lower Fraser River Estuary*, Symposium Proceedings, Fraser Basin Council: 34-39.

Engelund, F. and Hansen, E. 1967. *A Monograph on Sediment Transport in Alluvial Streams*, Teknisk Forlag, Copenhagen, Denmark.

Fraser River Board 1958. *Preliminary Report on Flood Control and Hydro-Electric Power in the Fraser River Basin.* Victoria, B.C..

Ham, D. 2005.The Morphodynamics of a wandering gravel-bed channel: Fraser River, British Columbia. Ph.D. Thesis, Department of Geography, University of British Columbia, Vancouver, BC.

Inter-Governmental Panel on Climate Change 2001. *Climate Change 2001: The Scientific Basis. United Nations Environment Program*, Published for the International Panel on Climate Change, Cambridge University Press, United Kingdom.

Inland Waters Branch 1969. *Fraser River Flood Profiles, Department of Energy, Mines and Resources*, Vancouver, British Columbia.

Inland Waters Branch 1970. Hydrometric and *Sediment Survey, Lower Fraser River Progress Report 1965-68*, Water Survey of Canada, Department of Energy, Mines and Resources, Ottawa.

Keller, B. 1976. *Legends of the River People.* Collected by Norman Lerman, November House, Vancouver, BC.

Mathews, W. H., Fyles, J. and H. Nasmith 1970. Postglacial Crustal Movements in Southwestern British Columbia and Adjacent Washington State. *Canadian Journal of Earth Sciences*, NRC Research Press 7: 690-702.

McLean, D., Mannerström, M. and T. Lyle 2006. Impact of Human Interventions on the Lower Fraser River, Water Under Pressure-Balancing Values, Demands and Extremes , Canadian Water Resources Conference, October 2006, Vancouver, BC.

McNaughton, A. L. 1951. *Memorandum No.2 on Coincidence between Computed and Observed Levels, Fraser River at Mission*. Dominion Provincial Board, Fraser Basin.

Morton, K.W. 1949. *Fraser River System*, *Province of British Columbia, History of Improvements 1876 to Date*, Dominion Public Works Department, New Westminster, B.C..

Pretious, E.S. and Blench, T. 1951. *Final Report on Special Observations of Bed Movement in Lower Fraser River at Ladner Reach during 1950 Freshet*. National Research Council of Canada.

Shumuk, Y., Zabil, D., Ward, P., Millar, R., Kjelds, J., and R. Henry, 2000. Updating the Design Flood Profile for the Fraser River Gravel Reach with the MIKE11 Hydrodynamic Model, Canadian Water Resources Association, 53rd Annual Conference, 21-23 June 2000, Saskatoon, Saskatchewan.

UMA Engineering Ltd. 2000. *Fraser River Gravel Reach Hydraulic Modelling Study*. Report prepared for City of Chilliwack, B.C..

UMA Engineering Ltd. 2001. *Fraser and Harrison Rivers – Hydrologic and Hydraulic Investigations*. Report prepared for City of Chilliwack, B.C..

Water Land and Air Protection 2002. *Indicators of Climate Change for British Columbia*, Ministry of Water, Land and Air Protection, Province of British Columbia, Victoria B.C..

Yalin, M.S. 1992. *River Mechanics*. Pergamon Press, New York, NY, USA.