LOWER FRASER RIVER HYDRAULIC MODEL
FINAL REPORT

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EXECUTIVE SUMMARY

In September 2005, the Fraser Basin Council (FBC) retained Northwest Hydraulic Consultants Ltd. (**nhc**) to undertake a program of one-dimensional hydraulic modelling on the lower Fraser River using MIKE11 software developed by Danish Hydraulic Institute (DHI). The overall objective was to generate an up-to-date design flood profile based on the following two scenarios:

- The estimated flow during the 1894 Fraser River flood combined with high spring tide conditions.
- The 200 year winter storm surge with high tide combined with a Fraser River winter flood.

The two profiles were then overlaid and the higher of the two values was used to develop an overall design flood profile for the river. Initially, the study reach covered the 100 km distance from Sumas Mountain to the Georgia Strait, encompassing the North, Middle and South Arms, including Canoe Pass, as well as Pitt River to Pitt Lake inlet. Later on, the study was extended upstream to re-assess flood levels upstream of Mission in the reach up to the mouth of the Harrison River.

The hydraulic model was developed using field data collected in 2005. The field work included detailed bathymetric surveys of the channel, LIDAR surveys of the floodplain and ADCP velocity measurements to estimate flow splits at major channel branches. The model was calibrated and verified initially using recorded data from 2002, 1999 and 1997 flood events. Peak discharges from these floods ranged between 11,300 m^3/s and 12,200 m^3/s. Later on, a secondary "historic model" was developed for the reach between Mission and New Westminster, using channel and floodplain topography from 1951 to 1953. This secondary model was used to estimate the channel roughness during floods in 1948, 1950, 1969 and 1972.

The adopted design discharge for the model 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. An assessment of floodplain conditions in 1894 suggested that the flow at Mission may have been only 16,500 m^3/s. For this reason, the 1894 historic flood profile is not directly comparable to the computed design flood profile.

Channel roughness along sand bed rivers may 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. Field observations on the river during relatively high floods in 1950, 1986 and 1997 showed no evidence that the dunes wash out in the 12,200 m^3/s to 14,500 m^3/s Mission flow range. Results of the model calibration runs showed there is a weak trend for the channel roughness to decrease at high discharges. The channel roughness was estimated to average 0.03 in the Douglas Island to Mission reach using the 2002 flood data (maximum discharge of 11,300 m^3/s at Mission). Based on flow estimates for the 1948 flood (maximum discharge estimated to be 15,500 m^3/s), the average channel roughness was found to be 0.027 or approximately 10% lower. Based on this assessment, a value of 0.027 was adopted for the design flood profile computations in the reach between Douglas Island and

Mission. This coefficient corresponds to our best-estimate, based on currently available calibration data.

A statistical analysis of storm surges and astronomical tide levels was carried out to assess the design ocean water level. The Empirical Simulation Technique (EST), developed by the US Army Corps of Engineers for FEMA was used in this study. For the specified 200 year frequency the total water level was estimated to be 2.9 m (at a 95% confidence interval). The 200 year winter discharge of 9,130 m^3/s at Mission was used to estimate the winter flood profile. It was found that the winter flood level in the estuary was virtually independent of the discharge and was governed primarily by the ocean level.

The freshet and winter profiles were combined and the higher of the two profiles adopted as the design profile. The winter profile exceeds the freshet profile in the lower 28 km of the river, or downstream from a point 1,400 m downstream of the Alex Fraser Bridge.

Comparisons were made between the computed profile and the historic 1894 flood profile published from previous studies in 1969. The winter design profile downstream from Alex Fraser Bridge 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 at Mission.

Based on the dike information made available, the design flow of 18,900 m^3/s at Mission would overtop at one or more locations the dikes at Mission, Silverdale, Maple Ridge (Albion), Pitt Polder, Pitt Meadows (South), City of Coquitlam (Pitt), Matsqui, Glen Valley (East and West), Langley (Barnston, Fort Langley and West Langley) and Surrey. In addition, freeboard would be compromised at Pitt Meadows (North, North of Alouette and Middle), Port Coquitlam and Langley (CNR). Dikes upstream of Sumas Mountain are also at risk. At the design flood condition, the Nicomen Island dike would be overtopped over most of its length, along with portions of diking at Kent (downstream end), Matsqui, Dewdney and Chilliwack.

For present winter design conditions (i.e. with no sea-level rise due to climate change or delta subsidence) freeboard is inadequate at Delta (Westham Island, Marina Gardens and sections of River Road), Richmond (all except east end of Lulu Island), Surrey, Maple Ridge and Pitt Meadows (Pitt Polder).

An initial evaluation of the flood protection capacity of the present dikes was made by computing a series of water surface profiles for a range of discharges. 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 the freeboard for Pitt Polder Dike will 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 dike is over-topped at dike chainage 6+248. Freeboard for the City of Coquitlam dike along Pitt River is compromised at a flow just over 15,500 m^3/s. The same holds for the Barnston Island dike and a small segment of the Surrey dike.

The results from this study show that widespread dike overtopping and dike failures would occur throughout the region in the event of an occurrence of the 1894 design flood. Municipal, provincial, federal and First Nation authorities should be alerted and advised of this situation.

Sea-level is expected to rise over the next century in response to climate change although there is considerable uncertainty in the magnitude and rate of rise. Based on scientific studies reported by the Inter-Governmental Panel on Climate Change, a sea-level rise of 0.6 m over the next century was assumed (after accounting for potential ground subsidence on the Fraser delta). This will increase the winter design flood level in the lower 28 km of the river by approximately 0.6 m. The effect of sea-level rise on the freshet profile will be small upstream of New Westminster. The design flood profiles and assessments of dike freeboard are for present sea-level conditions and do not include any provision for future sea-level rise.

High priority should be given to re-assessing the adopted design flow currently based on an estimate of the 1894 flood of record at Hope. This should involve conducting hydrological studies and hydro-meteorological modelling to determine the magnitude and frequency of flood flows in the Fraser River basin. The analysis should include simulations under present climatic conditions and anticipated future conditions to account for changes in climate and basin forest cover (such as due to potential effects of Mountain Pine Beetle infestation).

High priority should also be given to assessing both flood management strategies on the floodplain of the Fraser River and the institutional framework for implementation of those strategies. Flood management strategies should include both non-structural, such as floodplain zoning, and structural alternatives like diking. The level of risk and appropriate design criteria for frequency and freeboard requirements for dikes and developments should be assessed.

The hydraulic model should be re-calibrated and verified if another large flood occurs (equal or greater than a 1972 flood event). This could confirm the channel resistance coefficients used in the model. Model results are quite sensitive to variations in channel roughness. A 10% increase in roughness would, for example, increase 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.

The hydrometric gauging network on the river is an essential component for flood forecasting applications and for model calibration and verification. Secure funding is required to ensure these stations will be available in the future.

Table of Contents

EXECUTIVE SUMMARY i
1. INTRODUCTION 1
1.1 Purpose and Objectives 1
1.2 Study Extent 1
1.3 Terms of Reference 1
1.4 Outline of Report 2
2. BACKGROUND INFORMATION 4
2.1 Physical Setting 4
2.2 History of Diking and River Training 4
2.3 Previous Investigations 5
3. HYDROLOGY AND RIVER HYDRAULICS 8
3.1 Hydrology 8
3.1.1 Fraser River 8
3.1.2 Tributaries between Hope and Mission 10
3.1.3 Tributaries Downstream of Mission 10
3.2 Hydrometric Measurements at Mission Gauge 10
3.2.1 Mission Rating Curve 10
3.2.2 Hydraulic Geometry at Mission Gauge 11
3.2.3 Assessment of Design Flood 12
3.3 Winter Floods 13
3.3.1 Ocean Levels 13
3.3.2 Winter Flood Discharges 14
3.4 Overview of River Hydraulics 14
3.5 Bedforms and Channel Resistance 16
3.5.1 Resistance in Sand-Bed Rivers 16
3.5.2 Test Calculations on Fraser River 17
3.6 Effects of River Training and Dredging 19
3.6.1 River Bed Changes Below New Westminster 19
3.6.2 Channel Changes Upstream of New Westminster 19
3.6.3 Impacts of Dredging on Water Levels 20
4. MODEL DEVELOPMENT 21
4.1 Model Formulation 21
4.1.1 Equations of Motion 21
4.1.2 Model Schematization 22
4.1.3 Boundary Conditions 23
4.2 Available Data-2006 Model 23
4.2.1 Channel and Floodplain Geometry 23
4.2.2 Hydraulic Structures and Bridges 24
4.2.3 Hydrometric and Tidal Data 24
4.2.4 Flow Split Data 25
4.3 2006 Model Development 25
4.3.1 River Cross Sections 25
4.3.2 Hydraulic Structures 26
4.4 2006 Model Winter Calibration and Verification 26
4.4.1 Boundary Conditions 26
4.4.2 Calibration and Verification 27
4.5 2006 Model Freshet Calibration and Verification 27
4.5.1 Boundary Conditions 27

4.5.2 2002 Calibration ... 27
4.5.3 1999 and 1997 Verification ... 28
4.6 Historic Model Development ... 28
4.6.1 Available Data ... 28
4.6.2 Model Assembly ... 29
4.6.3 1948 Flood Calibration ... 29
4.6.4 1950 Flood Verification ... 30
4.6.5 1969 and 1972 Simulation ... 30
4.6.6 Assessment of Dike Confinement Effects ... 30
4.7 Review of Model Roughness ... 30
4.7.1 Assessment of Model Calibration ... 30
4.7.2 Adopted Roughness Values ... 31
4.8 Assessment of Model Accuracy and Limitations ... 32
5. DESIGN FLOOD PROFILE ... 33
5.1 Freshet Profile ... 33
5.1.1 Boundary Conditions ... 33
5.1.2 Simulation Results ... 33
5.1.3 Upstream Model Extension ... 34
5.2 Winter Design Profile ... 34
5.2.1 Design Boundary Conditions ... 34
5.2.2 Simulation Results ... 35
5.3 Combined Profiles ... 35
5.4 Sensitivity Analysis-Freshet Conditions ... 35
5.4.1 Roughness ... 35
5.4.2 Discharge ... 36
5.4.3 Starting Ocean Water Level ... 36
5.4.4 Scour and Bed Level Changes ... 36
5.5 Sensitivity Analysis-Winter Flood Conditions ... 36
5.5.1 Roughness ... 36
5.5.2 Discharge ... 37
5.5.3 Starting Ocean Level ... 37
5.6 Assessment of Results ... 37
6. ASSESSMENT OF DIKE FREEBOARD ... 39
6.1 Freeboard Under 1894 Design Flood Condition ... 39
6.2 Comparison of Dike Crest Elevations and Flood Profiles ... 39
7. FUTURE SCENARIOS ... 41
7.1 General ... 41
7.2 Effect of Dredging on Flood Levels ... 41
7.2.1 Morphological Response to Reduced Dredging Activity ... 41
7.2.2 Short-Term Effects on Flood Profile ... 41
7.3 Sea Level Rise ... 42
7.3.1 Assumptions ... 42
7.3.2 Effect on Winter Flood Profile ... 43
7.3.3 Effect on Freshet Profile ... 43
8. FLOOD FORECASTING MODEL ... 44
8.1 Purpose ... 44
8.2 Flood Forecasting Input Requirements ... 44
8.3 User Instructions ... 44
8.4 Simulation Results ... 45
9. ADDITIONAL WORK ... 46
9.1 Hydrological Assessment of Design Flood ... 46

9.2 Design Frequency And Freeboard Assessment ... 46
9.3 Monitoring and Data Collection ... 47
9.4 Institutional Guidelines ... 48
10. CONCLUSIONS AND RECOMMENDATIONS ... 49
10.1 Conclusions ... 49
10.2 Recommendations ... 50
11. REFERENCES ... 51

APPENDIX A: LOWER FRASER RIVER HYDRAULIC MODEL SUMMARY RESULTS
APPENDIX B: PEER REVIEW LETTER OF ACCEPTANCE
APPENDIX C: FLOOD ATTENUATION-HOPE TO MISSION IN 1894 AND 1948
APPENDIX D: REVIEW OF MISSION RATING CURVE WATER SURVEY OF CANADA
APPENDIX E: DESIGN WINTER OCEAN LEVELS
APPENDIX F: HISTORIC RIVER CROSS-SECTIONAL CHANGES
APPENDIX G: CALIBRATION/VERIFICATION PLOTS
APPENDIX H: MODEL OPERATION MANUAL

List of Tables

Table 3.1 Published Flood Data
Table 3.2 Tributary Freshet Design Flows
Table 3.3 Measured Hydraulic Conditions during Large Floods
Table 3.4 Estimated Hydraulic Conditions in 1948 and 1894
Table 3.5 Winter Flood Frequency Analysis, Tributaries and Mission
Table 3.6 Channel Changes Downstream of New Westminster
Table 3.7 Net Channel Changes between Port Mann and Mission
Table 4.1 Bridge Location Summary
Table 4.2 Hydrometric and Tidal Data
Table 4.3 Freshet and Winter Calibration/Verification Data
Table 4.4 Winter Calibration/Verification Summary
Table 4.5 Freshet Calibration/Verification Summary
Table 4.6 Summary of Channel Roughness Values
Table 4.7 Observed and Calculated Flow Splits
Table 4.8 Historic Simulation Flow and Water Level Data
Table 5.1 Summary of 2006 Modelled and 1969 Calculated Water Levels
Table 5.2 Summary of 2006 and 2001 Modelled Water Levels for Hope to Mission Model
Table 5.3 Summary of Roughness Sensitivity Tests (Freshet Design Condition)
Table 5.4 Summary of Roughness Sensitivity Tests Douglas Island to Mission (Freshet Design Condition)
Table 5.5 Summary of Flow Sensitivity Tests (Freshet Design Condition)
Table 5.6 Summary of Ocean Level Sensitivity Tests (Freshet Design Condition)
Table 5.7 Modelled Local Scour Depths for Scour Sensitivity Test
Table 5.8 Summary of Scour Sensitivity Test (Freshet Design Condition)
Table 5.9 Summary of Roughness Sensitivity Test (Winter Design Condition)
Table 5.10 Summary of Inflow Sensitivity Test (Winter Design Condition-Standard)
Table 5.11 Summary of Inflow Sensitivity (Winter Design Condition – Design Events)
Table 5.12 Summary of Ocean Level Sensitivity Test (Winter Design Condition)
Table 6.1 Summary of Dike Crest Elevations and Water Levels for Various Freshet and Winter Flood Events

Table 6.2	Dike Information Summary (Downstream of Mission)
Table 6.3	Summary of Dike Freeboard for Various Freshet and Winter Flood Events
Table 6.4	Summary of Dike Crest Elevations and Design Flood Levels – District of Kent to District of Mission
Table 7.1	Variation in Dredging Effort by Location
Table 7.2	Prospective Rates of Sea Level Rise on Fraser Delta in the 21st Century
Table 8.1	Calibration and Design Boundary Conditions

List of Figures

Figure 1.1	Site Map
Figure 2.1	Fraser River Floodplain Prior to Development
Figure 3.1	Trends in Annual Fraser River Discharge at Hope
Figure 3.2	WSC Rating Curves at Mission Gauge
Figure 3.3	Hydraulic Geometry at Mission Gauge
Figure 3.4	Annual Maximum Discharges September to March Season, Mission
Figure 3.5	Longitudinal Bed Profile - Lower Fraser River
Figure 3.6	Dune Characteristics at Mission and Steveston
Figure 3.7	Predicted Channel Roughness at Mission
Figure 3.8	Historic Trends in Dredging on Lower Fraser River
Figure 3.9	Trends in Water Levels, Pitt River and Port Mann
Figure 4.1	River Control Structures and Road/Railroad Bridges
Figure 4.2	Tidal and Hydrometric Gauge Locations
Figure 4.3	2002 Freshet Calibration - Main Arm Water Levels
Figure 4.4	Recorded Flow Splits
Figure 4.5	Flow vs Roughness – Sumas Mountain toDouglas Island
Figure 4.6	Comparison of WSC Rating Curve and Model Predictions at Mission Gauge
Figure 5.1	Roughness Sensitivity – Fraser River Freshet Design
Figure 5.2	Roughness Sensitivity (Km 47 to Km 85) - Fraser River Freshet Design Flood
Figure 5.3	Inflow Sensitivity - Fraser River Freshet Design
Figure 5.4	Ocean Level Sensitivity - Fraser River Freshet Design
Figure 5.5	Sensitivity to Local Scour - Fraser River Freshet Design
Figure 5.6	Roughness Sensitivity - Fraser River Winter Design
Figure 5.7	Inflow Sensitivity - Fraser River Winter Design
Figure 5.8	Inflow Sensitivity (Event Based) - Fraser River Winter Design
Figure 5.9	Ocean Level Sensitivity - Fraser River Winter Design
Figure 6.1	Range of Inflows - Fraser River Freshet Design
Figure 6.2	Coquitlam Dike - Ground Survey vs LIDAR
Figure 7.1	Dredging Sensitivity - Fraser River Freshet Design

List of Map and Drawings

Map No.1	Model Layout
Drawing No.1	Profile Comparison-2006 Modelled and 1969 Calculate, below Sumas Mountain
Drawing No.2	Profile Comparison-2006 Modelled and 2001 Modelled Upstream of Mission
Drawing No.3	City of Abbotsford, District of Mission
Drawing No.4	Barnston Island
Drawing No.5	District of Pitt Meadows, City of Surrey
Drawing No.6	District of Pitt Meadows
Drawing No.7	City of New Westminster
Drawing No.8	City of Richmond (1)
Drawing No.9	City of Richmond (2)

Drawing No.10 Corporation of Delta (1)
Drawing No.11 Corporation of Delta (2)
Drawing No.12 Kent, Nicomen Island
Drawing No.13 Dewdney, Chilliwack, Matsqui, Mission
Drawing No.14 Vedder

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Dr. David McLean, P. Eng., was **nhc**'s project manager for the study. Monica Mannerstrom, P.Eng. was the senior hydraulic engineer. Hydraulic modelling work was carried out by Tamsin Lyle, P.Eng. and Bruce Hunter, EIT. Rob Odell, P.E. from **nhc**'s Sacramento office provided technical review and guidance for model development. Dr. Darren Ham was responsible for processing bathymetric and LIDAR data and for GIS development. Michael Tarbotton P.Eng. and Max Larsen P. Eng. of Triton Consultants carried out investigations related to tide levels and storm surge.

1. INTRODUCTION

1.1 PURPOSE AND OBJECTIVES

In September 2005, the Fraser Basin Council (FBC) retained Northwest Hydraulic Consultants Ltd. (**nhc**) to implement a program of one-dimensional hydraulic modelling on the lower Fraser River using the MIKE11software developed by Danish Hydraulic Institute (DHI). The overall objective was to generate an up-to-date design flood profile based on the following two scenarios:

- The 1894 Fraser River design freshet flood combined with high spring tide conditions.
- The 200 year winter ocean water level combined with a Fraser River winter flood event.

The two water surface profiles were then compared at various locations along the river and the higher of the two values was used to develop an overall design flood profile.

A secondary purpose of the study was to combine the new model of the sand-bed reach with an existing MIKE11 model of the gravel-bed reach between Hope and Mission. The combined model provides a valuable tool for real-time flood forecasting along the entire Lower Fraser River.

1.2 STUDY EXTENT

Initially, the design profile assessment extended from Sumas Mountain to Georgia Strait, encompassing the North, Middle and South Arms, including Canoe Pass, as well as Pitt River to Pitt Lake inlet. Later on, the study extent was shifted upstream to re-assess flood levels upstream of Mission in the reach up to the mouth of the Harrison River.

The study area contains portions of Greater Vancouver and Fraser Valley Regional Districts and the municipalities of Richmond, Delta, Vancouver, Burnaby, New Westminster, Coquitlam, Port Coquitlam, Pitt Meadows, Maple Ridge, Surrey, Langley, Mission, Abbotsford, Kent and Chilliwack. First Nations lands in the area belong to the Musqueam, Tsawwassen, Kwayhquitlum, Katzie, Kwantlen and Matsqui Nations.

1.3 TERMS OF REFERENCE

The general methodology and modelling approach was defined in the Lower Fraser flood profile hydraulic model scoping study (**nhc**, 2004). The terms of reference specified that the design freshet discharge at Mission be set equal to the 1894 flood as estimated by UMA (2000 and 2001) in their work to update the design profile for the upstream gravel-bed reach between Hope and Mission. The UMA profile was based on an estimated 1894 discharge at Hope of 17,000 m^3/s and a local inflow of 1,900 m^3/s, giving a Mission design discharge of 18,900 m^3/s. Previous estimates of the return period of the 1894 flood have ranged from 160 years to over 500 years, based on historic discharge data.

During the course of this investigation the design water level at Mission was found to be significantly higher than the level previously used by UMA as the downstream boundary for their flood profile computations between Hope and Mission (UMA, 2000). Consequently, FBC requested **nhc** to extend the model upstream of Mission to provide an updated flood profile in the portion of the gravel-bed reach below the Harrison River confluence influenced by the change in starting levels. This involved combining the UMA model with the **nhc** model and re-running the flood profile. No other changes were made to the UMA model and it was assumed that the model schematization and roughness was still representative for the modified flow conditions.

A number of supplementary tasks were performed to clarify the reasons for the differences between the historic flood levels and the 2006 computed design profile. These tasks included:

- Assessing potential flood attenuation and storage effects in the reach between Hope and Mission during historic flood events.
- Assessing resistance changes during extreme floods using 1948, 1950, 1969 and 1972 hydraulic information.
- Assessing impacts of dike confinement effects in the reach between Mission and New Westminster on historic flood levels.

A secondary numerical model, referred to as the "Historic Model", was developed for the reach between Sumas Mountain and New Westminster using river surveys from mainly 1951 and 1952. This model was used specifically for reproducing the historic floods in 1948 and 1950 in order to assess potential changes in roughness during extreme flood conditions.

These supplementary investigations were intended to improve the overall understanding of how changes to the river channel and floodplain over the last 100 years have affected present river hydraulic conditions. They were also intended to further increase our confidence in the model predictions during extreme flood conditions.

The terms of reference specified that the winter design flood be determined for a 200 year ocean level, including the combined effect from tides and storm surge. The localized effects from processes such as wave runup and wind setup were not considered in this investigation.

The winter and freshet flood profiles were superimposed and combined to generate a design flood profile along the river. It was specified that a standard freeboard value of 0.6 m be added to the design flood level. The adjusted design level was then plotted against existing dike crest elevations to provide a preliminary basis for identifying sections of dikes that may be at risk of overtopping. In some areas, the actual freeboard requirement to account for wave runup and other local factors may be substantially greater than 0.6 m. Furthermore, geotechnical factors affecting dike safety were not considered in this study. Therefore, additional site specific analysis should be carried out to finalize local design conditions.

1.4 OUTLINE OF REPORT

In addition to this brief introduction, the report includes ten chapters and five technical appendices. Chapter 2, ***Background Information***, summarizes the physical setting, history of diking and previous flood investigations. Chapter 3, ***Hydrology and River Hydraulics***, reviews and analyses the historic flood record at Hope and Mission and assesses the hydraulic conditions during floods using hydrometric measurements. This section also assesses past river conditions during historic floods and reviews long-term changes to the river due to the interventions from developments along the river. Chapter 4, ***Model Development***, describes the development, testing, calibration and verification of two one-dimensional hydrodynamic models of the Lower Fraser River. Chapter 5, ***Design Flood Profile***, summarizes results of predicted water levels for the design freshet and design winter flood condition. A sensitivity analysis is also described which illustrates the effect of uncertainties in hydraulic roughness, discharge, starting ocean water level and channel bed level changes on the flood profile. Chapter 6, ***Assessment of Dike Freeboard***, summarizes the freeboard of the dikes for a range of discharges, including the specified 1894 design flood. Chapter 7, ***Future Scenarios***, assesses the hydraulic effects from two hypothetical scenarios: (1) a reduced level of dredging effort below New Westminster and (2) an increase in ocean level due to climate change. Chapter 8, ***Flood Forecasting***, summarizes additional work to join the 2006 model with an existing hydrodynamic model from Hope to Mission. The combined model has been evaluated and tested for use as a real-time flood forecasting tool. Chapter 9, ***Further Investigations***, describes some of the next steps in terms of technical work that should be carried out to go forward with the results of the hydraulic model. Chapter 10, ***Conclusions and Recommendations***, summarizes the key findings of the study.

Five technical appendices have been included. **Appendix A**, provides a comprehensive summary of the study findings. **Appendix B**, is a statement on the study results prepared by the external reviewer, Dr. Robert Millar, Dept. of Civil Engineering, U.B.C. **Appendix C** summarizes results of preliminary computations to assess the effects of flood attenuation and flood storage on the peak discharge below Hope during the 1894 and 1948 floods. Air photography showing the extent of flooding in 1948 is included. **Appendix D**, prepared by Water Survey of Canada, summarizes a review and revision of the stage-discharge rating curve at Mission. **Appendix E** describes the oceanographic analysis that was carried out to assess the magnitude and frequency of extreme sea levels due to the combined effects of storm surge and high winter tides. Historic river cross-sectional changes are shown in **Appendix F**. **Appendix G** illustrates model calibration and verification results. **Appendix H** provides detailed instructions for operating the model.

2. BACKGROUND INFORMATION

2.1 PHYSICAL SETTING

Fraser River is the largest river on the west coast of Canada, draining approximately one-quarter of British Columbia. The river starts at Mount Robson in the Rocky Mountains and flows over 1,100 km to the sea. Between Hope and Sumas Mountain the river has an anabranching gravel-bed channel with frequent islands and large gravel bars. The river slope decreases rapidly below Chilliwack and the river changes abruptly to a sand-bed channel near Sumas Mountain, approximately 10 km upstream from Mission (**Map No. 1**). Downstream of Sumas Mountain, the river has a single main channel but frequently divides around large wooded islands (Matsqui, Crescent, McMillan, Barnston and Douglas Island). The slope of the river averages approximately 5 cm/km (0.00005) downstream of Mission and is tidally affected at most times of the year.

Important tributary inflows between Mission and New Westminster include the Stave, Alouette, Pitt and Coquitlam Rivers. Pitt River is tidally affected, with major flow reversals occurring in the winter months causing water and suspended sediment to be passed upstream into Pitt Lake.

The modern delta commences near New Westminster, 35 km upstream from the sea. Immediately downstream, at the trifurcation, the river splits into three branches: (1) the South Arm, which extends approximately 35 km to Sandheads; (2) Annacis Channel which rejoins the main channel a short distance downstream; and, (3) North Arm which further divides into Middle Arm near its mouth. Although much of the main channel is confined by training walls, there are several locations where the river can spill out of the main channel - including Ladner Reach (into Canoe Pass) and along the left bank downstream of Steveston. The distribution of flow through this branched network of channels is governed by several variables including discharge, tide level, bathymetry and by local control from training structures. The North Arm carries approximately 10% of the total flow. A portion of the flow is also lost through Canoe Pass and through the Albion Dike near Steveston. Losses through Albion Dike have increased over time due to the deterioration of the structure.

2.2 HISTORY OF DIKING AND RIVER TRAINING

Information on the early history of diking and river training was based on reports by Morton (1949) and Sinclair (1961). Only a few river structures and dikes were in place at the time of the flood of record in 1894. Efforts to control the mouth of the river began in 1886 with the building of brush mattress and rock jetties to close side channels breaking out from the main channel through Sandheads (Morton, 1949). Between 1886 and 1893 a 4 km training wall was constructed about 2,000 m south of the present main channel to confine the flow as it spilled across the northern end of Roberts Bank. This was followed by construction of a second structure on the north side of the channel in 1889 to 1892. Some low level berms were in place at Matsqui, Hatzic, Fort Langley, Langley and Westham Island by the 1880's. However, all were overtopped or failed during the 1894 flood.

More extensive development started in 1910. The Steveston North Jetty was built in stages from 1911 to 1932 and the South Jetty was constructed between 1930-1932. The training wall at Woodward Island was constructed in 1925-26 while the first Albion wall was constructed in 1935. As a result of these structures, the channel in the estuary was significantly narrowed and deepened in comparison to conditions that existed in 1894. The channelized section of river was also extended approximately 9 km seaward, mainly as a result of the Steveston North Jetty.

The largest early work on the river upstream of New Westminster involved the reclamation of Sumas Lake and construction of dikes and pumping stations at Chilliwack. This work was completed between 1919 and 1923 (Sinclair, 1961). By the time of the second largest flood of record in 1948, dikes had been constructed along much of the lower Fraser River (Fraser Basin Management Board, 1994). However, the dikes failed during this flood at a number of locations throughout the lower valley. Dike upgrades were undertaken following the 1948 flood and in 1968 the Fraser River Flood Control Program was established, which ensured further upgrading and expansion of dikes until 1995, when the agreement terminated. This extensive diking program has resulted in the river being confined to a relatively narrow strip, particularly downstream of Mission. At the time of reconstruction under the Fraser River Flood Control Program, most of the dikes in the Fraser Valley were upgraded to the 1894 computed profile plus a freeboard allowance of 0.6 m, as determined by Inland Waters Directorate in 1969.

2.3 PREVIOUS INVESTIGATIONS

There have been two major floods since European settlement in the Fraser Valley - 1894 and 1948. Watt (2006) provides a historical account of the 1894 and 1948 floods and includes a number of archival photos to illustrate conditions during these events. The 1894 flood is considered to be the largest flood event in the Lower Fraser Valley since at least 1882. Discharge measurements were not made in 1894 at either Mission or Hope. However, a discharge measurement at New Westminster was reported by the Columbian newspaper on June 5th, 1894. The peak discharge was reported to be 572,000 cfs (16,200 m^3/s). The accuracy of this estimate could not be verified. The Dominion Public Works Department carried out a study in 1898 to estimate the 1894 peak discharge. A brief account of this study was made by Morton (1949):

> *In 1898 an extensive survey of the lower reaches of the Fraser River was made from the Strait of Georgia to Hope. Triangulation, topography, soundings and the determination of the 1894 flood contours were established. Hydraulic calculations were made of discharges of the 1894 flood and river slopes were established, but all records were lost except those of the lower reach, in the fire of September 1898 which razed the down town section of the City of New Westminster. The recorded estimated maximum flood discharge in 1894 from established flood contours is given as 490,000 cfs (13,900 m^3/s) at New Westminster.*

The Fraser Basin Board carried out a number of studies to estimate the magnitude of historic floods in 1894 and 1948. Interim findings from these studies were published in a series of monographs and reports, including McNaughton (1951) and Fraser Basin Board (1958). These studies established the magnitude of the 1894 flood at Hope to be approximately 17,000 m^3/s on the basis of high water marks and correlations with discharges on the Columbia River. A brief account of this estimate is summarized below:

> "*It is stated in a memorandum dated 19th February 1934 that a Colonel Whyte of the Water Resources Branch of the Department of Northern Affairs and Natural Resources ran levels to two points within half a mile of the road bridge. These were known as the high water marks of the 1894 flood. The average level of these two marks was equal to a gauge reading of 38.5 feet, which, by extrapolation of the rating curve for 1948 gave a discharge of 620,000 cfs. It was decided to assume a discharge of 600,000 cfs (17,000 m^3/s) for the peak of the 1894 flood, after the basis of determining the 1894 high water marks was taken into account.*"

Studies were also undertaken to determine the maximum safe discharge that the dikes could contain. The effect of the upgraded dikes on flood levels at Mission was described as follows:

> "*The presence of the dykes has reduced the area of the floodplain and the river channel has changed since 1894, with the result that in several places water levels in 1948 exceeded those of the greater flood in 1894. The level of 25.75 feet* (7.92 m GSC) *recorded at Mission in 1894 is 2.25 feet lower than 28 feet, the level determined for the 1894 peak discharge from the curve relating the discharge at Hope with the Mission gauge, assuming that the river is confined within the present dyking system"*.

On this basis it was concluded that the confinement effect from the dikes would raise the water levels from an 1894 flood by 2.25 feet (0.68 m) at Mission. The estimated water level with dikes in-place was 8.6 m (GSC). Additional studies were made in 1964 using high water elevations and discharge measurements by Inland Waters Directorate, Department of Energy Mines and Resources. A number of staff gauges were installed along the river and high water readings were obtained during 1964, 1967 and 1968. Rating curves were developed based on these readings and using extensions of the curves, the 1948 and 1894 water-marks were adjusted to reflect the new channel conditions. Some backwater computations were also carried out and attempts were made to evaluate the effect of the 1948 dike breaches on observed water levels. Downstream of Km 17.0, the winter tide recorded on December 5, 1967 (2.53 m) was used as the design water level.

Inland Waters Branch (1969) summarized this work and suggested that the derived profile should not be used for design. However, due to the lack of more detailed modelling, the profile has served as a basis for establishing the dike crest elevations and the Flood Construction Level along the Lower Fraser River for the past forty years. One outcome of the Inland Waters Branch study was that the design 1894 water level at Mission was set-back to the observed high water mark (El. 7.92 m) rather than the value of 8.6 m determined by the Fraser River Board (1958). No explanation was given for this adjustment. It should be noted that none of these studies explicitly estimated the discharge of the 1894 flood at Mission.

UMA (2000 and 2001) updated the design profile for the reach from Mission to near Hope. A design flow of 17,000 m^3/s was specified by MOE as the upstream inflow boundary, corresponding to the 1894 discharge at Hope. Initially, UMA estimated corresponding local inflows between Hope and Mission to be 3,135 m^3/s but based on more detailed modelling of Harrison River flows, this value was reduced to 2,205 m^3/s. To account for the unlikely event of coinciding peak flows from all tributaries, this value was further reduced to 1,900 m^3/s for estimating confinement effects of Matsqui Prairie dike. This latter value was adopted for the present study resulting in a peak discharge at Mission of 18,900 m^3/s.

The UMA model assumed a starting level of 7.99 m at Mission, corresponding to the 1894 recorded flood level of 7.92 m plus a 0.07 m allowance for constriction of flow by Matsqui Prairie dike, which was not in place in 1894. This allowance was determined by modelling the river reach just upstream of Mission with and without the dike. Channel confinement downstream of Mission (introduced between 1894 and 2000) was not considered. It was also assumed that the discharge in the channel at Mission in 1894 was the same as the adopted design discharge.

Hydraulic modelling has been carried out previously on the tidal reach of the Fraser River since the 1970's for water resource planning and navigation planning. The Institute of Ocean Sciences carried out a program of one-dimensional hydrodynamic modelling and data collection on the Lower Fraser River between Mission and Sandheads for over two decades (Ages and Woollard, 1978). The model was developed in-house and used a one-dimensional, unsteady, finite difference computational scheme. The model was used to aid in the design of several major structures along the river including the Alex Fraser Bridge and ALRT Sky Train Bridge.

In 1995 the Canadian Coast Guard initiated a program of numerical modelling to assist in navigation on the lower river. Baird & Associates carried out a combination of one- and two-dimensional modelling, their one-dimensional model extending from Sandheads to Chilliwack and the two-dimensional model from Sandheads to Douglas Island (Baird, 1998, 1999). These models were set-up and calibrated for flows much less than the design flood and were not intended for flood profile modelling.

Environment Canada has also carried out one-dimensional modelling between Mission and Port Mann for the last 30 years in order to generate unsteady discharges at various stations in their hydrometric network. This work was carried out using Environment Canada's unsteady flow model One-D (Water Planning and Management, 1983).

3. HYDROLOGY AND RIVER HYDRAULICS

3.1 HYDROLOGY

The snowmelt-generated freshet dominates the hydrology of the river. Flow typically rises in early April, peaks in the first weeks of June and then recedes through the summer. Rainstorm generated floods may modify the runoff pattern. Occasionally (such as in 1980) the highest annual flow occurs in the winter season, although the magnitude of such floods are relatively minor in comparison to the largest freshet floods. The drainage area of the river is 217,000 km^2 at Hope, 228,000 km^2 at Mission and 232,000 km^2 at Port Mann. Tributaries between Hope and Mission include the Harrison River, Chehalis River and Chilliwack River, all entering downstream of Agassiz. During the summer freshet, these tributary inflows typically increase the maximum daily discharges at Mission by 5% to 15%. Tributaries downstream of Mission include the Stave River, Alouette River, Coquitlam River (all regulated) and Pitt River.

3.1.1 FRASER RIVER

Water Survey of Canada (WSC) has measured discharges on the lower Fraser River at Hope and Agassiz, in the gravel-bed reach, at Mission near the start of the sand-bed reach and at Port Mann in the tidally varying reach at the start of the estuary. Although water levels have been recorded at Mission since the 1890's, daily discharges are available only from 1965 to the early 1990's. Daily water levels and discharges have been published at Hope since 1912. The published long-term mean annual discharge is 2,720 m^3/s at Hope and 3,340 m^3/s at Mission.

There are 93 years of recorded annual maximum discharges at Hope and 40 years of annual maximum discharges at Mission. The mean annual flood is 8,670 m^3/s at Hope and 9,510 m^3/s at Mission. A simple correlation between annual maximum flows at Mission and Hope gives the following relation:

$$Q_{Mission} = 1.142Q_{Hope} - 135 \qquad R^2=0.92$$

On average, annual maximum discharges at Mission are 12.5% higher than at Hope although the variability from year to year can be large (range from 2% to 29%).

Table 3.1 summarizes key data on the largest flood events in the period of record. The flood of record in 1894 exceeded the 1948 flood stage at Mission by 0.3 m. As described in Section 2.3, the 1894 flood discharge at Hope was estimated to be 17,000 m^3/s by the Fraser Basin Board on the basis of surveyed high water marks. Based on the simple correlation relation, the peak discharge at Mission in 1894 would have been around 19,000 m^3/s. The 90% confidence limits on this estimate are $\pm$1,600 m^3/s, indicating the flood discharge in 1894 could actually have ranged between 17,400 m^3/s and 20,600 m^3/s.

Large areas of the floodplain were inundated between Hope and Mission during the 1894 flood and significant spills occurred into Sumas Lake and Harrison Lake (via Kent). A simplified flood routing analysis was carried out to assess whether these spills could have affected the magnitude of the peak discharge at Mission. Computational results from this analysis are summarized in **Appendix A**. Although difficult to assess accurately, it is possible that the 1894 flood peak at Mission was reduced by up to 2,300 m^3/s due to flow being stored on the floodplain and diverted to Harrison Lake. This flow reduction likely exceeded the magnitude of tributary inflows between Hope and Mission, so that the peak discharge at Mission may have actually been smaller than the flow at Hope. Based on the Water Survey of Canada rating curve at Mission, this loss of flow would have translated to a water level drop of about 0.7 m at Mission. In other words, assuming the present dikes between Hope and Mission could withstand a flood of the same magnitude as in 1894, water levels at Mission would be approximately 0.7 m higher than recorded in 1894 due to the lack of floodplain storage upstream of Mission. This result is consistent with the 1958 Fraser Basin Board's findings (McNaughton, 1951).

The 1948 flood had a published discharge of 15,200 m^3/s at Hope and a water level of 7.61 m at Mission. McNaughton (1951) estimated the peak discharge during the 1948 flood to be 15,840 m^3/s at Mission, virtually identical to the discharge at Hope. Based on a comparison of recorded discharges at Hope and Mission during the common period of record (1965 to 2005), the expected peak discharge at Mission in 1948 would be approximately 17,200 m^3/s. **Appendix A** describes estimates of spills and flood attenuation between Hope and Mission in 1948. It is possible that the difference between McNaughton's estimate of 15,840 m^3/s and the expected flow of 17,200 m^3/s to some extent represents the effects of spills in 1948. Other factors include uncertainties in the method of estimating flows at Mission and the year to year variability of tributary inflows between Hope and Mission.

It was noticed that the 1948 flood levels exceeded the 1894 water surface profile in most of the gravel-bed reach upstream of Chilliwack Mountain to Hope and also in the lower reach of the river downstream of Port Mann. The 1894 flood produced the highest water levels in the reach from the mouth of the Pitt River to the eastern end of Chilliwack Mountain. Air photography of the Mission and Glen Valley areas during the 1948 flood was reviewed (**Appendix A**). The photography shows that Mission was partly inundated and that Matsqui Prairie and Glen Valley were largely submerged.

The flood of 1950 produced the fourth highest recorded water level at Mission (exceeded only in 1882, 1894 and 1948). The discharge was not measured directly in 1950 at Mission, although McNaughton estimated the peak discharge to be 14,530 m^3/s. The published peak discharge in 1950 at Hope was reported by WSC to be 12,600 m^3/s.

The flood of 1972 was reported to have a peak discharge of 14,400 m^3/s at Mission and 12,900 m^3/s at Hope. These values represent the highest recorded discharge at Mission and the second highest recorded discharge at Hope. However, based on the historical water level data at Mission, the 1972 flood was probably only the fifth largest over the last century.

The peak freshet discharges have been relatively modest over the last 30 years in comparison to earlier decades. The highest discharge at Hope in recent years reached 11,300 m^3/s in 1997 and 10,600 m^3/s in 2002. During the 30 year period between 1975 and 2004 there were five years where the flow exceeded 10,000 m^3/s at Hope, with the flood peak averaging 8,173 m^3/s. By comparison, during the 27 year period between 1948 and 1974 the flows exceeded 10,000 m^3/s in eight years and the average annual maximum discharge was 9,560 m^3/s. **Figure 3.1** shows the overall pattern of flood peaks at Hope over the entire period of record. The period of lowest flood discharges occurred between 1926 and 1947. During this time, annual floods exceeded 10,000 m^3/s in only two years and the average annual flood was 7,910 m^3/s. This cycle of unusually low flows ended with the occurrence of the 1948 flood. Therefore, the period of relatively moderate floods in the last few decades does not necessarily indicate a long-term change in flood generation potential. The long-term periodicity of runoff and peak discharges on the Lower Fraser River has been noted previously by many researchers (Church et al, 1990).

3.1.2 TRIBUTARIES BETWEEN HOPE AND MISSION

UMA (2000 and 2001) summarized the hydrology of four main tributaries entering Fraser River between Hope and Mission. In order of basin size, these are Harrison River (7,870 km^2), Chilliwack River (1,230 km^2), Cheahalis River (383 km^2) and Silverhope Creek (350 km^2). Harrison peak flows tend to be snowmelt generated and typically occur a few days after the Fraser peak flow. The Chilliwack River and Silverhope Creek maximum annual discharges may occur in either summer or fall/winter, whereas Cheahalis River peak flood events occur in the fall and winter.

UMA (2001) estimated the maximum Harrison River flow coincident with a re-occurrence of the 1894 flood to be 1,300 m^3/s using the UBC Watershed Model. For the other tributaries, an approximate method of estimating flows during the Fraser River freshet was developed (UMA, 2000). The flow records for Fraser River at Hope were analyzed and for each year the time period was noted when daily flows were in the 94% to 100% range of the peak flow. This period varied from one to fourteen days. The tributary peak flow during the Fraser peak period was then found and the ratio $Q_{TribMax}/Q_{FraserMax}$ computed and averaged over the period of record for each tributary. Based on this average flow ratio and a Fraser discharge of 17,000 m^3/s the tributary design flows were computed and ranged from about 100 m^3/s to 400 m^3/s. A total design inflow between Hope and Mission of 1,900 m^3/s was presented, which attempted to take into account the low probability of coinciding tributary peak flows.

3.1.3 TRIBUTARIES DOWNSTREAM OF MISSION

Four additional main tributary basins are located downstream of Mission: Stave River, Pitt/Alouette Rivers and Coquitlam River. Stave River and Pitt River are the larger basins with drainage areas of respectively 1,140 km^2 and 795 km^2, while Coquitlam and Alouette Rivers each drain areas of 237 km^2 and 234 km^2.

The Stave, Alouette and Coquitlam Rivers are regulated by BC Hydro Dams and stream gauges used in the analysis are located below the control structures, near the Fraser River, and hence flow records were applied unadjusted. Pitt River is unregulated and the gauge is roughly centred in the watershed. Flow records were adjusted based on the sub-basin area ratio. Correlation coefficients showed almost no correlation between Fraser and tributary peaks.

Design outflows were estimated in the same manner as outlined by UMA and summarized in **Table 3.2**. Derived peak flow ratios were used to estimate tributary flows corresponding to a Fraser River discharge at Hope of 17,000 m^3/s. The design flows were 365 m^3/s for Stave River, 368 m^3/s for Pitt River, 10 m^3/s for Coquitlam River and 4 m^3/s for Alouette River. Unlike during winter flood conditions, rainstorms in the Fraser Valley account for only a small percentage of Fraser River freshet flows.

3.2 HYDROMETRIC MEASUREMENTS AT MISSION GAUGE

3.2.1 MISSION RATING CURVE

Water Survey of Canada (WSC) discussed the history of the Mission gauge operations a number of times during the course of this study and provided valuable information on the measurement techniques and data analysis that have been carried out at the station. During the course of this study, WSC carried out a technical review of the Mission rating curve and produced a new curve which better represents the observations and should improve extrapolation of the curve to higher flood events. Results of this review are summarized in Appendix D.

Discharge measurements are complicated by tidal effects at low flows, but generally show a consistent relation between stage and discharge for flow greater than 5,000 m^3/s. Most of the published discharge data in the freshet season have been derived from two rating curves, which were determined graphically (best-fit "by eye"). **Figure 3.2** summarizes the stage-discharge measurements at Mission, and shows the published curves and a new revised curve. Rating curve 2 was based on discharge measurements made in 1966 and 1967 and was continued in use without revision, until 1990. Rating curve 3 incorporated a small shift at the lower range of flows but coincided with Rating curve 2 at higher flows. The highest discharge measurement in the period 1966-1967 was 13,173 m^3/s on June 21, 1967. The highest discharge measurement in the entire period of record was 13,654 m^3/s on June 17, 1972. Based on the rating curve, this discharge was later adjusted to a value of 14,400 m^3/s. It should be noted that the nominal accuracy of discharge measurements is in the range of $\pm$ 5%, which implies the actual 1972 peak discharge was probably in the range of 12,970 to 14,340 m^3/s.

WSC's review of the available stage-discharge data (Appendix D) resulted in a revision to the rating curve. Rating curve 4 was expressed using the relation:

$Q = 0.607(HG+11.552)^{3.42}$ where HG is the stage in metres

The average departure of the observations from the curve is 2.86 % and the maximum departure was 9.3%. Comparison of rating curve 3 and 4 showed they are coincident from a stage of 3.0 m up to 5.2 m. However, the upper portion of curve 4 is significantly different, indicating the rate of change of stage with discharge is greater than was previously anticipated. Extrapolation of this curve to the adopted 1894 design discharge of 18,900 m^3/s results in a stage level of approximately 9.0 m.

3.2.2 HYDRAULIC GEOMETRY AT MISSION GAUGE

A detailed review was made of the hydrometric measurements at Mission using the current meter notes compiled by Water Survey of Canada. The purpose of this analysis was to assess the actual hydraulic conditions during periods of high freshet flows. These results were then used to estimate flow conditions during the floods in 1948 and 1894. The work involved:

- Plotting cross sections at the gauging line from the meter notes.
- Compiling hydraulic geometry from the measurements (mean velocity, top width and mean depth) and determining correlations with discharge.
- Assessing hydraulic characteristics at the site.

Successive cross sections were overlain using measurements from flows ranging between 6,000 m^3/s and 13,654 m^3/s. This plot shows that although some scour may occur during the rising limb of the flood, the channel fills back in by the time of the peak. Overall changes between low flow and high flow are relatively minor. Furthermore, a plot of the cross section from 1952 showed no significant difference at the site. As a result, the hydraulic geometry relations at the station are very consistent and show relatively little scatter (**Figure 3.3**). Best-fit hydraulic geometry relations are as follows:

$V = 0.0032Q^{0.6691}$
$d = 0.8545Q^{0.2893}$
$W= 369Q^{0.0416}$

where, V is the mean velocity, d is mean depth, W is the top width and Q is the discharge

Table 3.3 summarizes the hydraulic properties measured during floods in 1964, 1967, 1972 and 1974 using the WSC current meter notes. The highest measurements from 1972 and 1967 were used to estimate conditions during the floods in 1948 and 1894. This involved using the observed hydraulic parameters as reference values and then scaling them up to the 1948 and 1894 water levels using the observed hydraulic geometry relations. The discharge was then computed as

$$Q = VWd$$

The results for the 1948 and 1894 flood peaks are summarized in **Table 3.4**. The peak water level in 1948 was 0.43 m higher than in 1972, which represents an additional 240 m^2 of cross sectional area. The corresponding mean depth was 14.4 m. The flow in the channel at Mission was estimated to be in the range of 15,700 m^3/s to 15,600 m^3/s in 1948. In 1894, the flood peak was 7.92 m or 0.74 m higher than the peak in 1972. The mean depth in 1894 was estimated to be 14.65 m. The discharge in 1894 in the channel was estimated to be in the range of 16,500 to 16,800 m^3/s. The main assumption in this calculation is that the cross section did not change appreciably between 1894 and 1972. These results are in general agreement with McNaughton (1951). Additional calculations were made to estimate the water level that would be needed to convey a discharge of 18,900 m^3/s in the main channel. The water level was computed to be in the range of El. 8.8 m (scaling up from 1972 measurements) and El. 8.5 m (scaling up from 1967 measurements), which is reasonably close to WSC's extrapolation of rating curve 4. By comparison, the Fraser Basin Board (1958) estimated that if the 1894 flood was confined, the water level at Mission would have reached El. 8.6 m.

3.2.3 ASSESSMENT OF DESIGN FLOOD

The terms of reference specified that the design freshet discharge at Mission be set equal to the 1894 flood as estimated by UMA (2000 and 2001) in their work to update the design profile for the upstream gravel-bed reach between Hope and Mission. The UMA profile was based on an assumed discharge at Hope of 17,000 m^3/s and a local inflow of 1,900 m^3/s (lower bound), giving a Mission design discharge of 18,900 m^3/s.

It is believed that the adopted design flood at Mission is substantially higher than the actual flow that was experienced below Mission in 1894. This assessment is based on the following evidence:

- The historical accounts by Dominion Public Works Department (Morton, 1949) and published flow estimate (Columbian, 1894) report peak discharge downstream of Mission in 1894 of 13,800 m^3/s to 16,500 m^3/s.
- Hydraulic estimates based on the observed water level at Mission in 1894 and discharge measurements by Water Survey of Canada during floods in 1967 and 1972, suggest the 1894 discharge at Mission was in the order of 16,500 to 16,800 m^3/s.
- It is known that substantial spills and overbank flooding occurred in 1894 over a distance of nearly 80 km between Hope and Mission. A simplified analysis indicated the flood peak at Mission could have been reduced by up to 2,300 m^3/s as a result of spills and overbank flooding (**Appendix A**).
- Studies by the Fraser Basin Board in the 1950's concluded that confinement effects from the upgraded dikes along the Lower Fraser valley would significantly increase flood levels at Mission if another event similar in magnitude to 1894 occurred.

The adopted design flood assumes the present dikes between Hope and Mission will prevent overbank spilling and flood attenuation from occurring. As a result, the magnitude of the flow in the channel below Mission is substantially higher than the historic conditions that were experienced in 1894 before significant diking was in-place. It should also be noted that there can be considerable variability in tributary inflows downstream of Hope which could also partly account for possible differences between the adopted design flood and the actual historic event that occurred in 1894. For these reasons, the design flood used in the modelling investigation downstream of Mission is not directly comparable to the historic situation in 1894.

3.3 WINTER FLOODS

3.3.1 OCEAN LEVELS

Extreme water levels at the sea dikes and in the lower estuary are governed by the occurrence of high tides and storm surge in the winter season, rather than high discharges during the freshet. Therefore, a separate assessment was made of flood levels in the winter season. **Appendix B** contains a detailed analysis of ocean water levels. The following section briefly summarizes the key points from this analysis.

The assessment of ocean water levels was based on measurements at Point Atkinson tide gauge. This station has operated intermittently for a period of 92 years and contains 66 years of complete records suitable for statistical analysis. Some examples of high water levels recorded at this station are as follows:

December 16, 1982:	2.56 m (GSC)
December 5, 1967:	2.53 m (GSC)
February 4, 2006:	2.49 m (GSC)

The combined effects of tide (deterministic component) and non-tidal mechanisms (probabilistic component) were assessed in this study. The non-tidal mechanisms explicitly or implicitly include: storm surge (barometric and wind-induced), seasonal fluctuations (e.g., freshwater discharge, seasonal weather features) and other long-term variations in mean water level (e.g., El Niño, La Niña, global climate change). Non-tidal effects are secondary to tides but are of considerable importance. Storm surge in the Strait of Georgia is in the order of one metre at the return periods of interest to this study. Wave-induced effects such as setup and the super-elevation of water levels by littoral currents are considered to be secondary and were not evaluated.

Two approaches were used for assessing the frequency of ocean levels. Initially, statistical analyses were performed on historical measurements of total water level without distinguishing between tide and surge components. An extremal distribution was fit to the data, which was then used to estimate return periods of extreme events. The advantage of this approach is its simplicity and robustness even though it is predicated on the theoretically-unsound practice of creating statistics from a parameter that includes both deterministic and probabilistic components. The method was used primarily as a check on the reasonableness of more sophisticated methods.

The second approach involved undertaking a tidal analysis of historical total water level observations so that the deterministic tidal component could be inferred and removed from the record, leaving only the storm surge component. Statistical analyses were then performed on this purely probabilistic component to estimate return periods of extreme storm surge events. The non-tidal component was added to the deterministic tide to yield the total combined water level using a method known as the Empirical Simulation Technique (EST). EST is the presently preferred method of the US Army Corps of Engineers and US Federal Emergency Management Agency (FEMA) for dealing with combined risks such as hurricane waves, water levels and tide. The Empirical Simulation Technique is a procedure for simulating multiple life-cycle sequences of non-deterministic multi-parameter systems such as storm events and their corresponding environmental impacts. EST is based on a *Bootstrap* re-sampling-with-replacement, interpolation, and subsequent smoothing technique in which a random sampling of a finite length database is used to generate a larger database. The only assumption is that future events will be statistically similar in magnitude and frequency to past events.

The EST indicates a water level at a 200 year return period of about 2.8 m (GSC) excluding global climate change and wind wave effects. The water level at a 1000 year return period is about 3.0 m (GSC) excluding global climate change and wind wave effects.
For a 95% confidence limit these maxima should be increased by 0.1 m (200 year) and 0.2 m (1000 year). The computed values are similar in magnitude to estimates based on simple addition of the HHWLT (Higher High Water Large Tide) and maximum recorded surge (3.1 m GSC) and Annual Maxima statistics (Gumbel Method). Therefore, we believe the results are very robust.

3.3.2 WINTER FLOOD DISCHARGES

The highest recorded one-day winter flood reached 7,850 m^3/s at Mission on December 27, 1980 and the three-day discharge averaged 6,950 m^3/s. A review of the discharge records at Hope, Mission and the intervening tributaries showed that intense localized rainstorms in the Fraser Valley triggered this unusual winter flood event. A frequency analysis of maximum daily discharges in the winter season (October to February) was carried out using 36 years of data for Mission between 1965 and 2001. **Figure 3.4** shows a frequency plot. Estimated discharge values and frequencies are summarized in **Table 3.5**. The 200 year winter discharge at Mission is estimated to be 9,130 m^3/s, which corresponds approximately to a mean annual flood in the freshet season.

The 200 year winter discharges were also estimated for the four tributaries downstream of Mission as summarized in **Table 3.5**. The 200 year flow at Mission combined with the 200 year tributary flows resulted in a discharge of 12,690 m^3/s at New Westminster. Combining this flow with the winter design ocean level may seem overly conservative. However, subsequent sensitivity runs (Section 5.5.3) showed that in the reach where the winter design condition governs, the magnitude of the river discharge had almost no effect on the computed water level. This indicated that during winter flood conditions the ocean level has a high degree of control on the river profile. Therefore, a detailed joint frequency analysis of coinciding storm surge/ high flow events was not considered warranted.

3.4 OVERVIEW OF RIVER HYDRAULICS

Key physical characteristics of the river that govern hydraulic conditions in the river are summarized below:

Features	**Factors to be considered**
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 small now. 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.

Some additional comments on each of these features are provided below.

Gradient

Figure 3.5 shows a longitudinal profile of the riverbed and water surface from Sumas Mountain (Km 95) to the mouth of the river (Km 0). The riverbed displays a series of deep pools, typically in the river bends where secondary currents erode deep scour holes, or at locations where the channel narrows and becomes constricted. The water surface drops fairly uniformly below Mission to the sea, but there is no discernable gradient to the riverbed. The water surface slope is governed primarily by friction losses along the channel and not by the overall gradient of the river valley. During extreme flood conditions, the river has an average gradient of about 5 cm/km, which is very flat. This indicates that the water level at any particular site is strongly affected by downstream control, rather than local hydraulic conditions.

Large Tidal Range

The river is tidal at virtually all flow conditions and is subject to flow reversal during the low flow season, typically between October and March. Tidal fluctuations in the river depend on the freshwater discharge and the tidal range in the Strait of Georgia. Water storage areas such as lakes, marshes and basins dampen the magnitude of water level fluctuations caused by tides or rapid fluctuations in discharge. Pitt Lake, with a surface area of 55 km^2, is the largest storage area in the study area and has a significant dampening effect on tidal fluctuations in the mainstem, particularly during periods of low river inflows. The effect is much less significant during the freshet season. During the peak of the 1972 flood (approximately 20 year return period), the tidal range was only about 0.1 m upstream of Port Mann. To ensure steady flow conditions at the upper boundary, the Lower Fraser model was extended 10 km above Mission to Sumas Mountain.

Flow Stratification

Mixing of the fresh river water and denser saline ocean water produces stratified flow, with the lower estuary developing a well defined “salt-wedge” during flood tides. Flow stratification reduces the conveyance of the channel (since the fresh water is forced to flow over top of the salty water). Furthermore, the wedge introduces additional turbulence and energy dissipation. One-dimensional and two-dimensional depth averaged hydraulic models cannot simulate flow stratification and instead it must be accounted for by calibrating roughness values or making a portion of the channel cross-section ineffective. The salt wedge is generally restricted to the reach downstream of Deas Island for most times of the year (Ward, 1976). The position of the salt-wedge shifts further downstream with increasing fresh water discharges.

Effect of Dikes

Map No.1 outlines the Lower Fraser River floodplain and shows the extent of diking. Dikes confine virtually the entire channel downstream of Sumas Mountain and restrict the amount of flow that is conveyed overbank. Dikes also protect major islands, such as Barnston Island and Lulu Island, further restricting the flow to the main channels. This situation is different than for the upper reach between Laidlaw and Sumas, where many of the islands are subject to overbank flooding and inundation.

Varying Roughness

River bed dunes form in sand-bed rivers as a consequence of sediment transport. Dunes start off as small irregularities on the bed and grow as flow increases. They migrate in the downstream direction, producing large fluctuations in bed levels. Dunes typically produce most of the energy losses in sand-bed channels. Alternatively, if the dunes wash out at high velocities, the roughness will be reduced.

River Training Structures / Islands
A series of river training structures influence flow conditions in the lower river. Also, several large islands split the main channel between Port Mann and Mission. For accurate model calibration, flow split data is required.

Effects of Dredging and River Confinement
Over the last century, the Lower Fraser River has been dredged and mined for sand, confined by dikes and training walls, and re-aligned to accommodate deep-draft vessels. These changes have all induced long-term adjustments to the river bed topography and channel hydraulic characteristics. Consequently, it is difficult to compare historic flood profiles from extreme events such as 1972, 1950, 1948 and 1894 with more recent flood levels. It is also not correct to use these historic profiles to calibrate or verify a hydraulic model that is developed from recent channel and floodplain topography. Hence, the initial model calibration was restricted to using recent flood events from 2002, 1999 and 1997 even though these floods had significantly lower magnitudes than the design flow. Subsequently, additional calibration runs were made using 1948 and 1950 flood profiles in a second hydraulic model that was developed from channel and floodplain topography surveyed mainly in 1951.

3.5 BEDFORMS AND CHANNEL RESISTANCE

3.5.1 RESISTANCE IN SAND-BED RIVERS

Channel resistance in sand-bed rivers is produced by several different mechanisms:

- Energy losses due to the friction created by the surface of the river bed (so called grain resistance).
- Energy losses created by bedforms (dunes) on the river bed, which create eddies and flow separation (form losses).
- Energy losses induced by secondary currents produced by bends and plan form changes.
- Energy losses due to flow obstructions created by river structures and bridges.

One dimensional hydrodynamic models can represent some local energy losses due to bridges and other structures by using expansion and contraction coefficients. The energy losses due to grain resistance, bedform resistance and resistance induced by flow curvature and bank friction are lumped in a single parameter (Manning coefficient, n). The roughness in a uniform sand-bed channel can be expressed as follows:

$$n^2 = (n')^2 + (n'')^2 + (n''')^2 \qquad (1)$$

where, *n*' is the grain roughness, *n*" is the form roughness from dunes and *n'''* is the resistance from other plan form sources (banks, bends, log booms etc)

Grain roughness *n'* depends on the sediment size ~ D_{90}, while dune form roughness depends mainly on the dune height *H,* wavelength *L* and water depth *h*. Dune form roughness can be estimated from the geometry of the dune and water depth by an equation derived independently by Engelund and Yalin in the 1960's (Yalin, 1992):

$$\frac{n''^2}{h^{1/3}} = \frac{1}{2g}\left(\frac{L}{h}\right)\left(\frac{H}{L}\right)^2 \qquad (2)$$

Dunes are one of the main factors governing hydraulic resistance in the channel of a sand-bed river. An initial smooth flat bed can increase its roughness from 0.020 to 0.030 by the presence of dunes (Julien and Klassen, 1995). Most field studies on large sand-bed rivers show Manning's n declines with rising stage and flow (ASCE, 1996). The decline is generally attributed to two factors, (1) a decrease in the relative roughness and (2) change in the dune geometry with flow velocity. Relative roughness is the ratio of the height of the predominant bed projections on the bed to the flow depth. For an alluvial sand-bed channel, these projections are the bedforms (dunes). Furthermore, as the flow velocity and shear stress increase at high flows it is sometimes observed that the bedforms "wash-out", creating a smoother, flat-bed condition. For example, repeat measurements on the Mississippi River showed the n value is about 0.06 at very low flow and decreases to 0.025 at high flow (ASCE, 1996). However, a recent case study on the Rhine River during the flood of 1998 flood showed the Manning n value increased with discharge (Julien et al, 2002).

A number of methods have been developed to predict alluvial channel roughness using sediment transport theory. Among the most widely referenced include Einstein and Barbarossa (1952), Engelund-Hansen (1967) and van Rijn (1989). The methods estimate the state of the bed at various stages of sediment transport using relations between channel form roughness, bed shear stress and sediment transport. The methods were derived mainly from small laboratory flumes and are subject to considerable uncertainty when scaled up to actual rivers. Field verifications on larger low-gradient sand bed rivers have generally produced poor results (Julien and Klaasen, 1995, Julien et al, 2002). It was found that theoretical methods to predict roughness worked best when actual measurements of dune geometry were available-this is seldom possible for the case of design flood conditions. Other studies of bedforms in the lower Mississippi River showed the size and roughness characteristics of dunes are not predicted well by experimental and theoretical relations, even though intensive flow measurements were made in the study area.

3.5.2 TEST CALCULATIONS ON FRASER RIVER

Observations of dunes on the Fraser River during freshet flows have been made mainly at the upstream and downstream ends of the sand-bed reach:

- At the Mission gauge (km 85) in the 1984, 1985 and 1986 freshet seasons. Velocity was estimated using measurements made at Mission gauge (McLean, 1990);
- In the Woodward-Ladner Reach of the estuary (km 16) by Pretious and Blench (1951) during the 1950 freshet. These data have also been analysed by Allen (1973);
- At Stevenson (km 10) in the estuary during the 1989 and 1997 freshet seasons as reported by Villard and Church (2003, 2005) and Kostaschuk et al. (2004).

The observations by Pretious and Blench (1951) were made throughout the 1950 flood and include measurements at unusually high flow conditions. An extract from the observations is as follows:

> *While the Hope discharge remained below 350,000 cfs (10,000 m^3/s) the waves (dunes) were of the order of 20 to 50 feet long and a couple of feet high. When 350,000 cfs was exceeded. the waves became conspicuously larger. As river stage increased so did wave size. On the 23rd, a specially large wave that had started to grow on the 20th attained a full size of about 500 feet (150 m) long and 15 feet (4.5 m) from trough to crest and maintained a fairly steady rate of progression of 250 feet/day (75 m/day) till the 29th.*

These observations illustrate that the dunes were still growing in size on June 20th, when the Fraser River reached its peak flow of 15,840 m^3/s (estimated by McNaughton, 1951). There is no evidence the dunes washed out or developed a flat-bed that would cause the resistance to decrease appreciably.

The observations at Mission were made over a range of flows, from a low of 5,860 m^3/s (August 3, 1984) up to a maximum of 12,300 m^3/s on June 5, 1986. The dune height and wave length generally increased with discharge. Observations at the highest flow conditions indicated the dunes had a wave length of 44 m, an average height of 1.7 m and a maximum height of 2.8 m. The observations at Steveston were made during the 1997 freshet (peak discharge at Mission of 12,180 m^3/s). These measurements were made in the tidally-dominated reach of the river and are less representative of conditions upstream of New Westminster.

Figure 3.6 shows the values of dune height versus dune length for data at Mission, Steveston 1 and Steveston 2. Both *H* and *L* increase with discharge, keeping the value of *H/L* almost constant. If H/L remains constant, then Equation 1 implies the channel roughness should increase with increasing discharge. Only data sets Mission and Steveston 2 include simultaneous measurements of *H, L* and *h*, making it possible to compute dune roughness *n*". Assuming grain roughness *n*' = 0.018, the total roughness *n* can be computed from equation (1). Roughness varies between 0.020 at very low flow to 0.032 for Mission, and between 0.024 and 0.042 for Steveston 2. The average value at Mission is *n* = 0.025 at high flows. The computed roughness values for Steveston 2 are somewhat higher, due to the larger dunes.

Test calculations were also made at Mission using several different alluvial roughness predictors including the equations of van Rijn, Engelund-Hansen and Einstein-Barbarossa. These relations do not require measurements of dune geometry and so in principal, can be used to estimate roughness conditions beyond the range of field observations. Van Rijn's method significantly underestimated the dune height measured in 1984 to 1986 at Mission (by a factor of five at higher flows). Van Rijn's equation also predicted the dunes would wash out completely when the mean velocity exceeded approximately 2 m/s, while field measurements under these same conditions showed the dunes were typically 2 m in height and continuing to increase in size. These tests confirmed previous studies by Julien and Klaassen (1995) which showed that van Rijn's equations are not reliable predictors of bedform properties on large sand-bed rivers.

The equations were also tested against the observed hydraulic geometry data collected by WSC at the Mission gauge. The Engelund-Hansen equation produced reasonably good agreement in terms of predicting mean velocity and mean depth during moderate freshet flows (**Figure 3.7A**). All relations tended to over-estimate the velocity (particularly at flows less than 10,000 m^3/s) which indicates roughness was under-estimated. However, all of the theoretical equations are sensitive to the assumed sediment grain size and water surface slope. For example, changing the sediment size from 0.35 mm to 0.25 mm in the Engelund-Hansen equation increased the predicted velocity (decreased roughness) by 13%.

The relation between channel roughness (grain resistance plus dune resistance) and mean velocity is shown in **Figure 3.7B**. It should be noted that all of these predictions represent only the effect of form roughness and grain roughness and do not include other losses caused by bends or local effect along the river banks (such as log-booms). The Engelund-Hansen equation predicted roughness decreases from 0.028 at a discharge of 9,000 m^3/s to 0.024 at a discharge of 19,000 m^3/s. The Einstein-Barbarossa equation predicted much lower roughness at low flow and the opposite trend (roughness increases with flow). Also superimposed on this plot are the estimated roughness values based on the measurements of dune height and wave length made at Mission in 1984 to 1986. These results show a trend of increasing roughness with discharge. Estimated roughness values are generally in the range of 0.025 to 0.032, which is similar to the range of the Engelund-Hansen predictions (although the trends of the two equations are in the opposite directions).

Manning roughness values of 0.032 to 0.025 are typical of values observed on other large sand-bed rivers (Julien, 1989). However, given the uncertainty of the various theoretical equations, we would not recommend extrapolating these results to the design flood conditions on the Lower Fraser River. Instead, we believe the roughness values used in the hydraulic model should be based on direct calibration using recorded water surface profiles and discharges. However, the theoretical predictions provide a useful basis for assessing the reasonableness of the calibration results, particularly for extrapolating them to the design flood condition.

3.6 EFFECTS OF RIVER TRAINING AND DREDGING

3.6.1 RIVER BED CHANGES BELOW NEW WESTMINSTER

The channel has deepened appreciably below New Westminster in response to dredging, river training and confinement by bridges and dikes. An extreme example of bed lowering has occurred near New Westminster at the site of the Patullo Bridge and CN Rail Bridge. Surveys from 1903 indicate the bed has lowered by up to 10 m compared to recent surveys, probably mainly in response to local pier scour and the constriction induced by scour protection. Similar magnitude changes to the river bed occurred after Alex Fraser Bridge was constructed. This permanent bed lowering reduced the need for maintenance dredging in the reach of St. Mungo Bend (Km 28).

Annual river surveys from Public Works have been used to produce time series plots of bed levels in the navigation channel to illustrate the long-term overall channel response. Average bed levels in the channel have typically lowered by 3 m over a 30 year period or approximately 0.1 m/year (**nhc** 1999), with the greatest bed lowering occurring in the 1980's. This is consistent with the period when the rate of sediment removal exceeded the incoming bed material load. Since the mid-1990's the rate of bed lowering has slowed considerably or in some locations (below Steveston Cut, Km 8-11) actually reversed in some years due to the reduced dredging effort.

Appendix C summarizes typical cross sections in 1946-1951 and in 2005. **Table 3.6** summarizes overall channel changes on the South Arm of the Fraser River between Sandheads (Km 0) and New Westminster (Km 35) for various time periods. The volumes represent net changes (deposition – erosion) computed from a comparison of annual surveys by Public Works & Government Services Canada (PWGSC) or their predecessors. **Figure 3.8** shows the annual dredge removals on the river and compares these removals to the incoming bed material load at Mission (**nhc**, 2002).

3.6.2 CHANNEL CHANGES UPSTREAM OF NEW WESTMINSTER

Dredging and channel excavation below New Westminster appears to have initiated progressive degradation that is migrating upstream towards the end of the sand-bed reach at Sumas Mountain. This degradation was initiated by hydraulic changes along the river, notably the flattening of the water surface profile between Sandheads and New Westminster and steepening of the profile between New Westminster and Sumas Mountain. A simplified model of the degradation process was presented in McLean, Mannerström and Hunter (2005) using the one dimensional sediment program GSTARS 3 (Yang, 2002). These simulations showed the degradation would take a number of decades to approach equilibrium 50 km upstream at Mission.

Relatively complete surveys of the channel between New Westminster and Mission were made in 1952, 1991 and 2005, which provides a good basis for assessing channel changes in this reach. The survey data were recently compared to identify systematic channel changes in this reach. Typical cross section changes between 1951 and 2005 are summarized in **Appendix C**. The net channel volume changes between Douglas Island just upstream of Port Mann (Km 47) and Mission (Km 85) were also computed. Net channel changes are summarized in **Table 3.7**. Approximately 21 million m^3 of sediment has been removed from the channel reach between Port Mann and Mission over the last 50 years by degradation. This sediment has been transported downstream into the delta and has probably contributed to the dredging burden in the navigation channel.

3.6.3 IMPACTS OF DREDGING ON WATER LEVELS

The tide gauges at Steveston and New Westminster and hydrometric stations at Port Mann and Mission provide a good record for assessing the long-term cumulative effects of dredging and other river training works on water levels. The lowest recorded water levels at New Westminster decreased consistently from the mid 1960's until the mid 1990's, then remained approximately constant. The lowering at New Westminster amounted to approximately 0.7 m in 25 years (McLean and Tassone, 1988). It is believed this decrease is primarily due to channel bed lowering due to dredging and river training.

Water levels at average flows and moderate freshet flows show a similar trend of decreasing water levels over time. These trends are particularly evident at New Westminster and Port Mann and the Pitt River near the confluence with the Fraser. The water level trends are complicated by several factors including variations in discharge patterns and tides. Therefore, in order to reduce the effects of other variables, a "specific-gauge analysis" was carried out using hourly data at Port Mann and Pitt River near Port Coquitlam. This involved plotting recorded water levels for specific river discharges (as recorded at Mission) and specific tide levels. Separate curves were prepared for the minimum, mean and maximum tide levels at a Mission discharge of 8,000 m^3/s. **Figure 3.9** shows plots at Port Mann and Pitt River near Port Coquitlam at a mean tide condition.

At a discharge of 8,000 m^3/s, the corresponding water level decreased by approximately 0.6 m at the mouth of the Pitt River and Port Mann over the last 35 years. An analysis of discharge measurements at the Water Survey of Canada gauge at Mission (08MH024) showed a similar trend, although the magnitude of the changes was considerably smaller. At a constant discharge of 8,000 m^3/s, water levels have lowered at Mission by approximately 0.2 to 0.3 m over the last 40 years.

4. MODEL DEVELOPMENT

4.1 MODEL FORMULATION

4.1.1 EQUATIONS OF MOTION

MIKE11, 2005 version (SP 4), by the Danish Hydraulic Institute was used in this investigation. Hydrodynamic models such as MIKE11 solve the one-dimensional equations of mass conservation and momentum:

mass conservation: $$\frac{\partial Q}{\partial x} + B\frac{\partial y}{\partial t} = 0 \tag{3}$$

momentum: $$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \tag{4}$$

where, Q = Flow
B = Surface width
x = Distance
y = Depth
t = Time
S_f = Friction Slope
S_o = Bed Slope
v = Velocity
g = Gravity

There are several techniques for simplifying these equations. For the case of steady gradually varied flow, the momentum equation can be reduced to:

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} \tag{5}$$

This is the basis of all standard-step backwater analysis models used on non-tidal rivers. The equation can be simplified further by neglecting the inertial term:

$$S_f = S_0 - \frac{\partial y}{\partial x} \tag{6}$$

Equation (6) can be combined with the continuity of mass equation, leading to the result:

$$\frac{\partial y}{\partial t} + c\frac{\partial y}{\partial x} = K\frac{\partial^2 y}{\partial x^2} \quad where \quad K = \frac{cy}{3\left(S_0 - \frac{\partial y}{\partial x}\right)} \; or \; \frac{cy}{3S_0} \; (approx) \tag{7}$$

and, c is the wave speed

This equation is commonly used in flood routing to predict the speed and subsidence of a flood wave along a channel. It is termed the "diffusion" method since the equation is similar in form to the diffusion equation. The equation implies that an observer, moving along with the crest of the flood wave will measure a subsidence in the magnitude of the wave over time, with the rate of subsidence governed by the diffusion coefficient "*K*".

A further simplification can be made by assuming $S_f = S_0$. This leads to the kinematic wave equation:

$$\frac{\partial y}{\partial t} + c\frac{\partial y}{\partial x} = 0 \qquad (8)$$

which represents the special case where the diffusion coefficient *K* is zero (the flood wave does not subside as it travels along the channel).

MIKE11 has three different options for solving the equations:

- (High order) fully dynamic method,
- Diffusive wave approximation, and;
- Kinematic wave approximation

In each case, the equations of motion are transformed to a set of implicit finite difference equations in a computational grid consisting of alternating Q (discharge) and h (water level) points that are computed for each time step. The adopted numerical method for solving the finite-difference equations is a six point Abott scheme.

For the tidally varying reach between Mission and Georgia Strait a fully dynamic method must be used since accelerations terms are important components that cannot be neglected. UMA (2000) encountered instability problems when trying to use the dynamic method for the MIKE11 model of the gravel-bed reach between Mission and Hope. Consequently, the diffusive wave approximation was adopted. This simplification may to some extent have been the reason for difficulty in achieving good calibration results in locations where backwater effects were significant.

The terms of reference specified that **nhc**'s 2006 model be joined with UMA's 2000 model to provide a flood forecasting capability (Section 8). MIKE11 allows for the use of different solution methods for different reaches within a single model and hence the full dynamic method was used for the tidal reach from the Strait of Georgia to Mission and the diffusive wave solution method for the Mission to Hope reach.

4.1.2 MODEL SCHEMATIZATION

Model schematization involves developing a network of river reaches, branches and junctions to represent the river channel and floodplain geometry. Considering the excellent bathymetric coverage and extensive LIDAR survey, choosing closely spaced sections was possible. However, as the section spacing is reduced, the computational time step must also be reduced to maintain computational stability, resulting in lengthy model runs without significant gain in accuracy. Based on the cross-sectional channel geometry and the Courant criterion (DHI, 2004) a spacing of 400 m was selected. In the narrower side channels, the spacing was reduced to 200 m. Cross-sections, including overbank sections, were located perpendicular to channel flow as shown on the model layout in Map No.1.

The main channel was modelled as a single branch and cross-sections were numbered based on the thalweg chainage. The distances roughly correspond to PWGSC standard Fraser River chainage. Side branch sections were numbered from the downstream end and flow direction in MIKE11 was specified as negative. For the calibration/verification floods, flow is mainly contained within banks, and overbank sections were modelled by extending the channel sections across the floodplain.

Branches were connected within MIKE11 at pre-selected points. After each cross-section import and after connecting a set of branches, the model was run to see that no hydraulic instabilities formed. Where problems were encountered, these were rectified as much as possible before proceeding. For all runs, the model was started with an initial parameter file. A run time step of 2 seconds was selected as optimum based on Courant criterion. Initially, freshet time periods of only two weeks were modelled to reduce run-times but were later extended to span the entire freshet duration.

4.1.3 BOUNDARY CONDITIONS

The following boundary conditions were specified for each scenario in the 2006 model:

- Inflow at upstream end (near Sumas Mountain). The flow was assumed to be equal to flow at Mission, since no significant tributaries enter the in-between reach.
- Inflow from main tributaries: Pitt, Alouette, Stave and Coquitlam Rivers. (The tributaries are small relative to the Fraser and have minor impact on the Fraser profile).
- Tidal levels at the mouth of the North Arm, Middle Arm, South Arm and Canoe Pass.

The historic model which was used specifically for reproducing the 1948 and 1950 floods was developed only for the reach between Mission and New Westminster, based on available historic cross section survey information. The New Westminster tide gauge was used to establish the downstream boundary for this model.

4.2 AVAILABLE DATA-2006 MODEL

4.2.1 CHANNEL AND FLOODPLAIN GEOMETRY

Public Works and Government Services Canada (PWGSC) provided sounding data comprising of single track and multi-beam bathymetry, collected mainly in 2004. The data were geo-referenced to GVRD datum and provided in ASCII format. The complete dataset consisted of roughly 19 million individual points collected on either survey or swath transects. The data files were imported into ArcMap GIS and then converted to point coverage in Arc/Info format. Soundings upstream of Mission were collected in 2003. The initial dataset contained some gaps in Middle Arm, Canoe Passage, and south of Steveston South Jetty. Additional small gaps were also identified along channel margins where log booms prevented boat access. To provide coverage for these areas PWGSC conducted additional surveys in December, 2005 and also supplied information from a 1989 survey that included bank edges.

The 2004 PWGSC surveys extended up Pitt River into the shipping channel of Pitt Lake, but did not include the lake. Lake bathymetry was obtained by digitizing a hydrographic chart of Pitt Lake (Chart 3062, surveyed 1984).

Bank and floodplain topography was compiled from LIDAR data, collected by Terra Remote Sensing Inc. in June, 2005. The LIDAR data consisted of both full return and bare earth elevations, where water, buildings, vegetation, vehicles etc. were removed from the full return file through a combination of software and manual editing by Terra. The LIDAR data consisted of 1 m ground coordinates and elevations in ASCII format and contained approximately 130 million elevation points. The survey did not extend upstream of Matsqui Prairie and LIDAR data collected in 1999 and 2004 was incorporated to extend floodplain cross-sections to Sumas Mountain.

The geometry of the river and floodplain was constructed within six overlapping sub-areas. The boundaries correspond to limits of the LIDAR data except near the mouth, where they were modified to include tidal flat areas. This division was necessary to keep the size of each created TIN model below the maximum allowable file size within GIS. The input data for each model were thinned to 3 metres within the TIN module to eliminate duplicate and redundant points. This spacing was found to reduce computing time and file storage size, while preserving the shape of channel and floodplain features. Each TIN model represents a seamless interpolated model of the channel bed, banks and floodplain based on the available topographic data and was used to extract cross-sections for input to MIKE11.

4.2.2 HYDRAULIC STRUCTURES AND BRIDGES

Figure 4.1 shows the location of hydraulic structures within the study reach. The purpose of most structures is to locally confine the river channel to increase flow velocities and consequently reduce sediment deposition in order to help maintain a navigable channel. Design drawings of the structures where obtained from PWGSC and each structure was reviewed to determine the best method of representation in the model. In most instances, the structures form boundaries that simply define the extent of the effective channel area and do not require specific modelling. However, the Albion Wall passes substantial amounts of flow (in the order of 15%) and was modelled as a side branch.

Bridge crossings are summarized in **Table 4.1** and **Figure 4.1**. There are nearly 30 bridges within the study area, 27 of which were modelled. To accommodate navigation, the bridge decks are typically well above the design flood level. However, some of the railroad structures have swing spans and decks that could potentially get submerged during the design flood. In order to model the bridges, information on pier configurations, low chord and deck elevations was obtained from Ministry of Transportation, BC Transit, Airport Authority and various municipalities and consulting firms. Some bridge dimensions and details were verified during field trips.

4.2.3 HYDROMETRIC AND TIDAL DATA

Hydrometric and tidal data was required to establish model boundary conditions for calibration, verification and design flood profile computation. For all model runs, inflow at the upstream end and from main tributaries, along with downstream water levels formed the boundary conditions. **Table 4.2** summarizes available stream-flow, water level and tidal data. The main source of information was Water Survey Canada (WSC), but water level data was also provided by Marine Environmental Data Services (MEDS) as well as Langley, Surrey, Richmond and Delta. Flow records for Stave River were obtained from BC Hydro. Non-continuous water level records collected at staff gauges were available for 1999 (Sigma, 1999) and for 1997 from MOE.

In the Scoping Study (2004), **nhc** recommended that the 1999 flood be used for model calibration and the 1997 flood for verification. However, in view of the more extensive water level data now available for 2002, the 2002 flood was selected for primary calibration with the 1999 and 1997 floods used for verification. The 2002 peak flow (11,270 m^3/s at Mission) is slightly less than the 1999 and 1997 floods but as the more recent event it is more representative of present channel conditions. Also, significantly more continuous water level data is available for this flood than the other two. Detailed available water level records for the freshet calibration/verification floods are summarized in **Table 4.3** and gauge locations are shown in

Figure 4.2. **Figure 4.3** shows the 2002 calibration data for various continuous recording gauges along the Main Arm. These calibration/verification flows are only about 60% of the design flow at Mission of 18,900 m^3/s and hence a secondary calibration using a historic model was also carried out.

Few flow and water level records are available for the winter storm surge season. Following 1992, winter daily flows at Mission were not published and had to be estimated based on flow records for Fraser River at Hope, Harrison River near Harrison Hot Springs, Chehalis River near Harrison Mills and Chilliwack River at Vedder Crossing. Available winter calibration/verification data are shown in **Table 4.3**.

4.2.4 FLOW SPLIT DATA

Recorded flow spilt data is essential for confirming that the model correctly distributes flow into separate channels. In late May and early June of 2005, PWGSC collected flow split data using the ADCP discharge measurement method at Matsqui, Crescent, McMillan, Barnston and Douglas Islands as well as at the Trifurcation. Total flow during the period ranged from 6,000 m^3/s to 10,000 m^3/s. Transects at each location were repeated at least twice and then averaged. Good agreement was generally obtained between recorded total flows and the sum of splits, indicating good measurement accuracy.

In November, 2005 and January, 2006, PWGSC obtained further measurements at Sea Island, in the Albion Wall area as well as for the Woodward/Ladner reaches and Canoe Pass. Measurements were obtained during both ebb and flood tides with total flows in the 5,000 m^3/s to 8,000 m^3/s range. Recorded average flow splits are summarized in **Figure 4.4**. The flow distribution measured in 2005 is remarkably similar to the estimated values measured by Keane (1957) during the period May to August 1954, as summarized by the Inland Waters Directorate (1970).

4.3 2006 MODEL DEVELOPMENT

4.3.1 RIVER CROSS SECTIONS

A total of 615 sections were digitized and used in the 2006 model. Following initial model setup, some sections were added/removed, split or re-oriented in order to more accurately model flow conditions. The channel geometry for each cross-section was extracted from the TIN models by recording the elevation of the TIN surface at a user specified distance using an interpolation method in Arc/Info. Elevations were sampled every 3 m, coincident with the density of data used to create each TIN. Since MIKE11 has a limit of 1000 points per section, sections longer than 3 km (found at Pitt Lake and the tidal flats) were re-sampled at 9 metres. The points corresponding to each section were written to a text file and imported into ArcMap for processing. This reference information was read into MIKE11 from a text file using a Visual Basic routine. The GIS file was used to establish chainages and positional coordinates for network junctions, bridges, and water level gauges.

For initial model assembly, a uniform Manning's roughness coefficient of 0.03 was used for all channels. Approximate overbank values ranged from 0.06 to 0.15 and were determined using orthophotography provided by FBC or field observations.

4.3.2 HYDRAULIC STRUCTURES

Bridges were modelled using the energy method. In most instances river training structures were not specifically modelled but were used to limit effective channel areas. Flow passing through Albion Wall was modelled as a side branch. Roughness coefficients were increased to represent the pile wall and walers.

4.4 2006 MODEL WINTER CALIBRATION AND VERIFICATION

4.4.1 BOUNDARY CONDITIONS

The winter calibration and verification events were selected to represent periods of high ocean tide levels rather than high river flows. Only very limited winter water level data is available and it was not possible to calibrate or verify the model to the extreme tides listed in Section 3.3. Instead a two week period in November, 2002 with reasonably high tides and good water level records was chosen for calibration. The verification period was December, 2002 / January, 2003 when tides were higher but unfortunately water level records were few, though somewhat more extensive than generally available for the winter.

Upstream Inflow
During medium and low flow, the discharge at Mission is strongly affected by the tide. Daily flows have not been published by WSC at Mission since 1992, although estimated non-freshet daily flows were previously obtained for the period 1993 to 2001. For 2002 and 2003, non-freshet flows at Mission were estimated by adding the Fraser River discharge at Hope to recorded flows from the major tributaries between Hope and Mission (Harrison, Chehalis and Chilliwack, plus an allowance for local inflow). Based on daily flows, hourly hydrographs were generated for the calibration/verification periods in November and December of 2002 and January 2003. The November calibration flow at Mission was estimated to be about 1,500 m^3/s and the December/January verification flow just under 1,000 m^3/s. **Table 4.3** summarizes winter calibration and verification data. Both flows were slightly less than the corresponding monthly averages.

Tributary Inflow
Daily flow records were available for Stave, Coquitlam and Alouette Rivers for the calibration and verification periods. Pitt River flows were generated based on average flow ratios. The combined inflow from all four tributaries was roughly 100 m^3/s in November and close to 200 m^3/s in December. These flows are well below seasonal peaks.

Tidal Levels
The November peak tide reached a water level of 2.22 m at Point Atkinson resulting in maximum water levels of 2.24 m at Steveston gauge and 2.27 m in the North Arm at Vancouver South gauge. The corresponding ocean starting levels generated by Triton ranged from 2.22 m at the North Arm to 2.13 m at Canoe Pass, with a total swing range of 4.4 m.

During the December, 2002 and January, 2003 verification period, tides reached a maximum high water level of 2.42 m at Point Atkinson (January 3). Corresponding generated starting levels ranged from 2.42 m at the North Arm to 2.33 m at Canoe Pass, with a maximum swing of 4.75 m. The recorded Steveston peak level was 2.48 m.

4.4.2 Calibration and Verification

Winter calibration and verification results are provided in **Table 4.4**. The tabulated verification levels are for December 30, 2002, when water levels were available for the key gauge at Vancouver South. Plots of observed and modelled water levels are included in **Appendix D**. Modelled peak levels are generally within 0.15 m of observed levels.

Measurement errors were detected in the Nelson Road trough levels for November 2002 and in the Vancouver South trough levels for December 2002. Peak levels in the North Arm were under-predicted; by 0.26 m for the calibration and 0.15 m for the verification run. Observed water levels in the North Arm are sparse and the calibration is considered less accurate than for the Main Arm. It was not possible to rectify these discrepancies through roughness adjustments. The errors may have been the result of variations between estimated and actual ocean levels. Generally, the model agreed well with observed data and was considered valid for winter design conditions.

4.5 2006 Model Freshet Calibration and Verification

4.5.1 Boundary Conditions

Upstream Inflow
For freshet model calibration (2002 – maximum flow of 11,270 m^3/s) and verification (1999 and 1997, maximum flows of respectively 11,820 m^3/s and 12,180 m^3/s), recorded hourly flows were available for Mission and transferred to the upstream end of the model. During all three years, water levels at Mission were slightly influenced by tidal fluctuations, resulting in WSC reported flows fluctuating by up to 200 m^3/s. Since flows at Sumas Mountain would not have been tidal, the inflow hydrographs were mathematically smoothened.

Tributary Inflow
Recorded flows were available for three of the four main tributaries: Stave, Alouette and Coquitlam Rivers (**Table 4.3**). All three watersheds are regulated by BC Hydro Dams. Summer peak flows occur throughout the Fraser freshet season and do not show any correlation with the Fraser peak. Flow records for Pitt River ended in 1964. The unregulated Pitt basin correlates poorly with other watersheds in the area. Considering the relatively small contribution from Pitt River, a constant flow equal to the average summer base flow of 100 m^3/s was assumed for the freshet calibration and verification. Total tributary inflows during the 2002, 1999 and 1997 Fraser peaks were respectively 246 m^3/s, 280 m^3/s and 285 m^3/s. The tributaries typically contribute less than 3% of the total freshet flow. Local inflow and flows from smaller watersheds were ignored as their contribution is minimal.

Tidal Levels
By combining recorded surge levels at Point Atkinson and predicted tide levels at a number of locations in the area, Triton used their harmonic tidal model of Georgia Strait to estimate hourly water levels at the four outlet arms for 2002, 1999 and 1997 (see **Appendix E** for details). For the days corresponding to the peak flows, maximum tide levels at the South Arm outlet were respectively 1.44 m, 1.19 m and 1.39 m. Water levels at all outlets were within 0.1 m of the Point Atkinson level.

4.5.2 2002 Calibration

Results of the final calibration for the 2002 flood are listed in **Table 4.5**. The agreement with recorded peak levels is generally within the target accuracy of ±0.10 m, with an average absolute error of 0.09 m. Plots of observed and modelled water levels are provided in **Appendix D**. Some measurement errors were noted at the 192nd Street and Salmon River gauges. The Vancouver South and Bathslough gauges are within a short distance of each other, yet peak readings are typically about 0.15 m apart. The WSC Vancouver gauge is considered more reliable. Some uncertainty is associated with the ocean starting levels and it was not possible to more closely match peak levels at Steveston, under-predicted by 0.20 m. Tidal trough levels are not quite as well matched as peak levels (**Table 4.5**) but since the main purpose of the model is to simulate peak levels, the calibration was considered sufficiently accurate. The average absolute trough error was 0.15 m. Due to the two-dimensional nature of the flow, presence of a salt wedge, bed formations and wave action, the hydraulic conditions at the ocean interface are extremely complex and highly precise modelling using a one-dimensional model is difficult.

Estimated Manning's roughness coefficients are provided in **Table 4.6**. MIKE11 interpolates linearly between upstream and downstream roughness coefficients for intermittent cross-sections. Coefficients ranged from 0.025 to 0.033 in the main channels and were slightly higher in side channels (up to 0.035). For cross-sections downstream of Port Mann, the relative roughness coefficient was varied from 1.0 at peak tidal levels to 0.75 at trough levels to better match water levels at troughs. Within the ocean, where flow is partly over salt water, this reduced n-values to as low as 0.015 at tidal troughs. Over-bank roughness coefficients, estimated from air-photography, ranged from about 0.08 to 0.10.

Observed and calculated flow splits are listed in **Table 4.7**. In general, the agreement between observed and simulated splits is within a few percent. The observed Ladner/Canoe and North Arm/Middle Arm splits appear suspect and the model was not adjusted to try and match the observed splits. Measurements at these locations were obtained during winter conditions and are probably not representative of freshet flows.

4.5.3 1999 AND 1997 VERIFICATION

Following calibration, the model was verified using water levels recorded during the 1999 and 1997 floods. Good agreement was found between the 1999 flood levels as shown in **Table 4.5** (average error less than 0.1 m). Computed water levels were also compared with miscellaneous staff gauge readings obtained during the 1999 flood (Sigma, 1999). The levels generally agreed well, although some random discrepancies were noted, as shown in **Appendix D**. The 1997 peak levels upstream of New Westminster were over-predicted on average by 0.16 m (**Table 4.5**). The 1997 peak flow of 12,180 m^3/s was somewhat higher than the 2002 flood of 11,270 m^3/s, suggesting a possible reduction in roughness with increasing discharge.

Whereas the model calibration is accurate for discharges in the 11,000 to 12,000 m^3/s range, simulated water levels for the almost 70% larger design flow could vary from actual flood levels and further investigations were undertaken to try and estimate the roughness variation with flow.

4.6 HISTORIC MODEL DEVELOPMENT

4.6.1 AVAILABLE DATA

The historical model was developed for the reach extending from New Westminster to Mission to evaluate conditions during the large floods in 1948 and 1950. This model was developed from river surveys carried out in 1951 and 1952, collected by the Dominion Public Works Department. In the reach from New Westminster to Douglas Island, 1953 bathymetry was used, whereas the Pitt River channel cross-sections were extracted from more recent surveys. Overall, the model cross-sections used in the 1950's model are less accurate than the 2006 model. The old charts provided limited data along bank lines and were in some instances difficult to decipher.

Available flow and water level information used in the secondary calibration is listed in **Table 4.8**. The historic data are not as accurate as that used for the 2002 to 1997 calibration. Some flows were estimated values, rather than recorded and peak levels were typically not obtained from continuous recorders but in some instances correspond to high watermarks surveyed after floods. In spite of these limitations, the data represents hydraulic conditions during the second highest flood within historic times and provides the best means available for evaluating variations in roughness with flow.

4.6.2 MODEL ASSEMBLY

Assembly of the historic model involved digitizing the 1951,1952 and 1953 soundings from Sumas Mountain to New Westminster, geo-referencing the bathymetry and floodplain topographic maps and then extracting cross-sections. A spacing of about 1 km was used instead of the 400 m spacing in the 2006 model. Steps were taken to ensure that the 1950's historical model performed similarly to the 2006 model and that estimated roughness coefficients would be comparable. The 2006 model was modified to reflect the reduced number of cross-sections and using the 2002 calibration profile as a test, the simplified model was found not to significantly deviate from the high density cross-section model. Some junction locations were adjusted compared to the 2006 model to reflect the slightly different island configurations of the 1950's.

The 1950, 1969, and 1972 floods were contained by diking whereas the 1948 flood spilled on to the floodplain, with flow actively conveyed over bank. Therefore two versions of the model were assembled, one with flow confined to the main channel and the other allowing sections of the floodplain to actively convey flow. The two versions also allowed assessing dike confinement effects in the reach between Sumas Mountain and New Westminster.

The historic model extended only from near Sumas Mountain to New Westminster (including Pitt River) and was used specifically for assessing the 1948 and 1950 floods. The following boundary conditions were specified:

- Inflow at upstream end near Sumas Mountain.
- Inflow from Pitt River.
- Water levels at New Westminster tide gauge.

4.6.3 1948 FLOOD CALIBRATION

The model was calibrated to the 1948 water level profile using a Mission flow of 15,500 m^3/s (best estimate). Based on 1948 air photography, included in **Appendix A**, large portions of the floodplain were submerged. Assuming a roughness on the floodplain of 0.075 and a total flow at Mission of 15,500 m^3/s, the corresponding channel roughness coefficient would have been 0.027. The above analysis suggests that for the reach between Douglas Island (Km 47) and Mission (Km 85) the average channel roughness was 10% less than obtained through calibration of the 2002 flood. If no flow were conveyed over-bank (i.e. water ponded on the floodplain but was conveyed entirely within the main channel), the roughness may have been as low as 0.026. However, it is not possible to confirm the percentage of flow conveyed in the channel and it seems likely that at least some of the flow was over-bank and hence a coefficient of 0.027 was assumed. A comparison of observed and computed water levels at different locations along the channel is provided in **Table 4.8**.

4.6.4 1950 Flood Verification

The 1950 flood simulation was based on a discharge of 14,500 m^3/s (estimated by McNaughton, 1951). The flow was entirely contained within the dikes. It was found that the recorded and computed water levels at the Mission gauge matched for a channel roughness coefficient of 0.028. The observed and computed water levels are listed in **Table 4.8**.

Overall, the deviations from observed 1948 and 1950 levels are higher than for the 2006 model calibration. This was expected, considering potential inaccuracies in channel geometry, flow estimates, over-bank flow assumptions and lower quality high watermark information for the historic simulations. Also, a single average roughness coefficient was used for the entire New Westminster - Sumas Mountain reach, without effort to try and calibrate to other gauges than to the one at Mission.

4.6.5 1969 and 1972 Simulation

The 1969 flood had a peak flow of 9,660 m^3/s at Mission and the 1972 flood a flow of 13,650 m^3/s (based on a WSC discharge measurement rather than published flow). These intermediate flood flows were selected for simulation to see if roughness varies in a systematic way in the 10,000 to 15,500 m^3/s range. Some channel changes would have occurred between the early 1950's and 1972, but would not have been as significant as from 1972 to present. Fairly complete boundary condition data and observed water level information was available for the floods. A roughness coefficient of 0.030 was required for the 1969 flood and a coefficient of 0.029 for 1972.

4.6.6 Assessment of Dike Confinement Effects

The influence of dike construction between Mission and New Westminster on the 1894 design profile was assessed using the historic model. This assessment focused on confinement effects rather than flood attenuation due to storage. The historic model was used since it was considered more representative of the floodplain in 1894. Effects of dikes downstream of New Westminster were ignored. The historic model was run both with and without dikes and the results compared. For the runs without dikes, the maximum likely floodplain conveyance was assumed, giving the highest probable water level variation.

At the 1894 design discharge, the dikes raised the water level by a maximum of 0.4 m at Mission. Water level differences did not increase uniformly through the reach but showed irregular variations as a result of the dike configurations. For lower discharges, contained within the channel, the water level rise due to confinement diminished to zero. The results confirm that if present dikes were to withstand a flood of the same magnitude as in 1894, water levels at Mission could be up to 0.4 m higher than observed in 1894 due to dike confinement effects alone.

4.7 Review of Model Roughness

4.7.1 Assessment of Model Calibration

Figure 4.5 summarizes all available information on the relation between discharge and channel roughness for the reach downstream of Mission. The series of triangles corresponding to n=0.030 represents the overall best-fit relation for flows between 6,500 m^3/s and 11,300 m^3/s and is based on the model calibration for the continuous period of June 1 to July 1, 2002. The points labeled "1972 and 1974" represent estimated point roughness values determined using Water Survey of Canada's hydrometric measurements at Mission during flood conditions for these two years. The roughness values were derived by calibrating a truncated 2006 model for the Sumas Mountain - Whonock reach to Mission water levels, using recorded Whonock water levels as the downstream boundary condition and the Mission discharge as the upstream boundary condition. Most of these values fluctuate in the range between 0.032 and 0.028 and there is no obvious trend with flow. It is likely that some of the scatter in the 1972 and 1974 n-value estimates is due to the short reach between Whonock and Mission and the problem of estimating flow losses around Matsqui Island. Some additional source of uncertainty was also introduced by using the 2005 cross sections to represent conditions in 1972 and 1974. Therefore, although these results are indicative, they are believed to be less reliable than the 2002 calibration data. The filled circles on the graph represent roughness values estimated from floods in 1969, 1950 and 1948 using the historic model (1950's cross sections). The computed Manning roughness values decrease from 0.029 at a discharge of 13,700 m^3/s to 0.027 at 15,500 m^3/s. Error bounds (ranging from a high of 0.029 to a low of 0.026) are shown for the roughness value during the 1948 flood. These error bounds were determined by using the range of estimated peak discharges for the 1948 flood.

The plot suggests there is a weak trend for roughness to decrease when the flow exceeds approximately 12,000 m^3/s. These results show a similar trend as the theoretical predictions using the Engelund-Hansen equation (Section 3.5.3) except the measured roughness values are displaced higher than the theoretical predictions. Based on these results it was decided to adopt a channel roughness of 0.027 for the design flood condition of 18,900 m^3/s at Mission. This value was used for the reach extending from Mission downstream to near Port Mann/Douglas Island. Further downstream, the main channel roughness was set to average 0.03. There were two reasons for maintaining a slightly higher roughness value in the lower reach:

- The historic data is not adequate to estimate roughness values in 1948 or 1950 for the reach downstream of Port Mann. The calibration/verification results from 1997, 1999 and 2002 provide an average roughness value of 0.03 in this reach. Therefore, these values have been adopted.
- There is some indication that for the same river inflow conditions, the amplitude of the dunes in the lower reach is greater than in the upper reach near Mission. This is probably due to the greater tidal effects in the estuary. Consequently, it is reasonable to maintain a slightly higher roughness value in the lower reach.

Figure 4.6 compares the computed relation between discharge and water level at Mission along with WSC's 2006 updated rating curve (rating curve 4). The curve from the MIKE11 model represents an overall "best-fit" using the results from the 2002 freshet calibration runs and the predicted water levels for flows corresponding to the 1948, 1950, 1972 and adopted 1894 design flood condition. The model predictions and WSC rating curve match very closely over the entire range of measured discharge data between 6,000 m^3/s and 14,000 m^3/s. There is also good agreement at higher flood flows where the WSC rating curve is extrapolated. The revised WSC curve predicts a slightly higher water level than the model at design flood conditions (8.89 m model versus 9.05 m rating curve). Given the uncertainties involved in extrapolating the rating curve as well as the uncertainties in hydraulic roughness and sediment transport-induced changes in topography during extreme floods, it is believed that the differences between the rating curve and model are not significant.

4.7.2 ADOPTED ROUGHNESS VALUES

Table 4.6 summarizes the final roughness values that were adopted for the freshet design flood profile computations. The Resistance Radius method was used throughout the modelling except for the cross-sections within the ocean where the Total Area, Hydraulic Radius method was considered more appropriate. The values represent "best-estimates" rather than "upper/lower bound" limits of channel roughness.

The adopted roughness values are generally comparable to previous hydraulic modelling on the river. For example, Baird (1998, 1999) reported the main channel roughness ranged between 0.032 and 0.030. However, these values were estimated for moderate flows, not extreme flood conditions.

4.8 ASSESSMENT OF MODEL ACCURACY AND LIMITATIONS

MIKE11 is a one dimensional hydraulic model and does not take into account two or three dimensional flow effects. Saltwater intrusion in the lower estuary was not specifically modelled but indirectly accounted for in roughness coefficients. Flow at the ocean boundary is highly two-dimensional but was modelled one-dimensionally with cross-sections modified to incorporate increased storage areas. Based on calibration results these methods were found to be sufficiently accurate for the purposes of computing the flood profile.

The calibration and verification runs generally showed excellent agreement with measured water levels and were well within the normal limits for floodplain mapping and flood hazard assessment when compared to other rivers in British Columbia. The additional calibration and verification runs using 1948, 1950, 1969 and 1972 floods demonstrate that the model can be used for a wide range of flow conditions.

5. DESIGN FLOOD PROFILE

5.1 FRESHET PROFILE

5.1.1 BOUNDARY CONDITIONS

Following successful model calibration and verification, the model boundary conditions were set to design values. As specified in the terms of reference, a design inflow of 18,900 m^3/s was used at the upstream end of the model, corresponding to a discharge of 17,000 m^3/s at Hope (estimated to have occurred in 1894), plus local inflows between Hope and Mission estimated by UMA to be 1,900 m^3/s (lower bound value, taking into account the low probability of tributaries peaking at the same time).

Tributary design inflows from Stave, Pitt, Alouette and Coquitlam Rivers were estimated based on peak flow ratios in the same manner as UMA (2000) determined local inflows to the gravel-bed reach (Section 3.1.3). Results were summarized in **Table 3.2** and suggested a total flow of 19,650 m^3/s at New Westminster.

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). The levels roughly correspond to a two-year return period summer high tide (no surge). Since winter flood conditions exceed freshet levels at the downstream end, an in-depth analysis of summer tides was not carried out. The sensitivity of the freshet profile to ocean levels is described in Section 5.4.3.

5.1.2 SIMULATION RESULTS

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 MOE guidelines for floodplain mapping studies. However, unprotected floodplain areas, as covered by cross-section lines in Map No. 1, were included as actively conveying flow.

Flow levels at the bridges were reviewed and only one bridge, Jacob-Haldi Bridge at McMillan Island, was subject to pressure flow, with water touching the bridge deck but not over-flowing it. The performance of the river control structures was also reviewed.

Computed flow splits at the design flow were nearly the same as for the calibration/verification flows. The water level at Mission was found to be 8.9 m or 1.0 m higher than the design water level computed in 1969 which was equal to the observed 1894 water level. The design profile is plotted in **Drawing No. 34325-1** and tabulated in **Table 5.1**. Also listed in the table are the design level increases compared to the 1894 profile calculated in 1969.

5.1.3 UPSTREAM MODEL EXTENSION

The 2006 model ends at Sumas Mountain, approximately at the end of the tidal reach of the river, 10 km upstream of Mission. A MIKE11 model of the gravel-bed reach between Hope (Laidlaw) and Mission was developed by UMA in 1999. The two models were joined together, to provide a single model of the entire reach of the river from Hope to the Strait of Georgia. In order to combine the models, some technical details had to be resolved. The downstream model is fully hydrodynamic and computations are done at a time interval of two seconds. The upstream model uses the diffusive wave approximation method (neglecting velocity head) and a computational time step of one hour. The upstream model has a large number of linked channels, crossing islands and floodplain. The MIKE11 tool “pfs.MERGE” was used to join the models but some editing of the combined network and hydrodynamic parameter files was required. The joined model used a time step of 2 seconds and for a five-day modelling scenario the run took approximately 4 hours to complete.

The design flood profile from Laidlaw to Mission computed by UMA assumed a starting level of 7.99 m, corresponding to a water level of 7.92 m recorded in 1894 and a 0.07 m allowance for confinement effects caused by Matsqui dike upstream of Mission. Other downstream effects of diking were not taken into account. Upstream diking was modeled and assumed not to fail. These starting conditions were outlined in UMA’s terms of reference. However, the new downstream model showed that for the design flow, the upstream model starting level should be 8.9 m, or 0.9 m higher than the previously used level. This rise in the starting level has an impact on water levels up to Harrison Mills, as shown in **Drawing No. 34325-2**. Considering the much higher water levels in the reach upstream of Mission a review of the UMA model was undertaken. Cross-sections were generally sufficiently high to contain the flow. However, the backwater component, previously not included in the water surface computations, is likely more significant now. As specified in the terms of reference, this model was adopted unmodified for extending the 2006 model. There are several differences between the two models. The upstream model uses the diffusive wave solution, has frequent and crossing linked channels and cross-section bed-levels are highly irregular affecting relative roughness. The downstream model is fully hydrodynamic, has no linked channels and a smoother bed topography.

The revised profile for the Mission-Harrison Mills reach is plotted in **Drawing No. 34325-2** and is tabulated in **Table 5.2**. The extended model can be used as a flood-forecasting tool as described in Section 8. Predicted tide levels can be set as the downstream boundary condition and forecasted flood hydrographs used as upstream and tributary inflows. Water levels at any point and time along the river can then be computed.

5.2 WINTER DESIGN PROFILE

5.2.1 DESIGN BOUNDARY CONDITIONS

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 at the 95% upper confidence limit (**Appendix B**). Using a harmonic tidal model of Georgia Strait, Triton translated this elevation to a 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 MIKE11 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 (Section 3.3.2). **Table 3.5** summarized adopted tributary inflows downstream of Mission for the winter flood profile. 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.

5.2.2 Simulation Results

Simulated winter design profiles for the South, Middle and North Arms are included in **Drawing No. 34325-1** and tabulated in **Table 5.2** for the river reaches where winter design levels exceed freshet levels. The profiles in Middle Arm and Canoe Pass were essentially horizontal. **Figure 5.8** of Section 5.5.2 shows profiles corresponding to the 200, 100 and 20 year discharges and demonstrates that in the reach where the winter profile exceeds the freshet profile, the discharge used for modelling has almost no effect on the profile. A detailed joint frequency analysis of coinciding storm surge/ high flow events was therefore not required.

5.3 Combined Profiles

As specified, the freshet and winter profiles were combined and the higher of the two profiles adopted as the design profile. The design profile for the South, Middle and North Arms as well as Pitt River are shown in **Drawing No. 34325-1**. The point where the winter profile exceeds the freshet profile is roughly at Km 28 or about 1,400 m downstream of the Alex Fraser Bridge.

When the profile was plotted at each model cross-section, the computed water levels showed some irregularities caused by sudden energy losses at branch junctions and hydraulic structures. These sudden dips or spikes of up to about 20 cm are not directly representative of actual river conditions and the plotted design profile was graphically smoothened.

For comparison, the previous design profile, corresponding to the 1894 profile computed in 1969 is also shown in **Drawing No. 34325-1**. 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. Then, 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. Original and updated water levels at upstream and downstream boundaries of municipalities along the river are listed in **Table 5.1**.

5.4 Sensitivity Analysis-Freshet Conditions

The sensitivity of the design flood profile to variations in roughness, flow, ocean starting levels, and bed level changes due to scour was evaluated. This involved adjusting each variable and then determining the resulting deviation from the base profile.

5.4.1 Roughness

Computed water levels upstream from about New Westminster were found to be quite sensitive to the channel roughness coefficients used. A 10% roughness increase or decrease over the entire river system raised and lowered the water level at Mission by 0.57 m and 0.65 m respectively. A 10% variation in roughness corresponds to a range in values between 0.03 and 0.024 for the reach between Mission and New Westminster (base n value = 0.027). As discussed in Section 3.5.2, an n value of 0.024 is probably approaching a theoretical lower limit of roughness for the design discharge. A 20% universal increase raised the Mission water level by 1.13 m. Results are listed in **Table 5.3** and plotted in **Figure 5.1**. The variations correspond to standard values used for design profile studies.

A separate roughness sensitivity test was carried out for the historical 1950's model, by varying the roughness coefficient between 0.024 and 0.029 in 0.001 increments from Douglas Island to Sumas Mountain. Results are summarized in **Figure 5.2** and **Table 5.4**. There was no basis for varying roughness in this manner within the lower reaches. Historic bathymetry was not available and modelling of the 1948 and 1950 floods could not be performed. Section 4.7.1 described why roughness reductions in the lower reaches would be less likely than in the upper reach.

5.4.2 DISCHARGE

The model was also found to be very sensitive to discharge (**Table 5.5** and **Figure 5.3**). By increasing and decreasing the design inflow by 10%, the water level at Mission was raised by 0.63 m or lowered by 0.71 m. Relatively small adjustments to the design discharge can therefore significantly alter the design water levels. The tributary inflows downstream of Mission constitute a small percentage of the total flow. Therefore, differing assumptions about the timing and magnitude of these inflows will have only a minor effect on the computed water levels. The assumptions on these inflows that were used to develop the design flood are believed to be conservative. Since three of the four tributary watersheds are regulated, these design flows could conceivably be reduced somewhat from the assumed values.

5.4.3 STARTING OCEAN WATER LEVEL

Freshet water levels were found to be fairly insensitive to ocean level starting conditions. A 10% increase in tidal swing (equivalent to about 0.2 m) had very limited effect on water levels upstream of New Westminster (**Table 5.6** and **Figure 5.4**).

5.4.4 SCOUR AND BED LEVEL CHANGES

Local changes in bed level affect the profile only marginally. Regime bed scour elevations were estimated for the reach between Douglas Island and Mission at channel bends, narrow reaches and junctions (**Table 5.7**). This local bed lowering was entered in the model and found to reduce the design level at Mission by only 0.14 m (**Table 5.8** and **Figure 5.5**). The bed changes observed in this reach from 1991 to 2006 were also entered in the model and seen to only slightly affect the profile.

5.5 SENSITIVITY ANALYSIS-WINTER FLOOD CONDITIONS

Some of the sensitivity analyses were repeated to determine the effect of changes in roughness, flow and ocean level on the winter design profile. As was seen in **Drawing No.1**, the winter design profile is nearly horizontal and heavily backwater influenced.

5.5.1 ROUGHNESS

The roughness coefficients were varied by $\pm$10% which increased and decreased water levels by only 0.08 m at the upstream end of the reach where the winter design profile governs. The winter profile can therefore be considered quite insensitive to variations in roughness. Results are listed in **Table 5.9** and plotted in **Figure 5.6**.

5.5.2 DISCHARGE

The combined 200 year discharge of 12,690 m^3/s (Mission - 9,130 m^3/s and total tributaries – 3,560 m^3/s) was varied by $\pm$10%, which altered the water level at the winter/freshet profile transition point, downstream of New Westminster, by +0.09 m and -0.08 m. Profile variations are summarized in **Table 5.10** and **Figure 5.7**.

A separate discharge sensitivity test was undertaken to assess larger flow variations, corresponding to different return period inflows listed in **Table 3.5**. Profiles were computed for the combined 100 year flow of 11,590 m^3/s and 20 year flow of 9,060 m^3/s. Results are provided in **Table 5.11** and **Figure 5.8**. At the transition point, the 100-year winter flood resulted in a 0.07 m drop in the water level and the 20 year winter flood reduced the design level by 0.13 m. These variations are relatively small and the 200 year flow was considered appropriate for the design level.

5.5.3 STARTING OCEAN LEVEL

The design winter ocean starting level was varied by raising the entire tidal cycle by 0.6 m. This models the estimated change in water levels over the next one hundred years due to climate change and delta subsidence as discussed in detail in Section 7.3. Because of the very flat gradient of the winter profile, the starting level has considerable impact on the entire profile. At the transition point, defined by the base winter / freshet profiles, the water level was 0.55 m higher, which in turn shifted the actual transition point upstream by about 4 km, to approximately Km 33. Results are summarized in **Table 5.12** and **Figure 5.9**.

5.6 ASSESSMENT OF RESULTS

The investigations carried out in this study indicate that flow attenuation due to flood storage and overbank spilling between Hope and Mission during the 1894 flood event affected the magnitude of the discharge at Mission. The peak discharge downstream of Mission in 1894 was approximately 16,500 m^3/s. The adopted design discharge for the flood model is 18,900 m^3/s at Mission and 19,650 m^3/s at New Westminster. The 1894 historic flood profile is not directly comparable to the 2006 computed flood profile.

The channel and floodplain of the Fraser River has undergone significant changes over the last century due to the effects of dredging, river training and diking. These factors have certainly affected flood levels along the river although determining their exact magnitude is difficult. Section 4.6.6 estimated that confinement effects of diking between Mission and New Westminster may have increased the water level at Mission by about 0.4 m compared to undiked conditions. Bed degradation, discussed in Section 3.6.3 has lowered the water level at Mission in the order of 0.2 to 0.3 m. 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 (Section 3.1.1). 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. Instead, the following break-down supports a water level approximately equal to the modelled Mission level of 8.89 m:

- 1894 historic flood level: Elev. 7.92 m
- Confinement from dikes, Mission to New Westminster: Approx. +0.4 m
- Bed degradation Mission to New Westminster: Approx. - 0.2 m
- Loss of flood storage downstream of Hope: Approx. +0.7 m

Mission minimum design level for present river conditions: Approx. Elev. 8.82 m

It is also useful to compare computed MIKE11 water level at Mission with the range of predictions made using other simple methods:

- WSC rating curve extension: 9.05 m (Section 3.2.1)
- Hydraulic geometry analysis: 8.80 m to 8.5 m (Section 3.2.2)
- Fraser Basin Board (McNaughton): 8.60 m (Section 2.3)

All of these methods indicate that under the present conditions, a flood of 18,900 m^3/s at Mission would produce significantly higher water levels than was experienced in 1894.

The most appropriate way to express the uncertainty of the predicted flood levels is through the sensitivity analysis described in Section 5.4 and 5.5. The two most critical parameters affecting freshet water levels were the channel roughness (Manning n) and discharge. The influence these parameters on river stage varies with location, since the water surface profile is controlled at its downstream end by the level of the ocean and approaches near-uniform flow conditions upstream near Mission. As a result, the influence of uncertainties in discharge and channel roughness become more significant in the upstream direction. This effect is clearly seen on Figure 5.1 and 5.3. Increasing or decreasing the discharge by 10% has virtually no effect on water levels downstream of Steveston, raised or lowered water levels by approximately 0.4 m at New Westminster and raised or lowered water levels by +0.63 m to -0.71 m at Mission. The effect of changes in roughness along the profile was virtually the same. In the limit, as uniform flow conditions are approached near Mission, the sensitivity of river stage is governed by the form of the Manning equation. For uniform flow in a wide approximately rectangular channel the flow depth can be expressed as (assuming uniform, steady flow):

$$y = \left(\frac{nQ}{WS_f^{1/2}} \right)^{3/5} \tag{9}$$

where y is the flow depth, S_f is the water surface slope n is the Manning roughness value and W is the channel width.

Since flow depth is a function of both n and Q raised to a power of 3/5th, the effects of uncertainties in n and Q on flow depth are similar. The relative error in flow depth ($\Delta y/y$) will be related to uncertainties in discharge ($\Delta Q/Q$) and channel roughness ($\Delta n/n$) as follows:

$\Delta y/y = 3/5\ \Delta Q/Q$

$\Delta y/y = 3/5\ \Delta n/n$

This indicates a 10 % variation in discharge or roughness would be expected induce a 6% variation in flow depth. This represents the upper limit for the case of uniform flow which appears to be approached near the upstream end of the study reach near Mission. Typical mean depths in the sand-bed reach range between 10 to 14 m, so the variation in river stage should be in the order of 0.6 to 0.8 m, which is comparable to the results from the model simulations.

For the winter flood profile, water levels in the lower 28 km of the estuary are controlled by the level of the ocean and are virtually independent of the roughness or winter discharge. Fortunately, there is a relatively long record available for predicting the statistical properties of the ocean levels and the variability of maximum tide levels is relatively small. Applying a 95% confidence limit to the predicted 200-year ocean level changed the overall value by approximately 0.1 m.

6. ASSESSMENT OF DIKE FREEBOARD

This section of the report assesses the freeboard of the existing dikes under both an 1894 design flood condition and for 200 year winter ocean levels. The freeboard is also estimated for other historic flood events and other discharges.

6.1 FREEBOARD UNDER 1894 DESIGN FLOOD CONDITION

Crest elevations for dikes downstream of Sumas Mountain, are summarized in **Table 6.1**. The elevations were derived from various sources (**Table 6.2**), such as FRFCP Operation and Maintenance Manuals, municipal dike drawings or LIDAR, and represent the best data made available for this study. Actual dike crest elevations may vary and need to be verified by each diking authority. The tabulated elevations represent typical low points in the dike as well as the upstream and downstream limits of each dike. Also tabulated are modelled design water levels at the selected locations for a range of flows. Not all low points were included in the table and for a continuous assessment, the dike crest and water surface profiles plotted in **Drawings No. 3 – 11** should be referred to. The freeboard available at the selected locations is listed in **Table 6.3**, with a negative freeboard indicating the amount of overtopping that would occur with the specified river design flow. The winter profile does not include an allowance for water level rise due to global climate change or delta settlement.

Drawings No. 12 - 14 provide a similar comparison for diking upstream of Sumas Mountain, with results tabulated in **Table 6.4**. The dike crest elevations are the same as used by UMA (2001) and were provided by MOE. Also shown in the figures is the design flood profile computed using the UMA model and a starting level at Mission of 8.9 m, derived from the downstream model.

The dikes, designed to the 1894 flood profile computed in 1969, are generally inadequate. In some instances, the dikes would need to be raised by up to 1.3 m in order to provide a standard 0.6 m freeboard allowance.

6.2 COMPARISON OF DIKE CREST ELEVATIONS AND FLOOD PROFILES

As an initial evaluation of the flood protection capacity of present dikes, a series of water surface profiles corresponding to a range of discharges were computed using the MIKE11 model and were then compared to the 1894-profile computed in 1969, as shown in **Figure 6.1**. 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. A short distance downstream of New Westminster, the winter design profile determines the degree of protection offered by diking.

For a detailed assessment, it was initially assumed that the LIDAR data would provide sufficiently accurate and up-to-date profiles of the dike crests. However, with a vertical accuracy of $\pm$ 0.25 m, dike crest elevations were seen to vary irregularly. As an example, the Coquitlam Dike profile based on LIDAR was compared with the surveyed crest and seen to show some scatter (**Figure 6.2**). LIDAR was used for the Barnston, Silverdale, Albion, Fort Langley and Pitt Polder dikes. For all other diking, information was provided by MOE or various municipalities as indicated on the drawings.

Table 6.3 suggests that based on the dike crest elevations used for analysis, short segments of the Albion and Silverdale Dikes will overtop at discharges of less than 14,000 m^3/s. Freeboard at Glen Valley West Dike will be compromised at this flow and at Pitt Polder Dike at roughly 14,500 m^3/s (equivalent to the 1950 flood). At a flow just exceeding 16,000 m^3/s the dike is overtopped at dike chainage 6+850.

Freeboard for the Barnston Island dike is compromised at a flow just under 15,000 m^3/s. The same holds for a small segment of the Surrey dike at 15,500 m^3/s.

At the design flow of 18,900 m^3/s the dikes at Mission, Silverdale, Maple Ridge (Albion), Pitt Polder, Pitt Meadows (South), City of Coquitlam (Pitt), Matsqui, Glen Valley (East and West), Langley (Barnston, Fort Langley and West Langley) and Surrey would all be overtopped at one or more locations. In addition, freeboard would be compromised at the Pitt Meadows (North, North of Alouette and Middle), Port Coquitlam and Langley (CNR).

The existing diking system cannot convey a flood of 17,200 m^3/s at Mission (approximately equivalent to a 1948 flood event without flood spills or attenuation) with a freeboard of 0.6 m. Under this equivalent 1948 flood condition, six dikes would be overtopped and the freeboard would be compromised at six other dikes.

Dikes upstream of Sumas Mountain are also at risk. These dikes are outside the original study area and a detailed assessment of the exact magnitude of flow that reduces freeboard below 0.6 m was not carried out. However, from **Drawings No. 12 to 14**, it is evident that for the design condition, the Nicomen Island dike would be overtopped over most of its length, along with portions of diking at Kent (near Harrison River), Matsqui, Dewdney and Chilliwack.

For present winter design conditions (no ocean rise or delta settlement) freeboard would be inadequate in Delta (Westham Island, Marina Gardens and some sections of River Road), in Richmond (all except east end of Lulu Island), Surrey, Maple Ridge and at Pitt Meadows (Pitt Polder).

As the duration that water levels exceed freeboard increases, the risks of a dike failure also increases. Dikes, even with adequate freeboard, may fail due to seepage, piping and geotechnical conditions. The above analysis suggests that for both the freshet and winter design conditions, catastrophic flooding will occur along the Lower Fraser River.

7. FUTURE SCENARIOS

7.1 GENERAL

The calibrated model is a very useful tool for assessing the effect of different future scenarios on the flood profile. Future uses of the model include:

- Forecasting freshet flood levels for undertaking emergency measures or issuing evacuation orders (discussed in Detail in Section 8).
- Estimating effects of sedimentation, dredging and changed hydrological conditions on the flood profile.
- Assessing new flood mitigation options including rehabilitation of existing flood protection systems.
- Assessing impacts of future developments such as dikes, bridge abutments and hydraulic structures.
- Simulating dike breach scenarios in support of floodplain mapping.

This section describes two potential future effects by looking at how reduced dredging may change the river as well as what design profile changes may result from global sea-level rise.

7.2 EFFECT OF DREDGING ON FLOOD LEVELS

7.2.1 MORPHOLOGICAL RESPONSE TO REDUCED DREDGING ACTIVITY

Questions have arisen on the flood control benefits associated with navigation dredging along the Lower Fraser River. A complicating factor is that the river channel has been modified to such a large extent over the last 60 years by river training and jetties that it is difficult to measure the effect of dredging in isolation from these other works using the historical data that is available. Instead, this was addressed by assessing what would happen if dredging operations were significantly reduced in the future and a sensitivity analysis to rising bed levels was carried out involving:

- Identifying the reaches where dredging has been carried out over the last 20 years;
- Estimating the bed levels that would develop in each reach if dredging were curtailed, using the trends shown in **Appendix C** as a guide;
- Modifying the cross sections in the model and re-running the model for the design flood condition.

7.2.2 SHORT-TERM EFFECTS ON FLOOD PROFILE

Four separate runs were made for this analysis:

1. infill channel bed between Km 0 (Sandheads) to Km 11 (Steveston Cut)
2. infill channel bed between Km 0 (Sandheads) to Km 25.5 (Purfleet Point)
3. infill channel bed between Km 0 (Sandheads) to Km 31.5 (New Westminster)
4. infill channel bed between Km 0 (Sandheads) to Km 40 (Port Mann)

This provided a means to assess the relative importance of dredging in each reach to the overall changes in flood levels.

Figure 7.1 shows the computed difference in flood levels along the river. Raising the bed level in the lower reach (downstream of Steveston Cut) had the least impact on flood levels, resulting in a rise of approximately 0.1 m at New Westminster. Dredging operations at Steveston Cut, Steveston Bend and Sandheads have typically accounted for between 44% and 34% of total dredging effort since 1975 (**Table 7.1**). Raising the bed levels in all reaches from Sandheads to Port Mann resulted in an increase in flood stage of up to 0.4 m near New Westminster and Port Mann. The impact decreased further upstream, reaching approximately 0.1 m at Mission. This result is consistent with observed trends at the Mission gauge. This simulation represents the short-term (one or two years) effect caused by local infilling. Based on the previous morphological studies it is expected that the bed would gradually infill upstream of Port Mann again. Therefore, over a period of several decades the 0.4 m increase at New Westminster and Port Mann would eventually be experienced at Mission.

7.3 SEA LEVEL RISE

7.3.1 ASSUMPTIONS

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. The US Environmental Protection Agency (Titus and Narayanan, 1995) has provided probability-based estimates for various future scenarios. For the median case, sea level was estimated to rise 0.15 m by the year 2050 and 0.34 m by 2100 (corresponding to a rate of 3.4 mm/year). It was estimated there would be a 1% chance that climate change will raise the sea level by 1 m by the year 2100.

The Inter-Governmental Panel on Climate Change (IGPCC) issued predictions of changes to sea level in 1990, 1995 and 2001 for a range of future scenarios. However, the range in projections was very large. IGPCC, 2001 stated:

> *Projections of global average sea-level rise from 1990 to 2100, using a range of AOGCMs (*Atmosphere-Ocean Global Climate Models*) following the IS92a scenario lie in the range 0.11 to 0.77 m. This range reflects the systematic uncertainty of modelling. The main contributions to this sea-level rise are:*
>
> - *a thermal expansion of 0.11 to 0.43 m, accelerating through the 21st century;*
> - *a glacier contribution of 0.01 to 0.23 m;*
> - *a Greenland contribution of -0.02 to 0.09 m; and*
> - *an Antarctic contribution of -0.17 to 0.02 m.*

Results from more recent studies quoted in a report by the Ministry of Water Land and Air Protection (2002) provide an even greater range of global sea-level rise scenarios, ranging 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. In deltaic areas, ground subsidence may significantly affect local or relative sea-level differences. 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). Significant local variations are expected to occur. Church (2002) indicated at most locations the rate of vertical movement will be less than 3 mm/year.

Table 7.2 summarizes a range of potential sea level rise values for the Fraser delta and adjacent region from Church (2002). Net sea level rise in the 21st century was estimated to range from a low of 2.8 mm/year to a high of 5.6 mm/year. For the purposes of the sensitivity analyses of this study we have assumed a potential net rise of 0.6 m over the next century, which is on the higher side of the rates given in Church (2002) but well within the range of scenarios provided by IGPCC (2001). The actual sea level rise may vary.

7.3.2 Effect on Winter Flood Profile

Assuming a sea level rise of 0.6 m, the winter starting level was raised to 3.38 m. Computed water levels are listed in **Table 5.12** and plotted in **Figure 5.9**. The rise is nearly horizontal over the lower reaches and shifts the location where the winter and freshet profiles cross roughly 5 km upstream or roughly to the Trifurcation.

Selection of the final winter downstream boundary condition should be based on an economic risk assessment and the projected lifespan of the design profile. For a detailed assessment of dike elevations in the ocean reaches an evaluation of freeboard requirements, incorporating wave run-up and wind set-up should be completed.

The design profile does not include an allowance for sea level rise and will likely require updating over time.

7.3.3 Effect on Freshet Profile

When applied to the freshet design profile, a 0.6 m rise in ocean level (entire tidal cycle raised) 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 (just downstream of New Westminster at Km 28) were 0.33 m higher. The freshet flood profile was increased by 0.20 m at Barnston Island and merged with the base profile at Sumas Mountain. Consequently, adjustment of the freshet profile due to sea level rise may also be required.

8. FLOOD FORECASTING MODEL

8.1 PURPOSE

The terms of reference specified that the 2006 model be extended using the previously developed Laidlaw-Mission model to develop a flood forecasting tool for the Lower Fraser River, extending from Hope to the coast. The main purpose of this tool is to allow real-time predictions of water levels at any point along the river to support flood planning, preparedness and response. The primary use of the model is for freshet predictions but it can equally well be used for winter conditions.

The flood forecasting model is intended to be a "live" model, maintained by regular updates. The Sumas Mountain to Georgia Strait section of the model was found to be relatively insensitive to bed changes but UMA (2001) previously noted that the Laidlaw-Mission portion of the model is quite sensitive to bed changes and that it should be updated with new bathymetry at least every ten years.

DHI is continuously improving and expanding on the MIKE11 software. Older versions of the program are not necessarily compatible with newer versions and the model should preferably be kept up-to-date with newer software versions as they become available.

8.2 FLOOD FORECASTING INPUT REQUIREMENTS

For present river conditions the model is directly useable and only the boundary conditions need to be edited when applying the model in flood forecasting mode. The model boundary conditions include inflow at Hope, tributary flows from all major tributaries and ocean tide levels. Inflows at Hope can be based on predicted freshet flood hydrographs obtainable from MOE's River Forecast Centre in Victoria, which uses the UBC Watershed model to estimate freshet flows from basin snow-pack, precipitation, evaporation data etc. Alternatively, the model can be used for short term predictions based on actual recorded flows at Hope.

The major tributary downstream of Hope is Harrison River. UMA (2001) developed a UBC watershed model for Harrison River which can be used for estimating Harrison flows. Alternatively, Harrison flows can be predicted as a ratio of flows previously used for calibration or design. Design and calibration flows are listed in **Table 8.1** to give an indication of the magnitude of freshet flow that can be expected from each tributary basin.

Predicted tide levels are available for Point Atkinson from published tide tables or from the web at http://www.lau.chs-shc.gc.ca/english/Canada.shtml. These do not include water level increases due to local or surge conditions. Triton used a harmonic tidal model to adjust Point Atkinson data to the four outlet arms. The adjustments were found to be quite minor, of roughly 0.1 m or less, and considering the insensitivity of the freshet profile to ocean starting levels, the Point Atkinson data can be directly applied to the four arms without loss of accuracy.

8.3 USER INSTRUCTIONS

Detailed instructions for using the extended model in flood forecast mode are provided in **Appendix E**. The MIKE11 User Manual is in some respects not entirely user-friendly and the aim of the instructions in **Appendix E** is to clarify relevant sections of the MIKE11 manual and provide specific guidelines for running the Fraser model.

The MIKE11 model is formed of four distinct input files, which are combined in a simulation file. The four input files contain information on the river network, cross-sections, boundary conditions and hydraulic parameters. For running the model in flood forecast mode, typically only the boundary condition file and its associated time series file need to be revised. This editing is most easily done through the simulation file. The simulation file also specifies the time period of the run and boundary conditions must be available for the entire simulation period at all boundary locations.

The boundary condition data should preferably be hourly and the model run with a 2 second time step. The period of modelling can span the entire freshet period or only a few days. The computational time for a two week period (extended model) is roughly four hours. Generally the model must run for a duration of at least 6 hours (model) before the water surface computations have stabilized and results can be considered reliable.

In the flow range from 12,000 m^3/s to 15,500 m^3/s, the channel roughness was found to decrease (Douglas Island to Mission reach) from a Manning's coefficient of 0.030 to 0.027. For computing flood profiles in this range, the roughness is automatically adjusted within the model based on the average cross-section velocity.

8.4 SIMULATION RESULTS

Once the model has successfully been run, the output can be viewed in MIKE View, a software specifically provided for analyzing MIKE11 and other DHI program output. MIKE View provides a number of options for viewing the output as either plotted profiles or tabulated water levels. Discharge and other hydraulic parameters can also be viewed. Peak flood levels, the timing of peaks, the length of time water levels exceed a certain value and other important information can be directly extracted from MIKE View. Detailed instructions for MIKE View are also included in **Appendix E**.

As illustrated by the WSC rating curve for Mission, there are natural variations in the stage-discharge relationship and a certain flow does not necessarily result in the same water level all the time. There are fluctuations between the rising and falling limbs of freshets, seasonal variations and long-term systemic changes caused by permanent channel changes. Ideally, the model should be run in forecast mode for each freshet flood and the simulated profile compared with the observed profile following the flood. This would provide direct feed-back on the accuracy of the model and the need for new bathymetry to update cross-sections. During high flows, exceeding say 12,000 m^3/s such comparisons are essential. To ensure that future comparisons are feasible, flow and water level recording gauges must remain operational. Preferably, the gauging network should be expanded to include at least one additional gauge in the North Arm as well as a gauge between New Westminster and Steveston. Gauge requirements in the Laidlaw – Sumas Mountain reach were not assessed as part of this study.

The model is straightforward to run and operate. However, if given un-representative input or incorrectly modified, it will provide erroneous results. It is imperative that the model be operated by technically qualified persons only.

9. ADDITIONAL WORK

9.1 HYDROLOGICAL ASSESSMENT OF DESIGN FLOOD

The 1894 design discharge of 18,900 m^3/s at Mission appears to be a reasonable value for assessing flood hazards along the river. However, the actual level of risk associated with this flow under present hydrological conditions in the basin is unknown. Previous estimates of the return period of the 1894 flood have ranged from 160 years to over 500 years, based on the historic discharge data. A re-assessment of the frequency and magnitude of flood flows at Hope and Mission using the long-term flow records now available should be carried out.

However, given the potential effects of land-use change, climate change, flow regulation and other factors in the Fraser Basin, additional detailed hydrological analysis will be required to provide a reliable estimate of the frequency for a future flood equivalent to an 1894 event. We believe a detailed re-assessment of flood generation in the Fraser River basin is warranted. This analysis should include modelling runoff generation, flow routing through the various lakes and river network in the basin and simulating effects of flow regulation and diversions. The hydrological modelling should be capable of assessing the snowpack conditions and runoff generating conditions that are required for generating historic floods such as an 1894 event and 1948 event. The effect of potential land-use changes (due to changes in forest cover) or changes induced by climate change should also be assessed. The information from such a study would provide a better basis for assessing the level of risk associated with the design flood event.

9.2 DESIGN FREQUENCY AND FREEBOARD ASSESSMENT

In British Columbia, the 200 year flood (0.5% risk of exceedence) is commonly adopted for the design of dikes and for assessing flood hazards. On the Lower Fraser River, the 1894 flood of record has been used by the joint federal/provincial Fraser River Flood Control Program and the BC Ministry of Environment. Others, such as Indian and Northern Affairs Canada, use the 200 year flood level.

It is now common accepted practice that the level of flood protection should depend on the importance of the area being protected and the potential for loss of life and damage. In areas where there is a great threat to loss of life if there is a failure of flood control facilities, then much rarer design floods may be used. For example, in China along some sections of the Yellow River, the design discharge may range between 500 years to 2,000 years (0.2% to 0.005%). In the Netherlands, where much of the entire country is below sea level, very high standards of protection are used. In Central Holland, dikes are designed to carry the 10,000 year storm surge and river dikes are designed to carry the 1,250 year flood event. Poland uses the 1,000 year flood event for critical river levees.

The level of risk that is acceptable along the Lower Fraser River should be assessed on the basis of the potential damages and loss of life that could occur. The choice of the 1894 flood may prove to be very reasonable and appropriate, considering its historical significance and apparent rare frequency.

The requirements for freeboard along sea-dikes and river dikes should also be reviewed and assessed. The purpose of freeboard is to prevent overtopping of the dike caused by:

1. Waves
2. Wind setup
3. Tidal surges
4. Hydraulic jumps and standing waves in the channel
5. Super-elevation of the water surface in bends
6. Occurrence of unexpected higher water levels due to sedimentation or an increase in channel friction due to bedforms or vegetation
7. Effects of floating debris or ice
8. Settlement of the dikes or underlying floodplain
9. Other uncertainties in the hydrology and hydraulic conditions

Freeboard in British Columbia is typically specified to be 0.6 m. This value is in the range that is commonly specified in other regions or countries for "typical" conditions where the risk to loss of life or damage is minimal. However, the value of freeboard usually varies with several factors including:

1. Height of the dikes
2. Type of construction material used to build the dikes
3. Top width of the dikes
4. Velocity head and degree of curvature of the channel
5. Value of the land protected
6. Potential loss of life if the dikes were to fail

It is generally agreed that the amount of freeboard should be increased to protect areas with high value and high loss potential. A review of different freeboard requirements in other countries provided some examples of the range in values that have been adopted (McArthur, 1991):

Germany: The lowest allowable value is 0.8 m and can go up to 1.5 m to protect populated areas. A variety of very sophisticated methods are used for computing the design discharge, water surface elevation and freeboard.
Hungary: A fixed value of 1.0 m or 1.5 m is added to the design flood water surface elevation, depending on wave conditions and potential for erosion to the dikes.
Japan: The freeboard increases with the magnitude of the design discharge. For small streams (Q less than 200 m^3/s) the minimum freeboard is 0.6 m. For large rivers (Q greater than 10,000 m^3/s), a freeboard of 2.0 m is used.
Netherlands: Freeboard is computed using a detailed analysis related to the specific characteristics of the dikes and local hydraulic conditions. The minimum freeboard provided is always greater than or equal to 0.5 m. For sea dikes, the minimum freeboard is computed using the 2% significant wave run-up condition applied during the design high water event (10,000 year return period). Besides freeboard, a value for sea-level rise is added to the design height of the dikes to cover the design period (50-years).

There is inadequate information at present to determine the appropriate freeboard that should be provided along the Fraser River dikes and adjacent sea-dikes. It appears the commonly accepted value of 0.6 m is at the lower range of freeboard levels that are used in other highly developed urban areas.

9.3 Monitoring and Data Collection

For future re-calibration and verification of the model it is important that the present WSC and municipal water level recording stations are maintained and operated continuously during future flood events. The Mission gauge is particularly critical, since it provides both discharges and water level information. In recent years there has been some difficulty in obtaining published discharge information at this station, due to the additional effort and expense that is required to process the data. The WSC's efforts to improve the reliability and accuracy of discharge measurements should be supported. The water level gauges at Whonnock, Port Mann and New Westminster are important for future monitoring. The gauge at Port Hammond *(08MH043)*, which was discontinued should be re-activated, which would require additional funding. The gauges at Steveston and Vancouver South (North Arm) are critical for winter and freshet water level records. Preferably, a new gauge should be established between Vancouver South and the Trifurcation or alternatively, discontinued S*tation 08MH161, Fraser River (North Arm) below Tree Island* be activated. Similarly, *Station 08MH053, Fraser River at Deas Island* should be taken back into service.

9.4 INSTITUTIONAL GUIDELINES

A hydraulic model is only strictly valid for the conditions existing at the time of survey. If actual channel, floodplain and flow conditions begin to vary from the modelled conditions, the model results could become less representative. This reinforces the need for maintenance of the model.

Potential institutional arrangements were discussed in the hydraulic model scoping study (nhc, 2004). Different arrangements for model operation and maintenance were considered and funding issues, advantages and disadvantages associated with each option were discussed. The arrangements included FBC, MOE, PWGSC, WSC or a new River Management Authority acting as the lead agency. The Laidlaw-Mission model currently resides with MOE and hence MOE is a natural caretaker candidate of the Sumas Mountain-Georgia Strait extended model. However, on-going funding is required for this task and it is recommended that FBC continue to play catalyst, convenor and facilitation roles to coordinate model usage and updates. It would appear advantageous for FBC to continue facilitating the multi-interest and inter-disciplinary Technical Advisory Committee established for this present study.

A continued cost shared approach is recommended. The levels of government and relevant non-government and private sector sources of funding established for the present study should be further pursued as necessary. There may be opportunities for partners to pool resources, share in-kind technical resources and learn from the expertise of a range of partners. For example, there may be opportunities to collect up-to-date channel topography in conjunction with other bathymetric survey programs, floodplain mapping or modelling work.

A collaborative process would inevitably support a very broad dissemination of the results to share the benefits of this project. At present, there are a multitude of local, regional, provincial, federal, First Nations, private sector and non-governmental interests that can benefit from a continuous and up-to-date hydraulic model of the Lower Fraser River and related design flood profile. There are immediate and primary benefits for flood protection and floodplain management. However, there are also many secondary economic, social and environmental applications of the model.

Additional institutional and administrative arrangements are required to provide clarity and certainty regarding cost-sharing, roles and responsibilities, use of and access to the model, etc. Financial and administrative arrangements should be established to ensure continuous maintenance and operation. On-going funding options could be considered such as model license fees or fee-for-service payments to run specific model scenarios.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 CONCLUSIONS

1. The freeboard of the existing dikes along the Lower Fraser River is not adequate for the specified 1894 freshet flood and 200 year winter design flood conditions. Widespread dike overtopping and failure should be anticipated in the event these conditions occurred.

2. Flood levels in the lower 28 km reach of the river (downstream of Alex Fraser Bridge) are governed by winter high tides and storm surges. The estimated 200 year winter ocean level (astronomical tide plus storm surge) was estimated to be 2.9 m (GSC). This value does not include any freeboard allowance or provision for wave runup. Additional site specific analysis is required to estimate wave run-up along the sea dikes.

3. Flood levels upstream of Km 28 are governed by the freshet discharge. The computed water level at New Westminster is at 4.0 m GSC, which is 0.3 m above the historic 1894 flood profile established from previous studies in 1969. The computed level at Mission is at 8.9 m (GSC), which is 1 m higher than the historic 1894 flood level.

4. The design flood discharge used in this study was specified on the basis of previous investigations, and is intended to represent an 1894 flood event. A review was made to assess the physical changes to the river and its floodplain over the last century and the effects on peak discharges downstream of Mission. It was concluded that the 1894 design flood used in the model is higher than the actual peak discharge that occurred below Mission in 1894. Two factors could account for the higher discharges. First, the tributary inflows downstream of Hope in 1894 may not have been as large as the assumed inflows for the design flood condition. Second, the flood discharges in 1894 were attenuated downstream of Hope due to storage and retention of water on the floodplain, particularly in the reach between Agassiz and Sumas Mountain. Construction of dikes and reclamation of areas such as Sumas Lake have eliminated this flood attenuation effect.

5. The existing diking system cannot convey a flood of 17,200 m^3/s at Mission (approximately equivalent to a 1948 flood event without flood spills or attenuation) with a freeboard of 0.6 m. Under this equivalent 1948 flood condition, Pitt Polder dike would be overtopped and the freeboard would be only 0.2 to 0.3 m at nine other dikes.

6. Dredging and river training below Port Mann have caused substantial changes to the river over the last century, resulting in a narrower, deeper channel. The net effect has been to reduce freshet water levels by 0.4 to 0.6 m at Port Mann and by 0.2 to 0.3 m at Mission. Much of present-day dredging activity takes place in the lower reach of the river downstream of Steveston. If dredging were curtailed in the future in this reach, the impacts on winter or freshet flood levels would be very minor, since water levels are strongly controlled by the tide in this reach. Dredging upstream of Steveston has a greater impact on freshet water levels. Curtailing dredging in this reach in the future could cause freshet flood levels to increase by up to 0.4 m at Port Mann. The initial effect at Mission would be small (in the order of 0.1 m). Over a period of several decades it is expected the bed would gradually infill upstream of Port Mann and the water level rise at Mission would be similar to that at Port Mann.

7. Projected sea level changes due to climate change were reviewed. A 0.6 m increase (combined effect of sea level rise and delta subsidence) was modelled as a scenario. The corresponding ocean level (200 year return period combined storm surge and astronomical tide) was estimated to be 3.5 m (GSC).

8. The 2006 MIKE11 model was combined with an earlier model developed for the reach between Hope and Mission. The combined models provide a powerful flood forecasting tool for assessing flood hazards along the entire river system.

10.2 RECOMMENDATIONS

1. The results from this study show that widespread dike overtopping and dike failures would occur throughout the region in the event of an occurrence of the 1894 design flood. Municipal, provincial, federal and First Nation authorities should be alerted and advised of this situation.
2. High priority should be given to re-assessing the adopted design flood currently based on an estimate of the 1894 flood of record at Hope. This should involve conducting hydrological studies and hydro-meteorological modelling to determine the magnitude and frequency of flood flows in the Fraser River basin. The analysis should include simulations under present climatic conditions and anticipated future conditions to account for changes in climate and basin forest cover (due to potential effects of Mountain Pine Beetle infestation).
3. High priority should also be given to assessing both appropriate flood management strategies on the floodplain of the Fraser River and the institutional framework for implementation of those strategies. This should include both non-structural alternatives, such as floodplain zoning, and structural options like dike upgrading. The level of risk and appropriate design criteria for frequency and freeboard requirements for dikes and developments should be assessed.
4. The model should be re-calibrated and verified if another large flood occurs (equal or greater than a 1972 flood event). This could confirm the channel resistance coefficients used in the model. The model results are quite sensitive to variations in channel roughness. A 10% increase in roughness would for example, increase water levels at Mission by about 0.6 m. 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.
5. The hydrometric gauging network on the river is an essential component for flood forecasting applications and for model calibration and verification. Funds need to be secured to maintain and support the program, particularly operation of key discharge stations such as at Mission and Hope. Consideration should be given to expanding the system by re-activating gauges that have been shut down, in particular *Station 08MH043, Fraser River at Port Hammond;* S*tation 08MH161, Fraser River (North Arm) below Tree Island* and *Station 08MH053, Fraser River at Deas Island*.

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APPENDIX A
LOWER FRASER RIVER HYDRAULIC MODEL
SUMMARY RESULTS

Fraser Basin Council

Date: November 14, 2006

Lower Fraser River Hydraulic Model – Summary of Results

I. ISSUE

In September 2005, the Fraser Basin Council (FBC) retained Northwest Hydraulic Consultants Ltd. (nhc) to undertake a program of one-dimensional hydraulic modeling on the lower Fraser River using the MIKE 11 software developed by Danish Hydraulic Institute. The study area includes the lower Fraser River from the mouth of the Harrison River to the Strait of Georgia, encompassing the North, Middle and South Arms, including Canoe Pass, as well as Pitt River to Pitt Lake inlet. The overall objective was to generate an up-to-date design flood profile based on the following two scenarios:

- The 1894 Fraser River freshet flood combined with spring high tide conditions (Fraser freshet profile).
- The 1 in 200-year winter storm surge flood with winter high tide conditions combined with a Fraser River winter flow (the winter storm surge profile).

The modeled flood profile in 2006 is higher than the estimates that were made in 1969, which have been used as a basis to rehabilitate lower Fraser River dikes under the Fraser River Flood Control Program. The results from this study show that widespread dike overtopping and dike failures would occur throughout the region in the event of a re-occurrence of the 1894 flood of record.

II. BACKGROUND

In 2003, the Fraser Basin Council and the BC Ministry of Environment initiated a multi-year project to develop a hydraulic model of the lower river (focusing on the reach from Sumas Mountain to Richmond). The Fraser Basin Council is a nongovernmental, not-for-profit organization working in collaboration with others to resolve long-standing sustainability issues in the Fraser Basin. It should be stated at the outset that although the Fraser Basin Council has convened and facilitated numerous processes to assist in resolving Fraser River flood protection, gravel management and related issues, the Council has no jurisdiction or decision-making authority in these matters. The Fraser Basin Council along with provincial, federal, local governments and other partners have been pro-active in completing this study, as flood protection and dike safety are critical sustainability issues in the region. More than 20 Fraser Valley communities, including First Nations, are protected by over 300 km of Fraser River diking between Agassiz and Delta (including the sea dikes). Almost 250 km of these dikes were reconstructed by the federal/provincial Fraser River Flood Control Program between 1968 and 1994. The dike design levels (estimated flood water level plus 0.6 m freeboard) for reconstruction were established in 1969 by the federal Inland Waters Directorate. This profile was based on high water marks from the 1948 and 1894 floods, plus limited computer modeling. The 1894 flood is the largest flood in the last 112 years.

The main purpose of this project is to provide an up-to-date evaluation of the design flood profile for the lower Fraser River based on simulating a re-occurrence of the 1894 Fraser River flood of record, considering current river and floodplain conditions. Project objectives include:

- Update the dike design profile and assess the adequacy of existing diking systems;
- Better understand the effects of sedimentation and dredging on the dike design profile;
- Provide a flood level forecasting tool during spring freshet floods; and,
- Assist with land use planning decisions and floodproofing practices.

The project has been supported by financial and in-kind contributions from the BC Ministry of the Environment, Canadian Coast Guard, Public Works and Government Services Canada, Fraser River Port Authority and local governments, including the Greater Vancouver Regional District, Surrey, Richmond, Delta, Abbotsford, Township of Langley, Maple Ridge and Pitt Meadows.

III. FLOOD SCENARIO AND MODEL ASSUMPTIONS

The results of this study – an up-to-date design flood profile for the lower Fraser River – are based on the estimated 1894 Fraser River flood combined with high spring tide conditions (the Fraser freshet profile), and the 200-year winter storm surge with high tide combined with a Fraser River winter flow condition (the winter storm surge profile). The modeled Fraser freshet and winter storm surge profiles were overlaid and the higher of the two profiles was used to develop an overall design flood profile. The adopted design discharge for the 2006 flood model is based upon the 1894 flood of record with an estimated peak discharge of 17,000 m^3/s at Hope. This design discharge increases to 18,900 m^3/s at Mission and 19,600 m^3/s at New Westminster, when adding inflows for tributaries downstream of Hope. The adopted design discharge for the flood model assumes containment of the river by the existing dike system downstream of Hope under current and future floodplain conditions.

The hydraulic model was developed using field data collected in 2005, including comprehensive bathymetric surveys of the channel, LIDAR topographic surveys of the floodplain and ADCP velocity measurements to estimate flow splits at major channel branches. The model was calibrated and verified initially using recorded data from 2002, 1999 and 1997 high flow events. Peak discharges from these floods ranged between 11,000 cubic metres per second (m^3/s) and 12,200 m^3/s at Mission. Later, a secondary "historic model" was developed for the reach between Mission and New Westminster, using channel and floodplain topography from 1951 to 1953. This secondary model was used to estimate the channel roughness during floods in 1948, 1950, 1969 and 1972.

An assessment of floodplain conditions in 1894 was undertaken to estimate the flood attenuation, flood storage and over bank spilling effects during the 1894 flood and to estimate how these factors may have reduced the actual discharge and observed water levels at Mission during the flood. Taking into account these factors, the peak discharge in the river channel at Mission during the flood of 1894 was estimated to be 16,500 m^3/s. Also, actual tributary inflows below Hope during the 1894 flood may have been different than the assumed conditions adopted for the design flood in the model. For these reasons, it was concluded that the actual 1894 historic flood discharge and water levels are not directly comparable to the design flood profile computed by the model in 2006 using current river conditions.

Channel roughness (n-value) along sand bed rivers is subject to considerable variation with changing flow conditions due to the formation of sand dunes on the riverbed. A higher / lower n-value would result in a higher / lower flood profile. During very high flows the roughness may decrease substantially if the dunes wash out and flat bed conditions develop; however, field observations during floods in 1950, 1986 and 1997 showed no evidence of dunes washing out. The n-value was estimated to be 0.03 using the 2002 data (maximum discharge of 11,300 m^3/s at Mission). Based on flow estimates for the 1948 flood and observed water levels, the average n-value was found to be 0.027 or approximately 10% lower. Based on this assessment of Fraser River data, a value of 0.027 was adopted for the design flood profile computations between Douglas Island and Mission. However, there remains uncertainty about the appropriate n-value, which cannot be verified until the model is re-calibrated after a large flood event.

A statistical analysis of storm surges and astronomical tide levels was carried out to assess the design flood profile associated with a 1 in 200-year winter storm surge (at a 95% confidence interval). The Empirical Simulation Technique, developed by the US Army Corps of Engineers for the Federal Emergency Management Agency was used in this study. The Fraser River 200-year winter discharge of 9,180 m^3/s at Mission was used to estimate the winter flood profile. It was found that the winter flood level in the estuary was virtually independent of the discharge and was governed primarily by the ocean level. The final results for the winter storm surge profile assume no rise in sea level and no subsidence of the delta. The report does review recent published literature on sea level rise and local land subsidence. The report suggests a potential net rise of 0.6 metres over the next century.

IV. RESULTS

The winter storm surge profile exceeds the freshet profile in the lower 28 km of the river, or downstream from a point 1.4 km downstream of the Alex Fraser Bridge. Upstream of that point, the Fraser River freshet flood profile is the dominant flood hazard.

Comparisons were made between the flood profile computed in 2006 and that estimated and published from previous studies in 1969. The winter design profile downstream of the Alex Fraser Bridge 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.

- The results from this study show that widespread dike overtopping and dike failures would occur throughout the region in the event of a re-occurrence of the 1894 flood of record.
- The increase in predicted water levels suggests that dikes from Chilliwack and Kent to Surrey and Coquitlam would be overtopped at one or more locations. At the design flow of 18,900 m^3/s at Mission, the dikes at Chilliwack, Kent (downstream end), Matsqui, Dewdney, Mission, Silverdale, Pitt Polder, Pitt Meadows (South), City of Coquitlam (Coquitlam Diking District), City of Abbotsford (Matsqui and Glen Valley), Langley (Nathan and West Langley), Barnston Island and Surrey (Bridgeview and South Westminster) would all be overtopped at one or more locations. The Nicomen Island dike would be overtopped over most of its length. In addition, freeboard (see page 7) would be compromised at Pitt Meadows (North and Middle) and Langley (Salmon).
- For the winter storm surge flood, with the specified 200-year frequency the total water level was estimated to be 2.9 m GSC datum (at a 95% confidence interval). Freeboard would be inadequate at Delta (Westham Island, Fraser Shore and Marina Gardens) and Richmond (Fraser Shore and Lulu Island). The Delta dike at Fraser Shore would be overtopped at one location.
- Current dike elevations were derived from a variety of sources including recent LIDAR topographic surveys, recent surveys undertaken by diking authorities, or as-built dike crest elevations where recent survey data was unavailable. In some cases updated dike elevation data is warranted.

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 the freeboard for Pitt Polder Dike could 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 dike would be over-topped. Freeboard for the City of Coquitlam (Coquitlam Diking District) dike along the Pitt River would be compromised at a flow just over 15,500 m^3/s. The same holds for the Barnston Island dike and a small segment of the Surrey dike.

The final results of this project represent a comprehensive technical analysis using standard engineering practices, including calibration and verification utilizing data from different flood events. In addition the project used current computer modeling software, and best available data to calibrate and verify the model. In addition, confidence in the modeling has been supported by the development of a completely independent model (different software and river survey data), which produced similar results. It is now up to those responsible for managing flood hazards to determine how to apply this information, and what additional studies may be appropriate.

This briefing note represents a summary of key findings from this project, including the purpose, context, results and conclusions. A Final Technical Report will be available in December, which will include more details such as the methodology, sensitivity analyses, and several technical appendices.

V. CONCLUSIONS

The lower Fraser River and related flood hazard studies are highly complex. The engineers in 1969 simply did not have access to the sophisticated data gathering and analytical tools that are available today and could not explicitly deal with some of the river's complexities. Despite recent improvements in analytical tools, current studies also remain subject to sources of uncertainty. Federal, provincial, First Nations and local governments are advised to work collaboratively to examine the findings of this study and management options to help mitigate the Fraser River flood hazard, including costs and benefits.

In addition to applying these results to diking systems, it is also advisable that the following activities of local authorities take into consideration the final results:

- Establishing or refining local floodplain bylaws and/or flood construction levels;
- Updating Official Community Plans, development permit areas and other planning processes that relate to flood hazard management; and,
- Updating or refining local or regional emergency plans.

The Ministry of Environment under the *Dike Maintenance Act*, and through the office of the Inspector of Dikes, establishes standards for dike design, operation and maintenance, and issues approvals for changes to dikes and new dikes. Design criteria for dikes are updated from time to time based on the best available information. The Inspector of Dikes has advised that the study results and the new profile from Richmond to Chilliwack will now be adopted as the provincial standard for the Fraser River dikes.

However, significant further work is required before a major dike upgrading program is undertaken. Priority should be given to re-assessing the adopted design flow, which is currently based upon an estimate of the 1894 flood of record at Hope. This work should involve conducting hydrological studies and hydro-meteorological modeling to determine the magnitude and frequency of flood flows in the Fraser River Basin. The analysis should include simulations under present climatic conditions and anticipated future conditions to account for changes in climate and basin forest cover (due to potential effects of Mountain Pine Beetle infestation). The analysis should also include a risk analysis, which considers anticipated direct flood damages and indirect costs. The level of risk and appropriate design criteria for peak flow and associated freeboard requirements for dikes and developments should be assessed. Funding and governance arrangements for a major capital works program to rehabilitate the dikes and other mitigation options should be explored.

High priority should also be given to assessing all appropriate flood management strategies on the floodplain of the Fraser River and the institutional framework for implementation of those strategies. This should include the costs and benefits of both non-structural (land use planning, floodproofing practices, and emergency planning) and structural (flood protection dikes, erosion protection, river training structures, upstream storage, and dredging) flood management strategies.

The hydraulic model should be re-calibrated and verified if another large flood occurs (equal or greater than a 1972 flood event). This could confirm or revise the channel resistance coefficients used in the 2006 model. Model results are quite sensitive to variations in channel roughness. A 10% increase in roughness would, for example, increase water levels at Mission by a further 0.5 m. 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. The hydrometric gauging network on the river is an essential component for flood forecasting applications and for model calibration and verification. Secure funding is required to ensure these stations will be available in the future.

For more information about this issue, please contact Steve Litke, Program Manager, of the Fraser Basin Council's Integrated Flood Hazard Management Program at (604-488-5358) or slitke@fraserbasin.bc.ca.

VI. ATTACHMENTS

Summary of 2006 Modeled and 1969 Calculated Water Levels – Re-occurrence of the 1894 Flood Event

Municipality / Diking District	Location	2006 Modeled Water Level (m GSC)	1969 Calculated Water Level (m GSC)	Difference (m)
North Fraser				
City of Vancouver	West end UBC	2.88	2.62	0.26
	Burnaby border	3.03	2.68	0.35
City of Richmond	Sea Island: McDonald Slough	2.88	2.62	0.26
	Sea Island: West end at Middle Arm	2.89	2.62	0.27
	Middle and North Arm Confluence	2.89	2.62	0.27
	Terra Nova Park at Middle Arm	2.89	2.62	0.27
	New Westminster border	3.14	3.10	0.04
City of Burnaby	Vancouver border	3.03	2.68	0.35
	New Westminster border	3.26	3.17	0.09
City of New Westminster	Burnaby border	3.26	3.17	0.09
	Coquitlam border	4.24	3.95	0.29
City of Coquitlam	Burnaby border	4.24	3.95	0.29
	Port Coquitlam border	4.83	4.37	0.46
City of Port Coquitlam	Coquitlam border	4.83	4.37	0.46
	Pitt and Fraser Rivers confluence	5.08	4.50	0.58
	Pitt River at De Bouville Slough	4.90	4.57	0.33
District of Pitt Meadows	Pitt River at Sheridan Hill	4.90	4.57	0.33
	Pitt and Fraser Rivers confluence	5.08	4.50	0.58
	Maple Ridge border	6.05	5.11	0.94
District of Maple Ridge	Pitt Meadows border	6.05	5.11	0.94
	Whonnock Creek	7.69	6.71	0.98
	Mission border	7.96	7.00	0.96
District of Mission	Maple Ridge border	7.96	7.00	0.96
	Silverdale Creek	8.51	7.50	1.01
	Mission bridge	8.88	7.91	0.97
	FVRD border	9.30	8.40	0.90
FVRD	Mission border	9.30	8.40	0.90
	Nicomen Slough	9.84	9.05	0.79
South Fraser				
Corporation of Delta	Roberts Bank at Canoe Pass	2.87	2.59	0.28
	Massey Tunnel	2.93	2.65	0.28
	Surrey border	3.58	3.60	-0.02
	Westham Island: Roberts Bank at Canoe Pass	2.87	2.59	0.28
	Westham Island: Reifel Island at Ladner Reach	2.84	2.59	0.25
	Westham Island: Upstream end	2.88	2.59	0.29
City of Richmond	Steveston, Garry Point Park	2.84	2.59	0.25
	Massey Tunnel	2.93	2.71	0.22
	New Westminster Border	3.23	3.50	-0.27
City of New Westminster	City of Richmond Border	3.23	3.50	-0.27

	Trifurcation	3.84	3.60	0.24
City of Surrey	Delta border	3.58	3.60	-0.02
	Township of Langley border	5.91	5.05	0.86
Barnston Diking District	Barnston Island: downstream end	5.48	4.75	0.73
	Barnston Island: upstream end	6.03	5.11	0.92
Township of Langley	Surrey border	5.91	5.05	0.86
	Jacob-Haldi Bridge	6.97	5.95	1.02
	Abbotsford border	7.65	7.00	0.65
City of Abbotsford	Langley border	7.65	7.00	0.65
	Mission Bridge	8.88	7.91	0.97
	Sumas Mountain	9.65	8.84	0.81

Summary of 2006 Modeled and 2001 Modeled Water Levels – Re-occurrence of the 1894 Flood Event

Location	2006 Modeled Water Level (m GSC)	UMA 2001 Water Level (m GSC)	Difference (m)
Mission Bridge	8.89	7.99	0.90
D/S end Nicomen Slough	9.87	9.13	0.74
Confluence of Vedder Canal	10.29	9.61	0.68
D/S end Minto Channel	12.02	11.68	0.34
Confluence of Harrison River	13.69	13.52	0.17
Agassiz-Rosedale Bridge	18.80	18.79	0.01
U/S end Seabird Island	26.90	26.90	0.00
U/S extent of model	31.44	31.44	0.00

Relevant and Related Issues

Freeboard

Freeboard refers to an extra precaution that is typically added to the predicted design flood profile to account for uncertainty as well as other factors such as wave action. A freeboard of 0.6 m is typical in BC, meaning that dike crests are typically designed and constructed at an elevation 0.6 m higher than estimated water levels. When the expression "freeboard would be compromised" is used within this report, it refers to a circumstance where predicted water levels occur at an elevation that is within the 0.6 m of freeboard.

Dike Failure

Although this report and the final results of the 2006 hydraulic model identify specific locations where dike overtopping could occur, it is important to acknowledge that dikes can fail without being overtopped. For example, erosion, seepage, saturation and collapse are some of the processes by which dikes might fail prior to a situation where water levels overtop the crest of the dike. Therefore, it is important to emphasize that consideration be given to the overall strength and integrity of flood protection works in addition to concerns about overtopping of dikes by flood waters.

Sedimentation and Dredging

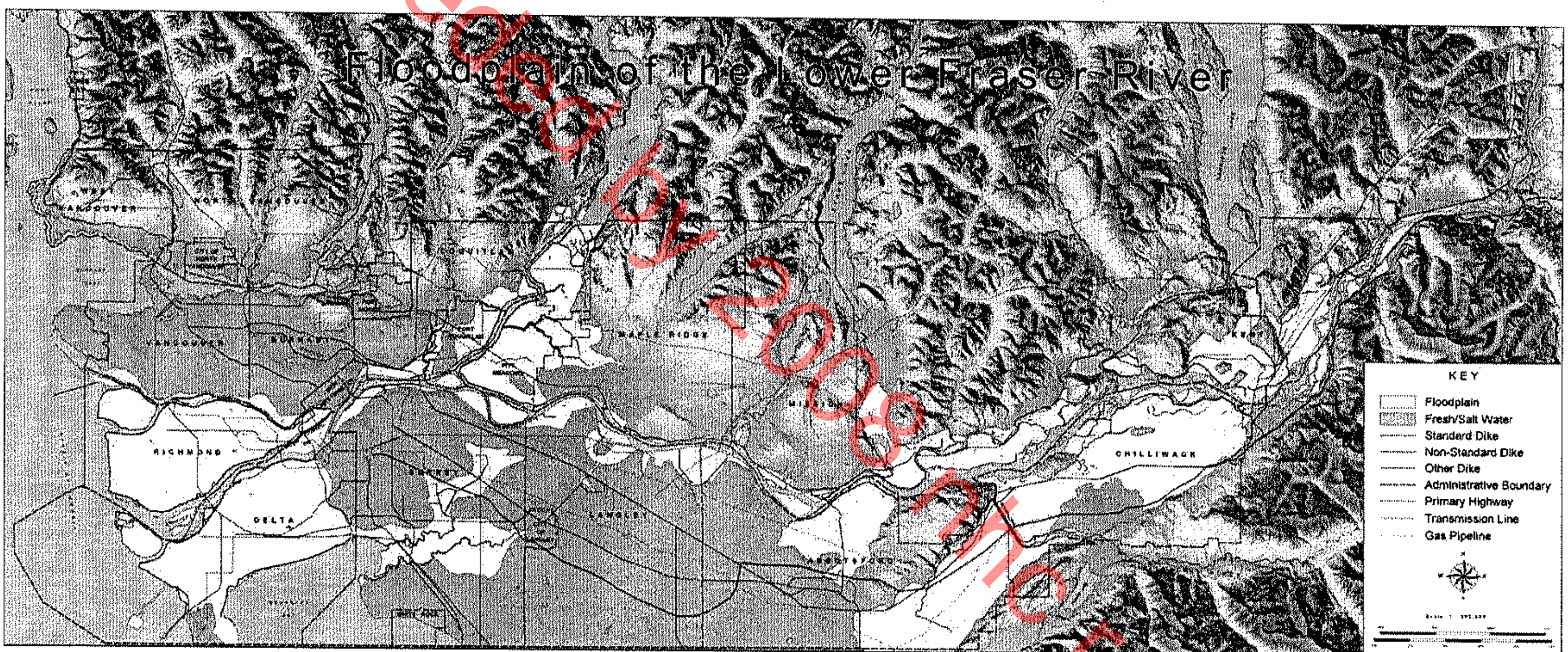
Some have cited dredging as a significant solution to managing the Fraser River flood risk. Dredging is a complex issue with varying relevance in different parts of the river. In some circumstances, rates of sedimentation of gravel and sand on the riverbed can have an influence on the flood profile, and similarly, dredging can – in some circumstances – contribute towards flood hazard management.

For example, river surveys and flood modeling in the Fraser River gravel reach (Hope to Mission) found that an increase in gravel deposition on the riverbed contributed to a rise in the flood profile. It is believed that gravel dredging may be a partial flood protection solution in this part of the river along with dike rehabilitation. If dredging is appropriate and approved, it should occur in strategic locations where flood protection benefits would result, and undertaken in a way that is sensitive to habitat and the environment. After substantial analysis and dialogue, a five-year agreement was signed by provincial and federal regulatory authorities to guide planning and decision-making with respect to gravel removal in the lower Fraser River. This agreement, developed with the best available technical information, outlines the timing, volumes and requirements of the permit approval process.

Although a comprehensive analysis of dredging scenarios was not within the scope of the 2006 hydraulic model study, sedimentation and dredging processes were considered within the sensitivity analyses. Regime bed scour elevations were estimated for the reach between Douglas Island and Mission, resulting in a minimal (0.13 m) reduction in predicted water levels. The effect of dredging and sedimentation on the flood profile was analyzed by simulating a significant reduction of dredging in the future and a subsequent rise in bed levels. For this analysis, bed levels were generally increased by 1 or 2 metres from Sandheads to Port Mann, resulting in an increase in predicted water levels of up to 0.4m at Port Mann and 0.1m at Mission.

Sedimentation and dredging processes appear to be even less influential in the lowest 28 km of the Fraser River, where the more significant flood threat is related to ocean storm surge events. Although dredging of the river channel might reduce the flood profile associated with the Fraser River freshet, it would not reduce the profile associated with the 1 in 200-year ocean winter storm surge, which is the dominant flood event that needs to be managed in this reach. The estimated peak water level (2.9 m GSC datum) is directly correlated to sea level, and would not be lowered with dredging of the channel.

Floodplain of the Lower Fraser River



Notes

- This map is provided for reference.
- The study area includes the lower Fraser River from the mouth of the Harrison River to the Strait of Georgia, encompassing the North, Middle and South Arms, including Canoe Pass, as well as Pitt River to the Pitt Lake inlet.

Appendix B

THE UNIVERSITY OF BRITISH COLUMBIA

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22 December, 2006

Mr Steve Litke
Fraser Basin Council
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Vancouver, BC V6C 1V5

Dear Steve:

Re: Review and Acceptance of the *Lower Fraser River Hydraulic Model Final Report* by northwest hydraulic consultants and Triton Consultants Ltd, December 2006

Over the last several years I have been fortunate to have been involved in several important studies on the Fraser River including:

- Co-Author with Dr Dilip Barua, Fraser River Model Scoping Study, 1999;
- Reviewer, Design Flood Profile Studies for the Fraser River Gravel Reach, conducted by UMA Engineering Ltd., 2000 and 2001;
- Research Supervisor, Mr. Faizal Yousef, M.A.Sc., "Application of a two-dimensional hydrodynamic model to the Fraser River Gravel reach", 2001;
- Co-Investigator, together with Drs Michael Church (UBC) and Ted Hickin (SFU), 4-year research project "Sediment transport models for lower Fraser River: tools for sustainable management" funded by the *Natural Sciences and Engineering Research Council of Canada.*

Throughout the course of the current study by northwest hydraulic consultants (nhc) and Triton Consultants Ltd., I have reviewed results as they developed, attended several Progress and Technical Committee meetings, and reviewed and commented on Progress and Draft reports, and Memoranda. As a consequence, I consider that I possess good knowledge and understanding of the current study, and am well qualified to provide a review.

In summary, this study represents an outstanding technical contribution that will form the basis for flood management and protection, and flood forecasting along the lower Fraser River for decades to come. The report is comprehensive and very well written, and the assumptions, limitations and uncertainties clearly and explicitly presented. I have complete confidence in the technical analyses and modeling. Based on current understanding and available data, I fully accept the study results including the updated Design Flood Profiles, Conclusions, Recommendations and Suggestions for Future Work.

It is not my intention here to provide a detailed point-by-point review. Many of my comments and concerns have already been incorporated into the Final Report. I would like to comment on some important aspects of the study, particularly as related to the Winter and Summer Flood Profiles.

Winter Flood Profile

The analysis for the winter flood profile was primarily conducted by Triton Consultants Ltd. The approach used for this component of the study is referred to as Empirical Estimation Technique, and comprises both deterministic (tidal) and probabilistic (storm surge) components. It is the preferred method of the major US agencies responsible for coastal flood protection, and represents a significant improvement over previous, purely statistical analyses used to derive design flood levels for the lowermost Fraser River.

One aspect of the study that is noteworthy is recommendation of the 95% confidence limit for estimation of the statistical component of the winter flood profile analysis, rather than using the mean estimate (Appendix B, Tables 14 and 15). Unlike a deterministic analysis used for the tidal component, the statistical approach for the storm surge does not result in a single answer, but rather a range of values with different levels of confidence. The 95% estimate means that there is a 95% probability that the true 200-year storm surge water level is less than or equal to the estimate, while for the mean estimate there is a 50% probability. Looking at it another way, with the 95% estimate there is only a 5% chance that the 200-year storm surge level is greater than the estimate, whereas for the mean value there is a 50% probability.

The 95% estimate is clearly more conservative than the mean estimate. In this instance the 95% estimate for the 200-year storm surge is equivalent to the 500-year mean estimate. In turns out that the Winter Design Flood Profile for the lowermost Fraser River is entirely controlled by the water level in the Strait of Georgia, and is not materially influenced by the discharge in the Fraser River. I strongly support the more conservative 95% estimate recommended and used in this study to develop the Winter Design Flood Profile. In my opinion, a design flood level based on the mean value for the 200-year water level would not provide adequate protection for the City of Richmond. The term "1-in-200 year flood" can be quite misleading. In any one year while there is only a 0.5% (1/200) probability that this level will be exceeded, over a period of say 20 years, the probability increases to almost 10%, and over 50 years, the probability increases to about 22%, or almost 1 in 4. Of course, the freeboard requirement does provide a substantial additional margin of safety, given that the difference between the 200 year and 1000 year storm surge estimates is some 0.34m, based on the 95% estimate.

I strongly support the call for additional assessment of design frequency and freeboard assessment in British Columbia (Section 9.2).

Summer (Freshet) Flood Profile

I was quite surprised by the results for the Freshet Design Flood Profile, which indicates an increase in the design water level by about 1m at Mission. The higher water level can be accounted for through a combination of differences between the actual 1894 discharge and the current Design Discharge. nhc show quite convincingly (Section 3.2.2) that the actual 1894 peak discharge at Mission is likely to have been in the order of 16,500m^3/s, rather than the 18,900m^3/s assumed for the Design Flood. This is supported by historical reports (page 14). Based on the rating curve developed using the MIKE 11 model for current channel conditions (Figure 4.6), this would indicate a water level at Mission of just over 8m GSC for 16,500m^3/s, compared to the observed 1894 water level of 7.92m GSC. The increase in the Design Flood Level at Mission from 7.72m GSC to 8.89m GSC therefore represents largely the effects of the higher discharge assumed for the current Design Flood (18,900m^3/s vs 16,500m^3/s), as well as changes in the channel conditions including channel alignment, dikes and bed lowering.

Model sensitivity analysis (Section 5.4) demonstrates the large influence of the channel roughness parameter (Manning's n) and the discharge on the Design Flood Profile. During the course of this study there has been debate, at times vigorous, regarding the appropriate value of Manning's n for the Design Flood conditions (18,900m^3/s). Calibration of the MIKE 11 model to historic floods shows a decreasing trend in Manning's n with increasing discharge (Table 1).

Table 1. Summary of Manning's n values obtained by nhc by calibrating the MIKE 11 model to historic floods (values taken from Sections 4.6.3-4.6.5)

Flood	Peak Discharge (m3/s)	Manning's n
1969	9,660	0.030
1972	13,650	0.029
1951	14,500	0.028
1948	15,500	0.027
Design	18,900	?

nhc have quite reasonably assumed a value of 0.027 for the Design Flood because this corresponds to the value obtained through calibration to the highest discharge (1948 flood), which most closely represents Design Flood conditions. Extrapolation of the trend out to 18,900m^3/s would suggest a value of Manning's n of some 0.024, or perhaps 10% less than the value used to compute the Design Freshet Profile. Figure 5.1 indicates that reducing Manning's n values by 10% would lower the water level at Mission by perhaps 0.7m. However, I fully accept the value of Manning's n used in the study and the resultant profile. In the absence of hydraulic measurements collected during flows greater than the 1948 flood, there is no basis for reducing channel roughness below 0.027 at this time.

In addition, the computed design water level at Mission draws strong and independent support from the revised rating curve (WSC Curve 4) recently developed by Water Survey of Canada (Figure 4.6 and Appendix D). Based on a revised extrapolation of observed flood levels, this new rating curve projects a water level of just over 9m GSC for a discharge of 18,900m^3/s, compared to 8.89m GSC obtained using the calibrated MIKE 11 model.

The study also shows a large sensitivity of the flood profile to changes in the discharge (Figure 5.3). A 10% reduction in the inflow discharge at Mission would reduce the profile by 0.7m or so (see also page 38). In my opinion the current Design Discharge of 18,900m^3/s is reasonable and well justified. However, I do strongly support the call for additional work (Section 9.1) to re-examine the hydrological basis of the design flood.

In summary, I am pleased to be able to provide this Letter of Acceptance for the *Lower Fraser River Hydraulic Model Final Report*. northwest hydraulic consultants and Triton Consultants are to be congratulated for completing a far reaching and outstanding study.

Sincerely,

Dr Robert Millar, P.Eng./P. Geo.
Associate Professor and Hydrotechnical Group Leader

APPENDIX C
FLOOD ATTENUATION: HOPE TO MISSION IN 1894 AND 1948

1. BACKGROUND

This appendix describes preliminary investigations to estimate the flood peak attenuation downstream of Hope that occurred in 1894. An additional comparative analysis was done for the 1948 flood. Results were previously presented in a memorandum to FBC prepared by **nhc** on July 24, 2006.

2. METHODOLOGY

The following main floodplain storage areas between Hope and Mission were identified and the topographic information shown in parentheses obtained:

- Matsqui Prairie (2004 LiDAR provided by FBC)
- Sumas Prairie (1994 0.5 m contours provided by MOE)
- Chilliwack (contours/spot elevations provided by City of Chilliwack)
- Hatzic (2004 LiDAR and topography provided by FVRD)
- Nicomen Island (2004 LiDAR and topography provided by FVRD)
- Kent (2004 LiDAR)
- Seabird Island (1999 LiDAR from gravel-reach study)

Using the available topography, storage-elevation tables were developed for each area. Channel storage was not included in the analysis. Top of bank profiles (excluding diking) were extracted for the riverbank along the floodplain areas.

Daily water levels were not recorded in 1894 but a peak flood level of 7.92 m GSC was observed at Mission. Based on newspaper reports describing the rise and fall of flood levels at different locations in the valley, an approximate water level hydrograph was developed for Mission. The hydrograph was then transferred to the different floodplain areas based on 1894 high watermarks and staff gauge rating curves developed in 1969 by Inland Waters Branch. Using the elevation-storage tables and extracted bank profiles, the floodplain storage available for a particular daily water level at each area was estimated. The equivalent mean daily inflow/outflow required to match the change in storage volume was computed and adjusted according to the progression of breaching dikes and floodplain inundation described in newspaper reports. Daily storage flows for the different areas were combined. The computations were then repeated for the 1948 flood.

Only limited water level and floodplain information is available from 1894, and a number of simplifying assumptions were made in the analysis, as described in Section 4. The method is approximate and is intended primarily to provide indicative results that demonstrate whether or not floodplain storage was a significant factor in 1894 and 1948.

3. FLOODPLAIN STORAGE

Storage area/volume tables derived from the available topography are summarized in Table 1. The Sumas and Chilliwack areas provided the largest volumes. However, Kent with a smaller floodplain storage, allowed Fraser River flows to drain into Harrison Lake, reducing downstream flows. At the time of the flood peak, the following total storage volumes were estimated, based on approximate flood levels at each site:

Floodplain Area	Storage (10^6 m^3)
Matsqui	145
Sumas	567
Chilliwack	375
Hatzic	49
Nicomen	157
Kent	152
Seabird	73
TOTAL:	**1,518**

Assuming a ten-day period from bank-full conditions to maximum flood levels, a constant flow of 1,750 m^3/s would be required to fill this volume. However, flow going into storage was not constant and typically there would have been a large amount of flow draining on to the floodplain as the bank was overtopped or berm/dike breached, followed by a much reduced flow that gradually filled the floodplain, raising ponded water levels to roughly the same elevation as the river. In order to estimate actual storage flows at the 1894 peak, the progression of the flood was reviewed in detail.

4. 1894 FLOOD

4.1 Flood Hydrograph

Figure 1 shows water levels at Mission for the three highest recorded floods of 1948, 1950 and 1972. For comparison, the 2002 calibration hydrograph is included. Also shown are estimated 1894 peak levels, derived from newspaper accounts of the flood. The levels are approximate, but reflect the observed peak water level and the fairly well documented duration of high levels. The hydrographs illustrate two distinct shapes, with the 1894 and 1950 floods having high peak levels but relatively short durations and the 1948, 1972 and 2002 floods maintaining high water levels for long periods.

A maximum flow of 17,000 m^3/s was estimated at Hope for 1894 (Fraser River Board, 1958). The flood was caused by a record snow pack during the winter of 1893-94 and snow remaining from the previous winter that failed to melt at high elevations during the cold summer of 1893.

In order to estimate the rise and fall of water levels at Mission and to determine the timing and amount of flow going into key storage areas, a detailed progression of the 1894 flood was prepared as shown in Table 2.

4.2 Estimated Storage

The estimated Mission hydrograph was transferred to the midpoint of each floodplain storage area based on a few available 1894 high watermarks, select rating curves derived in 1960's and 70's and the 1894 profile calculated in 1969 by Inland Waters Branch. To some extent, the rating curves and calculated profile account for dike confinement but were considered more representative of un-diked conditions than the UMA (2001) modelled profile.

The elevation corresponding to when overflow onto the floodplain began was extracted from the bank profiles at each floodplain area. The top-of-bank elevations typically fluctuate and an average elevation was selected at mid-reach. These estimated bank-full elevations are approximate and could result in substantial under- or overestimation of initial storage flows. However, around the peak of the 1894 flood, the bank elevations would have had less impact on the amount of flow going into storage. It was assumed that water levels on the floodplain would equal those in the river. Bank-full and peak flood elevations at storage area midpoints are listed in Table 3.

The daily change in storage volume was obtained from the storage-elevation tables based on the increase or decrease in water levels. The mean daily flow into or out of storage equivalent to the storage change was calculated and results are provided in Table 4. According to historic accounts, the floodplain became inundated very quickly once the low berms in place in 1894 were overtopped. However, based on simple weir calculations some areas could not have filled as quickly as implied by the river water level change and storage flows were adjusted as shown in Table 4.

Based on the above simplified method, the total mean daily flow going into storage between Hope and Mission during the flood peak on June 5, 1894 was estimated to be roughly 1,300 m^3/s. In addition, some flow was diverted into Harrison Lake across the Kent floodplain.

4.3 Effect of Harrison Lake

McMullen (1988) described how the 1948 Fraser River flood flows entered Harrison Lake through Kent. The area was sparsely populated in 1894 and there is little information regarding the flood at that time. References provided by McMullen suggested that the Harrison Lake level was at least 0.3 m to 0.5 m higher in 1894 than in 1948. According to Septer (2000), the Harrison River rose by 3 m on May 29,1894 and the small village of Harrison was completely submerged.

Using the Kent LiDAR topography, a simplified HEC-RAS model was assembled to compute the amount of flow that could have entered Harrison Lake in 1894. District of Kent slopes towards the lake and has a minimum width between higher ground of 1,030 m. The lake design water level is 13.9 m where as the 1894 profile, calculated in 1969, ranges from about 20 m at the upstream end of Kent to 17 m at the downstream end. Assuming a differential head of 3 m and a floodplain Manning's roughness coefficient of 0.06 up to 2000 m^3/s could have flown into the lake. However, considering flow obstructions (railway embankment, forested areas and any buildings) the flow was likely much less.

Accurate estimation of the amount of flow that went into Harrison Lake is difficult without knowing what the floodplain looked like at the time and more detailed modelling. The lake has an area of roughly 225 km^2 and inflows must have rapidly increased lake levels. Flow draining down Harrison River would have increased and re-entered the Fraser at Harrison Mills but this outflow would have been less than the inflow considering the geometry of the lake outlet and the head differential. For the water balance in the next section, a Harrison Lake diversion flow of 1,000 m^3/s is assumed.

4.4 Water Balance

Based on the above storage computations and assumed Harrison Lake diversion flow, a simple flow budget for the 1894 flood peak was prepared as follows:

Peak flow at Hope:	17,000 m^3/s
Local inflow, Hope to Mission:	1,900 m^3/s (design inflow)
Un-attenuated peak flow at Mission:	18,900 m^3
Flow to storage, Hope to Mission:	1,300 m^3/s
Diversion to Harrison Lake:	1,000 m^3/s (say)
Actual flow at Mission:	16,600 m^3/s
Local inflow, Mission to New Westminster:	750 m^3/s
Peak flow at New Westminster:	17,350 m^3/s

Using the above values, the peak flow at New Westminster is about 7% higher than the flow estimate of 16,200 m^3/s provided in the Daily Columbian on June 5, 1894. Local inflows between Hope and Mission may have varied considerably. Similarly, flows into Harrison Lake may have been quite different. Yet, this simple water balance suggests that a discharge estimate of 17,000 m^3/s at Hope for 1894 is reasonable. For present conditions, assuming no floodplain storage attenuation and no flow diversion into Harrison Lake, a design discharge of 19,650 m^3/s at New Westminster is not unrealistic.

5. 1948 FLOOD

A peak flow of 15,180 m^3/s was recorded at Hope on May 31, 1948. McNaughton (June, 1951) estimated the corresponding peak discharge at Mission to be 15,840 m^3/s. A maximum water level of 7.61 m was recorded on June 10, 1948. Water levels at Mission remained above 7 m for 19 days, from May 29 to June 16, or about twice as long as in 1894 (Figure 1). The flood was a result of above normal snow pack and a late, cold spring. May was wetter than average and near the beginning of the month there was a sudden rise in temperatures. A summary of the flood is provided in Table 5.

McNaughton (June, 1951 and October, 1951) described a significant shift in the Mission stage-discharge relationship due to sediment deposition during the falling limb of the 1948 flood hydrograph. He estimated the flow corresponding to the recorded maximum water level of 7.61 m to be 14,500 m^3/s, whereas the estimated maximum flow of 15,840 m^3/s had a corresponding water level of 7.49 m. The flows are approximate and were computed based on flow at Hope, estimated local inflows and flows going into storage. However, evidence of a rating-curve shift was provided.

5.1 Estimated Storage

Following the same methodology as for the 1894 flood, the change in floodplain storage and corresponding floodplain inflow/outflow was computed as listed in Table 6. During the peak flow on May 31 an estimated 1,050 m^3/s went into storage. However, due to the shape of the 1948 hydrograph, the available floodplain storage had essentially been filled by June 10 and at the time of the peak level, storage flows were only in the order of 180 m^3/s.

5.2 Effect of Harrison Lake

On May 27, 1948 the Fraser River overtopped the railway west of Agassiz, ponded in the Mountain Slough area and flooded northward into the Village of Harrison Hot Springs. This process continued until June 4 (McMullen, 1988). Water depths in low-lying areas were reported to have reached 3 m. An estimate of the flow going into Harrison Lake was not provided but a maximum lake level of 13.3 m was published by WSC.

McNaughton (August, 1951) estimated the flow entering Harrison Lake across Kent to be 400 m^3/s on May 31, 1948 reducing to 30 m^3/s on June 10, 1948. From descriptions of the flood it is not clear if the railroad across Kent was overtopped and/or washed out.

5.3 Water Balance

According to McNaughton (June, 1951) local inflow between Hope and Mission on May 31, 1948 was 2,200 m^3/s. In view of the 1,900 m^3/s design inflow derived by UMA (2001) this flow seems high. A total inflow of 1,700 m^3/s was assumed based on the design Hope/local flow ratio resulting in the following water balance:

Peak flow at Hope:	15,180 m^3/s (May 31)
Approx. local inflow, Hope to Mission:	1,700 m^3/s (say)
Un-attenuated peak flow at Mission:	16,880 m^3/s
Flow to storage, Hope to Mission:	1,050 m^3/s
Diversion to Harrison Lake:	400 m^3/s (McNaughton)
Estimated Discharge at Mission:	15,430 m^3/s
Maximum flow at Mission (McNaughton):	15,840 m^3/s

Actual local inflows and storage/diversion flows may have varied considerably but again the water balance suggests that storage flows provided attenuation of the Mission 1948 peak flow. Ten days later, at the recorded maximum water level, flows going into floodplain storage and into Harrison Lake were much reduced.

6. SUMMARY AND CONCLUSIONS

The above review suggests an 1894 discharge of 16,600 m^3/s at Mission corresponding to the observed water level of 7.92 m. The 1948 maximum flow was estimated to be 15,400 m^3/s at a water level of 7.49 m. In view of the flood attenuation that took place in 1894 due to floodplain storage and flow diversion to Harrison Lake, the flow estimate of 17,000 m^3/s at Hope appears reasonable.

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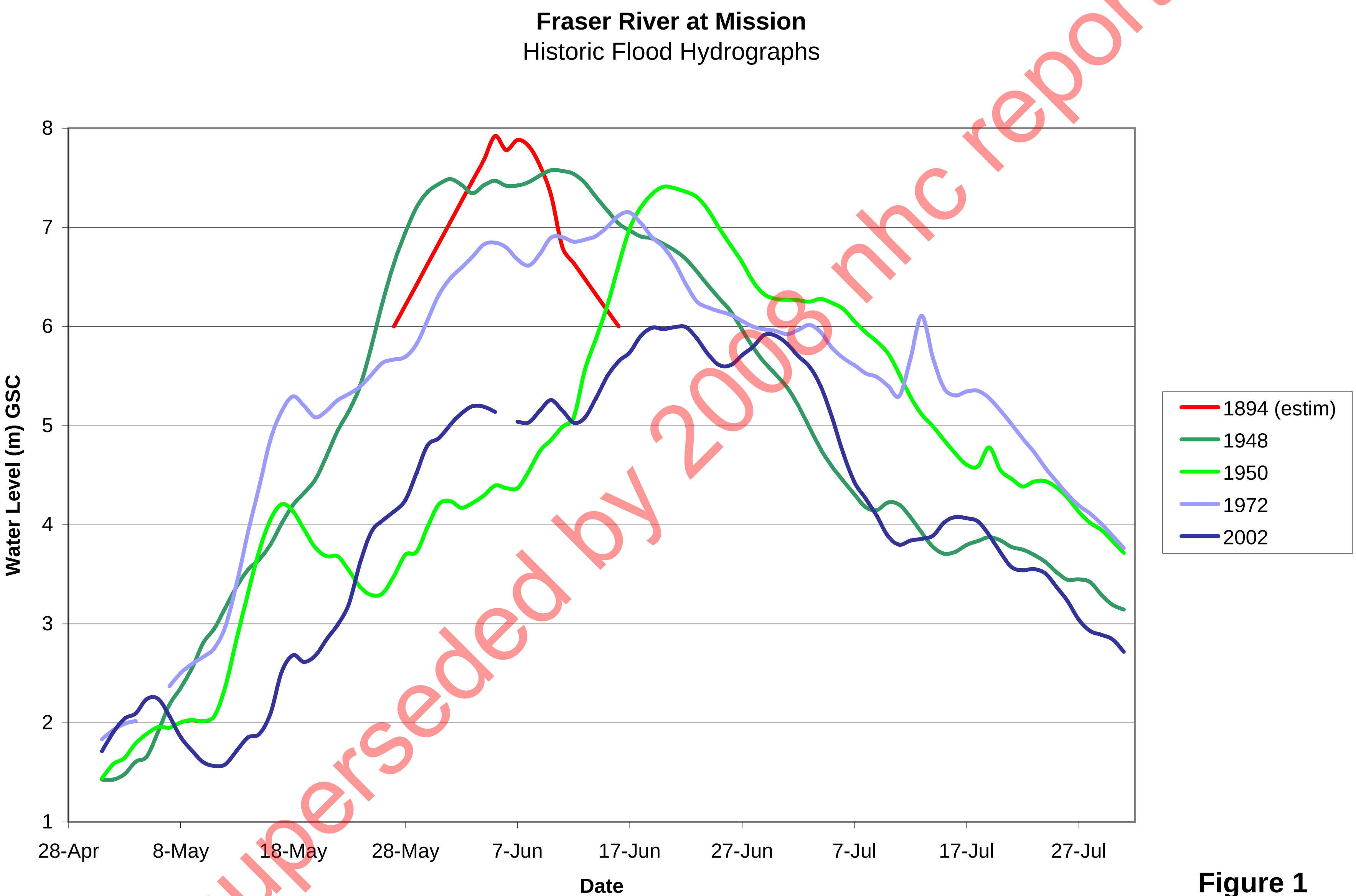
1948 Flood – Port Mann to Barnston Island
Source: BC Gov't Special Project



Fraser River at Mission
Historic Flood Hydrographs
Water Level (m) GSC
8
7
6
5
4
3
2
1
28-Apr
8-May
18-May
28-May
7-Jun
17-Jun
27-Jun
7-Jul
17-Jul
27-Jul
Date
1894 (estim)
1948
1950
1972
2002

Figure 1

Table 1: Floodplain Areas and Volumes

Matsqui			Sumas			Chilliwack			Hatzic			Nicomen			Kent			Seabird		
WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)	WL (m)	Area 10^6 (m^2)	Volume 10^6 (m^3)
1	0	0.00	0.5	1.33	0.76	1	0.00	0.00	1	0.00	0.00	2	0.00	0.00	9	0.00	0.00	14	0.00	0.00
1.5	0.0075	0.00	1.0	4.81	2.60	2	0.02	0.01	2	0.00	0.00	3	0.00	0.00	10	0.07	0.01	15	0.00	0.00
2	0.2456	0.05	1.5	10.64	7.25	3	0.11	0.05	3	0.01	0.01	4	0.23	0.08	11	0.34	0.18	16	0.04	0.02
2.5	0.5135	0.23	2.0	18.73	15.55	4	1.25	0.55	4	0.02	0.02	5	1.32	0.76	12	1.19	0.82	17	0.50	0.27
3	1.4247	0.65	2.5	27.11	27.95	5	5.45	3.89	5	0.10	0.07	6	3.28	2.84	13	5.39	3.73	18	1.84	1.54
3.5	4.8285	2.08	3.0	35.88	44.77	6	10.59	11.61	6	1.17	0.52	7	14.50	9.84	14	13.20	12.95	19	3.96	4.62
4	12.0366	6.17	3.5	42.34	65.02	7	17.57	26.32	7	14.32	8.30	8	40.18	39.59	15	20.23	29.80	20	7.31	10.50
4.5	20.1891	14.25	4.0	49.55	88.70	8	26.98	48.03	8	25.96	29.49	9	44.75	82.26	16	26.01	53.10	21	11.66	20.53
5	28.6505	26.49	4.5	56.32	115.90	9	43.82	84.01	9	27.59	56.62	10	47.11	128.38	17	31.04	81.63	22	14.78	34.11
5.5	35.6911	42.67	5.0	62.69	146.24	10	62.07	136.86	10	27.70	84.26	11	47.75	175.90	18	35.24	114.97	23	17.50	50.59
6	40.2332	61.76	5.5	68.78	179.73	11	84.07	211.49	11	27.73	111.98	12	47.85	223.71	19	37.67	151.56	24	18.90	69.08
7	43.8437	104.21	6.0	74.11	215.96	12	103.18	304.73							20	38.72	189.84	25	19.36	88.25
8	45.1311	148.74	6.5	79.49	254.87	13	125.26	421.95							21	38.97	228.73	26	19.48	107.71
			7.0	84.48	296.36	14	140.09	554.39										27	19.49	127.20
			8.0	92.40	385.70	15	152.51	702.54												
			9.0	96.77	481.04	16	159.09	858.44												
			10.0	98.00	576.00	17	163.93	1020.98												
						18	166.37	1186.17												

Note: Active storage range shown in red.

Table 2: 1894 Flood Progression

Date	Description
26-May	Hatzic dike failed, 300 m length of embankment was swept away and a large wave of water rolled in over the prairie.
27-May	Langley dike failed, in 5 minutes 400 ha were flooded. I.R. on McMillan Island was flooded.
28-May	Matsqui Dike failed, a large section of embankment collapsed in the morning and by 2pm the prairie was covered with water. River rising more slowly at 5cm/hr.
29-May	Hope Slough dike failed resulting in extensive flooding of Chilliwack (1000's of acres). Nicomen Island flooded by 1.2m, Matsqui Prairie submerged by 2m. Mission Bridge approaches washed out. Water in Harrison River rose by 3m and Harrison Village was submerged. River overflowed banks at Chilliwack landing and reached Centreville Village.
30-May	Annacis Island was flooded.
31-May	Water levels nearly same as in 1882 at Matsqui and Langley.(1)
1-Jun	Mission water levels increased 8cm overnight and 30 cm during the day. Matsqui flooded to foothills.
2-Jun	Water level at New Westminster slightly higher than in 1882. Pitt Meadows dike failed.
3-Jun	Central portion of Lulu Island flooded, initially through dike breach near upstream end, later from high tide levels at downstream end.
4-Jun	Westham Island dike failed. Water level at Langley 25cm higher than in 1882.
5-Jun	No1 Dike at Pitt Meadows failed, Deas Island dike failed. Water level dropped 2cm at Chilliwack since previous day but rose 4cm at New Westminster.
6-Jun	Westham Island bank failed. Slight water level reduction at Langley and Mission. At Chilliwack water dropped by 14 cm.
7-Jun	Ladner dike failed. Chilliwack water level dropped by 5cm.
8-Jun	Water level at Mission dropped by 6cm.
9-Jun	Chilliwack water level has dropped 23cm from the peak level.
10-Jun	Water levels continue to drop but Upper Sumas still accessed by boat.
11-Jun	Overnight water dropped 13cm at New Westminster and 46cm at Chilliwack.
14-Jun	Train service resumed.

(1) 1882 Flood level at Mission was 7.34m.

Table 3: Storage Area Bankfull and Flood Levels

Storage Area	Mid-point Chainage (km)	Bankfull Elevation (m)	Calc 1894 WL (m)	WL Range (m)
Matsqui	85	6.0	7.92	1.92
Sumas	102	7.2	9.90	2.70
Chilliwack	116	10.4	12.60	2.20
Hatzic	93	6.8	8.70	1.90
Nicomen	105	7.9	10.60	2.70
Kent	131	17.0	19.00	2.00
Seabird	143	22.0	24.20	2.20

Table 4: Estimated Mean Daily Storage Volumes and Flows for 1894

	Matsqui				Sumas					Chilliwack					Hatzic				Nicomen				Kent				Seabird				Total	Total
Date	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	Adjusted Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	Adjusted Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	WL (m)	Storage Volume 10^6 (m^3)	Change in Storage 10^6 (m^3)	Mean Daily Storage Flow (m^3/s)	Mean Daily Storage Flow (m^3/s)	Storage Volume 10^6 (m^3)
27-May	6.00	61.8	61.8	715	7.20	314.2	314.2	3637	637	10.40	166.7	166.7	1930	0	6.80	6.7	6.7	78	7.90	36.6	36.6	424	17.00	81.6	81.6	945	22.00	34.1	34.1	395	3193	701.8
28-May	6.21	70.7	8.9	103	7.50	340.6	26.4	305	1305	10.64	184.7	18.0	208	100	7.01	8.5	1.7	20	8.20	47.9	11.3	131	17.22	88.9	7.3	84	22.24	38.1	4.0	46	1789	779.3
29-May	6.42	79.6	8.9	103	7.79	367.0	26.4	305	1305	10.88	202.6	18.0	208	1000	7.22	12.9	4.4	51	8.49	60.5	12.6	146	17.44	96.2	7.3	84	22.48	42.0	4.0	46	2735	860.9
30-May	6.63	88.5	8.9	103	8.09	393.9	26.9	311	1311	11.12	222.9	20.2	234	548	7.42	17.3	4.4	51	8.79	73.1	12.6	146	17.66	103.5	7.3	84	22.72	46.0	4.0	46	2289	945.2
31-May	6.84	97.4	8.9	103	8.38	422.0	28.2	326	326	11.36	245.3	22.4	260	570	7.63	21.7	4.4	51	9.08	86.0	12.9	149	17.88	110.8	7.3	84	22.96	50.0	4.0	46	1329	1033.2
1-Jun	7.05	106.4	9.0	104	8.68	450.2	28.2	326	326	11.60	267.7	22.4	260	570	7.84	26.1	4.4	51	9.38	99.6	13.6	158	18.09	118.4	7.6	88	23.20	54.3	4.4	51	1347	1122.8
2-Jun	7.26	115.8	9.4	108	8.97	478.4	28.2	326	326	11.84	290.2	22.4	260	570	8.05	30.8	4.7	54	9.67	113.3	13.6	158	18.31	126.4	8.0	93	23.44	58.8	4.4	51	1360	1213.5
3-Jun	7.47	125.1	9.4	108	9.27	506.4	28.1	325	325	12.08	314.6	24.5	283	283	8.25	36.4	5.6	65	9.97	126.9	13.6	158	18.53	134.4	8.0	93	23.68	63.2	4.4	51	1083	1307.1
4-Jun	7.68	134.5	9.4	108	9.56	534.5	28.0	325	325	12.33	342.8	28.2	326	326	8.46	42.0	5.6	65	10.26	140.9	14.0	162	18.75	142.4	8.0	93	23.93	67.7	4.4	51	1130	1404.8
5-Jun	**7.92**	145.2	10.7	124	**9.90**	566.5	32.0	371	371	**12.60**	375.1	32.2	373	373	**8.70**	48.5	6.4	75	**10.60**	156.9	16.0	186	**19.00**	151.6	9.1	106	**24.20**	72.9	5.2	60	**1294**	**1516.6**
6-Jun	7.78	138.9	-6.2	-72	9.70	547.8	-18.7	-216	-216	12.44	356.3	-18.8	-218	-218	8.56	44.7	-3.8	-43	10.40	147.5	-9.4	-108	18.85	146.2	-5.3	-62	24.04	69.8	-3.1	-36	-756	1451.3
7-Jun	7.88	143.4	4.5	52	9.84	561.2	13.4	155	155	12.55	369.7	13.4	155	155	8.66	47.4	2.7	31	10.54	154.2	6.7	77	18.96	150.0	3.8	44	24.15	71.9	2.1	25	539	1497.8
8-Jun	7.82	140.7	-2.7	-31	9.76	553.2	-8.0	-93	-93	12.49	361.6	-8.1	-93	-93	8.60	45.8	-1.6	-19	10.46	150.2	-4.0	-46	18.90	147.7	-2.3	-26	24.09	70.7	-1.3	-15	-323	1469.9
9-Jun	7.62	131.8	-8.9	-103	9.48	526.4	-26.7	-309	-309	12.26	334.8	-26.9	-311	-311	8.40	40.4	-5.4	-62	10.18	136.8	-13.4	-155	18.69	140.1	-7.6	-88	23.86	66.4	-4.2	-49	-1077	1376.8
10-Jun	7.32	118.5	-13.4	-155	9.06	486.4	-40.1	-464	-464	11.91	296.6	-38.2	-442	-442	8.11	32.4	-8.1	-93	9.76	117.1	-19.7	-228	18.38	128.7	-11.4	-132	23.51	60.1	-6.4	-74	-1588	1239.7
11-Jun	6.80	95.7	-22.7	-263	8.33	416.7	-69.7	-807	-807	11.32	241.0	-55.6	-643	-643	7.59	20.8	-11.5	-134	9.03	83.4	-33.7	-390	17.83	109.4	-19.3	-223	22.92	49.2	-10.8	-126	-2585	1016.3
12-Jun	6.64	88.9	-6.8	-79	8.10	395.2	-21.5	-248	-248	11.13	223.9	-17.1	-198	-198	7.43	17.5	-3.4	-39	8.80	73.7	-9.7	-112	17.67	103.9	-5.6	-64	22.73	46.2	-3.0	-35	-775	949.3
13-Jun	6.48	82.1	-6.8	-79	7.88	374.5	-20.7	-240	-240	10.95	207.8	-16.2	-187	-187	7.28	14.1	-3.4	-39	8.58	64.1	-9.6	-111	17.50	98.3	-5.6	-64	22.55	43.2	-3.0	-35	-755	884.1
14-Jun	6.32	75.3	-6.8	-79	7.65	354.4	-20.1	-233	-233	10.77	194.1	-13.7	-158	-158	7.12	10.8	-3.4	-39	8.35	54.5	-9.6	-111	17.33	92.7	-5.6	-64	22.37	40.2	-3.0	-35	-719	822.0
15-Jun	6.16	68.5	-6.8	-79	7.43	334.3	-20.1	-233	-233	10.58	180.4	-13.7	-158	-158	6.96	8.0	-2.8	-32	8.13	44.9	-9.6	-111	17.17	87.2	-5.6	-64	22.18	37.1	-3.0	-35	-712	760.5
16-Jun	6.00	61.8	-6.8	-79	7.20	314.2	-20.1	-233	-233	10.40	166.7	-13.7	-158	-158	6.80	6.7	-1.2	-14	7.90	36.6	-8.3	-96	17.00	81.6	-5.6	-64	22.00	34.1	-3.0	-35	-679	701.8

Table 5: 1948 Flood Progression

Date	Description
27-May	Agassiz dike failed, Fraser flowed into Harrison Lake. Dewdney dike failed.
28-May	CPR washed out at Agassiz. Dikes at Fort Langley failed. Sandbagging started at Lulu Island.
29-May	Barnston Island almost inundated.
30-May	Nicomen Island dikes failed.
31-May	Matsqui and Rosedale dikes failed. Pitt Meadows and Harrison evacuated.
1-Jun	Dike failed at Lulu Island but was repaired. Dike failed at Burnaby.
2-Jun	Dike failed at Cannor, portion of Sumas Prarie (Chilliwack) flooded north and east of Vedder Canal.
3-Jun	Hatzic dike failed, 4.5m wall of water burst through dike, ripping out 90m of CPR line.
4-Jun	Barnston dike failed. Dewdney flooded.
7-Jun	Mission Bridge south span failed
11-Jun	Residents of Sumas Prairie south evacuated.
13-Jun	Water levels started to drop.

Table 6: Estimated Mean Daily Storage Volumes and Flows for 1948

Date	Matsqui WL (m)	Matsqui Storage Volume 10^6 (m^3)	Matsqui Change in Storage 10^6 (m^3)	Matsqui Mean Daily Storage flow (m^3/s)	Matsqui Adjusted Storage flow (m^3/s)	Chilliwack WL (m)	Chilliwack Storage Volume 10^6 (m^3)	Chilliwack Change in Storage 10^6 (m^3)	Chilliwack Mean Daily Storage flow (m^3/s)	Chilliwack Adjusted Storage flow (m^3/s)
26-May	6.245	72.2	72.2	835	10	8.00	44.3	44.3	513	10
27-May	6.634	88.7	16.5	191	10	8.49	61.5	17.3	200	10
28-May	6.946	101.9	13.3	153	10	8.89	75.4	13.9	161	10
29-May	7.202	113.2	11.3	131	10	9.21	83.4	7.9	92	10
30-May	7.355	120.0	6.8	79	10	9.40	87.0	3.6	42	10
31-May	**7.437**	123.7	3.7	42	542	**9.51**	88.9	2.0	23	10
1-Jun	7.487	125.9	2.2	26	426	9.57	90.1	1.2	14	10
2-Jun	7.429	123.3	-2.6	-30	270	9.50	88.8	-1.4	-16	484
3-Jun	7.342	119.5	-3.9	-45	94	9.39	86.7	-2.1	-24	371
4-Jun	7.426	123.2	3.7	43	43	9.50	88.7	2.0	23	98
5-Jun	7.470	125.2	2.0	23	23	9.55	89.7	1.1	12	12
6-Jun	7.420	122.9	-2.2	-26	-26	9.49	88.5	-1.2	-14	-14
7-Jun	7.422	123.0	0.1	1	1	9.49	88.6	0.0	0	0
8-Jun	7.457	124.5	1.6	18	18	9.53	89.4	0.8	10	10
9-Jun	7.524	127.5	3.0	35	35	9.62	91.0	1.6	18	18
10-Jun	**7.576**	129.8	2.3	27	27	**9.80**	94.4	3.4	40	40
11-Jun	7.570	129.6	-0.3	-3	-3	9.79	94.3	-0.1	-2	-2
12-Jun	7.539	128.2	-1.4	-16	-16	9.75	93.6	-0.7	-8	-8
13-Jun	7.451	124.3	-3.9	-46	-46	9.64	91.5	-2.1	-24	-24
14-Jun	7.307	117.9	-6.4	-74	-74	9.46	88.0	-3.4	-39	-39
15-Jun	7.170	111.8	-6.1	-71	-71	9.29	84.8	-3.3	-38	-38
16-Jun	7.039	106.0	-5.8	-68	-68	9.12	81.7	-3.1	-36	-36
17-Jun	6.969	102.9	-3.1	-35	-35	9.03	80.0	-1.7	-19	-19
18-Jun	6.907	100.2	-2.7	-31	-31	8.95	77.7	-2.3	-26	-26
19-Jun	6.888	99.5	-0.8	-9	-9	8.93	76.9	-0.8	-9	-9
20-Jun	6.832	97.1	-2.4	-28	-28	8.86	74.4	-2.5	-29	-29
21-Jun	6.768	94.4	-2.7	-31	-31	8.78	71.6	-2.8	-33	-33
22-Jun	6.680	90.6	-3.8	-43	-43	8.67	67.6	-3.9	-45	-45
23-Jun	6.550	85.1	-5.5	-64	-64	8.50	61.9	-5.8	-67	-67
24-Jun	6.413	79.3	-5.8	-67	-67	8.33	55.8	-6.1	-71	-71
25-Jun	6.280	73.7	-5.6	-65	-65	8.16	49.9	-5.9	-68	-68
26-Jun	6.149	68.1	-5.6	-64	-64	7.9	42.4	-7.5	-87	-87

Date	Hatzic WL (m)	Hatzic Storage Volume 10^6 (m^3)	Hatzic Change in Storage 10^6 (m^3)	Hatzic Mean Daily Storage flow (m^3/s)	Hatzic Adjusted Storage flow (m^3/s)	Nicomen WL (m)	Nicomen Storage Volume 10^6 (m^3)	Nicomen Change in Storage 10^6 (m^3)	Nicomen Mean Daily Storage flow (m^3/s)	Nicomen Adjusted Storage flow (m^3/s)
26-May	7.00	8.3	8.3	96	10	8.10	43.9	43.9	508	10
27-May	7.42	17.2	8.9	103	10	8.64	66.9	23.1	267	10
28-May	7.75	24.3	7.1	82	10	9.08	85.8	18.8	218	10
29-May	8.03	30.3	6.0	70	10	9.43	102.2	16.4	190	10
30-May	8.19	34.7	4.4	51	10	9.64	112.0	9.8	113	613
31-May	**8.28**	37.1	2.4	28	10	**9.76**	117.3	5.3	61	461
1-Jun	8.34	38.6	1.5	17	10	9.83	120.5	3.2	37	280
2-Jun	8.27	36.9	-1.7	-20	10	9.75	116.8	-3.7	-43	-43
3-Jun	8.18	34.4	-2.5	-29	318	9.63	111.2	-5.6	-65	-65
4-Jun	8.27	36.8	2.4	28	28	9.74	116.6	5.4	62	62
5-Jun	8.32	38.1	1.3	15	15	9.81	119.4	2.8	33	33
6-Jun	8.26	36.7	-1.5	-17	-17	9.74	116.2	-3.2	-37	-37
7-Jun	8.27	36.7	0.0	1	1	9.74	116.3	0.1	1	1
8-Jun	8.30	37.7	1.0	12	12	9.79	118.6	2.3	26	26
9-Jun	8.38	39.7	2.0	23	23	9.88	122.9	4.3	50	50
10-Jun	**8.50**	43.1	3.4	39	5	**10.10**	133.1	10.3	119	119
11-Jun	8.49	42.9	-0.2	-2	-2	10.09	132.7	-0.4	-5	-5
12-Jun	8.46	42.0	-0.9	-10	-8	10.05	130.7	-2.0	-23	-23
13-Jun	8.37	39.4	-2.6	-30	-23	9.93	125.0	-5.7	-66	-66
14-Jun	8.21	35.2	-4.2	-48	-226	9.73	115.8	-9.2	-106	-106
15-Jun	8.06	31.2	-4.0	-46	-13	9.54	107.0	-8.8	-102	-102
16-Jun	7.92	27.9	-3.4	-39	131	9.35	98.5	-8.4	-97	-97
17-Jun	7.85	26.3	-1.6	-19	-19	9.26	94.0	-4.5	-52	-52
18-Jun	7.78	24.8	-1.4	-16	-16	9.17	90.0	-4.0	-46	-46
19-Jun	7.76	24.4	-0.4	-5	-5	9.14	88.9	-1.2	-14	-14
20-Jun	7.70	23.1	-1.3	-15	-15	9.06	85.2	-3.6	-42	-42
21-Jun	7.63	21.7	-1.5	-17	-17	8.98	81.2	-4.0	-47	-192
22-Jun	7.54	19.7	-2.0	-23	-23	8.85	76.0	-5.3	-61	-42
23-Jun	7.40	16.7	-3.0	-34	-34	8.67	68.3	-7.7	-89	-62
24-Jun	7.25	13.6	-3.1	-36	-36	8.48	60.1	-8.1	-94	-66
25-Jun	7.11	10.5	-3.0	-35	-35	8.30	52.2	-7.9	-91	-64
26-Jun	6.90	7.5	-3.0	-35	-35	8.00	39.6	-12.6	-146	-102

Date	Kent WL (m)	Kent Storage Volume 10^6 (m^3)	Kent Change in Storage 10^6 (m^3)	Kent Mean Daily Storage flow (m^3/s)	Kent Adjusted Storage flow (m^3/s)	Total Mean Daily Storage flow (m^3/s)	Total Storage Volume 10^6 (m^3)
26-May	17.30	91.6	91.6	1061	10	50	260.2
27-May	17.64	103.1	11.5	133	733	773	337.4
28-May	17.92	112.3	9.2	107	558	598	399.7
29-May	18.15	120.4	8.0	93	93	133	449.5
30-May	18.28	125.3	4.9	57	57	700	479.1
31-May	**18.36**	128.0	2.7	31	31	**1054**	**495.0**
1-Jun	18.40	129.6	1.6	19	19	745	504.8
2-Jun	18.35	127.7	-1.9	-22	-22	699	493.6
3-Jun	18.27	124.9	-2.8	-33	-33	686	476.7
4-Jun	18.35	127.7	2.7	31	31	263	493.0
5-Jun	18.39	129.1	1.4	17	17	99	501.6
6-Jun	18.34	127.5	-1.6	-19	-19	-113	491.8
7-Jun	18.34	127.5	0.0	1	1	3	492.1
8-Jun	18.37	128.6	1.1	13	13	79	498.9
9-Jun	18.43	130.8	2.2	25	25	151	511.9
10-Jun	**18.40**	129.6	-1.2	-14	-14	**176**	**530.1**
11-Jun	18.39	129.4	-0.2	-2	-2	-13	528.9
12-Jun	18.37	128.4	-1.0	-11	-11	-67	522.9
13-Jun	18.29	125.6	-2.9	-33	-33	-193	505.7
14-Jun	18.16	120.9	-4.6	-54	-54	-500	477.8
15-Jun	18.04	116.5	-4.4	-51	-51	-275	451.2
16-Jun	17.92	112.5	-4.0	-46	-46	-117	426.5
17-Jun	17.86	110.4	-2.1	-24	-24	-149	413.6
18-Jun	17.81	108.5	-1.8	-21	-21	-141	401.4
19-Jun	17.79	108.0	-0.5	-6	-6	-43	397.7
20-Jun	17.74	106.3	-1.7	-19	-19	-133	386.2
21-Jun	17.68	104.4	-1.9	-22	-22	-296	373.3
22-Jun	17.61	101.8	-2.6	-30	-30	-185	355.7
23-Jun	17.49	98.0	-3.8	-44	-44	-271	330.0
24-Jun	17.37	94.0	-4.1	-47	-47	-287	302.7
25-Jun	17.25	90.0	-3.9	-45	-45	-277	276.4
26-Jun	17.2	88.3	-1.7	-20	-20	-309	245.9

APPENDIX D
REVIEW OF MISSION RATING CURVE
WATER SURVEY OF CANADA

Analysis of the Rating Curve for Fraser River at Mission 08MH024

Introduction

The prediction made by the Lower Fraser River Hydraulic Model of water levels at the Mission gauge for the design discharge of 18900 m^3/s differed from the estimate provided by Water Survey of Canada's rating curve. A review of the curve has been undertaken resulting in a new curve which better represents the observations and improves the extrapolation of the curve to flood levels.

History

Discharge measurements commenced at Mission in 1964. Between 1964 and 1966 thirty eight discharge measurements were collected. A rating curve, Curve 2, was constructed using a graphical technique. The maximum measured discharge was 13394 m^3/s on June 14 1964 which was confirmed by two other measurements above 13000 m^3/s within the same period. The rating curve was extended to 18900 m^3/s, an extension of approximately 140%. Curves are commonly extended to this degree as high flows, which define the upper portion of the rating curve, are infrequent and transitory so the maximum measured discharge during any given period may be considerably smaller than the maximum discharge experienced during that same period.

Due to degradation of the bed, measurements conducted during the eighties plotted to the right of Curve 2, i.e. for a given water level greater discharges were observed than predicted by the rating curve. In 1990 a new curve, Curve 3, was drawn based on measurements conducted between 1984 and 1989. The maximum measured discharge during this period was 12100 m^3/s. As there was no evidence to revise the upper portion of the rating relation Curve 3 was drawn coincident with Curve 2 above a stage of 7.5m.

Rating Curve Analysis

Since Curve 2 was drawn a number of measurements have been collected including the largest measured discharge on record at 13650 m^3/s. For this reason it is appropriate to revisit the curve and review how well the new observations fit the curve. Thus the first phase of the analysis is to evaluate the curves on record using all the available data and the second phase is to review the extrapolation of the curve.

Ideally a rating curve is based on a homogeneous population of observations. The available data span a period of 40 years and, as evidenced by the shift observed in the eighties, are not homogeneous. It would be conservative to only use measurements conducted after 1984 but the lack of high flow measurements in this period would undermine extrapolation of the rating curve to flood levels. Consequently the population of measurements used to derive the rating curve consists of two sub-populations; all measurements collected since 1984 and measurements collected prior to 1984 with gauge heights exceeding 5.6m. The 5.6m threshold represents the highest measured flow during the period 1984 to 1989.

less than 0.5 is caused by fitting a model to a data set with some variability, particularly at 5.6m stage.

The stage-velocity relation was derived using observations above a stage of 5.0m. Observations below this threshold were excluded as low flow discharge at Mission is tidally influenced and the trend suggested a segmented relation where the upper portion is relevant for extrapolation. There is an element of judgment in the selection of the appropriate population which influences the resulting estimates.

Table 1. Comparison of discharge estimates

Stage (m)	Log extension Q (m^3/s)	Area-velocity Q (m^3/s)	% Discrepancy
7.5	14813	14368	3.1
8	16251	15601	4.1
8.5	17730	16855	5.2
9	19247	18130	6.2

Table 1 illustrates the values obtained by the two estimation techniques. The discrepancy between discharge estimates by the two techniques is small but increases as a function of flow.

Uncertainty

A confidence interval cannot be computed for the velocity-area estimates nor the log extension estimates as they are based on extrapolation. The validity of the log extension is assessed by comparison with the trend of the observations and the known geometry of the cross section. The stage-area estimates can be assessed through comparison with other estimates of channel velocity and cross-sectional area.

The estimates of discharge by the-velocity area method can be compared with analyses performed by NHC during derivation of the Lower Fraser River Hydraulic Model. Figure 4 illustrates two stage area models; one derived from WSC measurements and one from bathymetric and Lidar surveys conducted at the WSC measurement cross-section in support of the model. While there is some discrepancy at low stage at high stage there is close agreement between the two techniques. Above the range of the observations from measurements the estimates of area by the two techniques are within 1% of each other. The discrepancy at low stage may be due to errors resulting from merging lidar and bathymetric data sets. The agreement of the two techniques to suggests reasonable predictions of channel cross sectional are can be made.

The stage-velocity relation exhibits considerable variance. Figures 5 and 6 illustrate the normalized residuals of the stage-velocity model and the distribution of the residuals over time, respectively. While it seems that the model is an unbiased fit to the observations there is clearly temporal variability contributing to the variance.

Predictions of velocity by the stage velocity relation can be compared with an up-scaling analysis performed by NHC. This analysis extrapolated based on high flows in 1967 and stage discharge table for Curve 4. This curve will be considered preliminary and subject to alteration until published by WSC.

The extrapolation of the curve was based on both the velocity-area and log extension estimates as this was considered the most conservative approach. Curve 4 effectively splits the difference between the velocity-area and log extension estimates. A comparison of a curve derived purely from the observations with no estimates with Curve 4 was made; over the range of the observations the curves are essentially co-incident and start to diverge above 7.5m. At 9.0 the discrepancy is only 1.3%. This suggests that a reasonable extrapolation has been made.

A comparison of Curves 3 and 4 shows that they are coincident from a stage of 3m up to 5.2m. This supports the validity of the shift to Curve 3 in 1990 as different techniques were used to derive the two curves. The upper portion of Curves 3 and 4 are significantly different. Curve 3 is identical to Curve 2 at high stage. The observations collected since 1968, when Curve 2 upon was drawn, indicate that the rate of change of discharge with increasing stage remains relatively constant. For the rate of change of discharge with stage to increase, i.e. for the curve to flatten out, an increase would have to occur in the rates of change of either velocity or area with stage. The cross section is virtually rectangular and confined by dykes to defined flow levels. Therefore any increase in the rate of change of discharge would likely be due to increases in the rate of change of velocity with increasing stage. While there is variability in the stage velocity relation there is little to support a sudden increase within the range of observations. Observations at high flow are required to determine the nature of the stage–velocity relation.

Model Comparison

The Lower Fraser River Hydraulic Model predicts a water level of 8.9m for the design discharge of 18900 m^3/s in the GSC datum. Using the WSC datum this is equivalent to a water level of 8.857m for which Curve 4 estimates a discharge of 18314 m^3/s. It is important to recall the nature of the estimates before comparing them; the Lower Fraser River Hydraulic Model predicts water level for a given discharge whereas the WSC rating curve predicts discharge for a given water level. If the rating curve is to be used to assess the hydraulic model the estimated discharges are within 3% of each other. Given the uncertainties involved it is unlikely that these two estimates are significantly different from one another.

Both the Lower Fraser River Hydraulic Model and WSC Curve 4 provide estimates of flow conditions beyond the range of observations. While the estimates may appear reasonable based on the information available, verification of the estimates will only be possible when discharge measurements for flows of the same magnitude as the estimates are available.

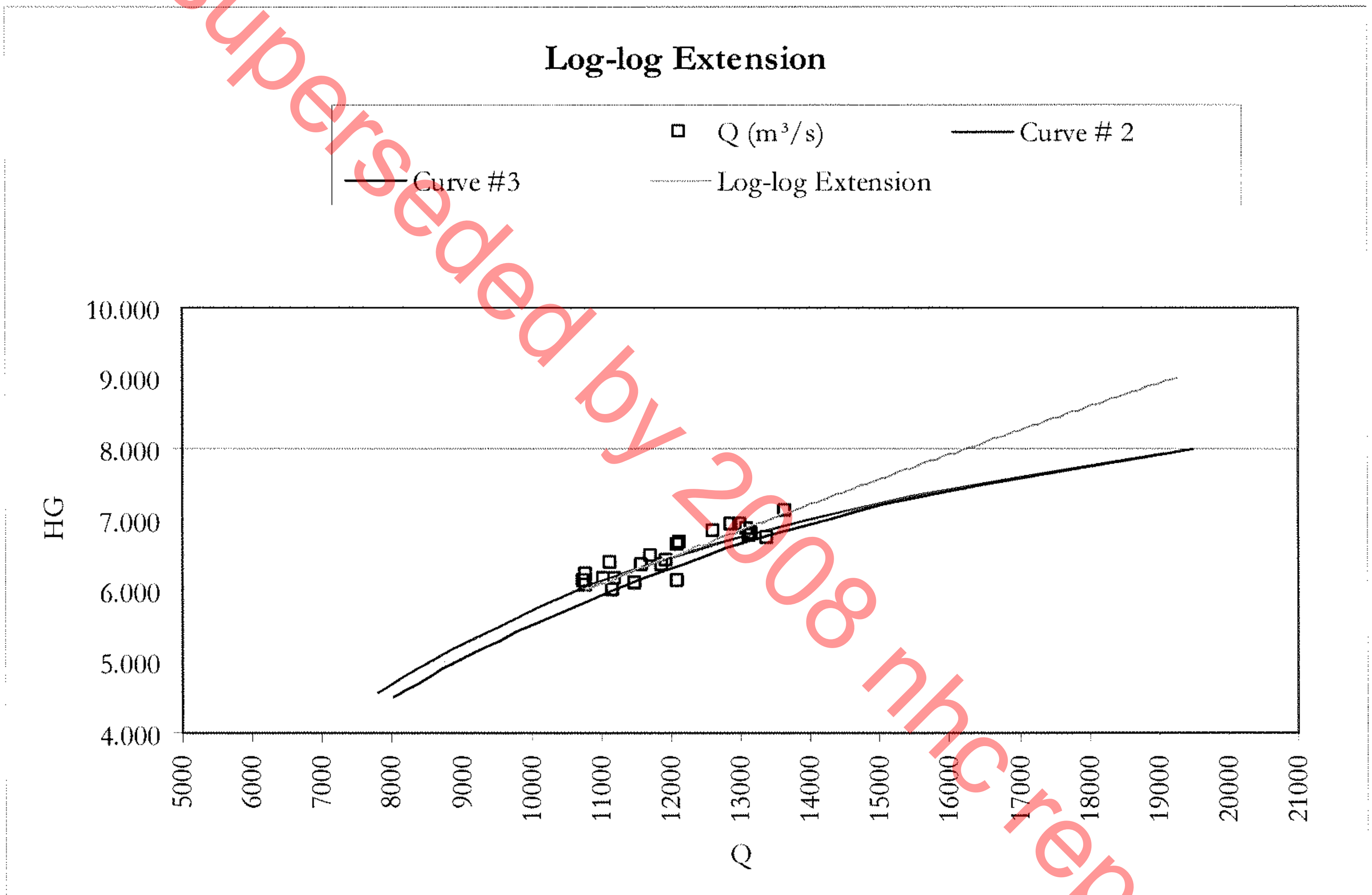


Figure1. Log-log extension based on discharge measurements taken at stages greater than 6.0m

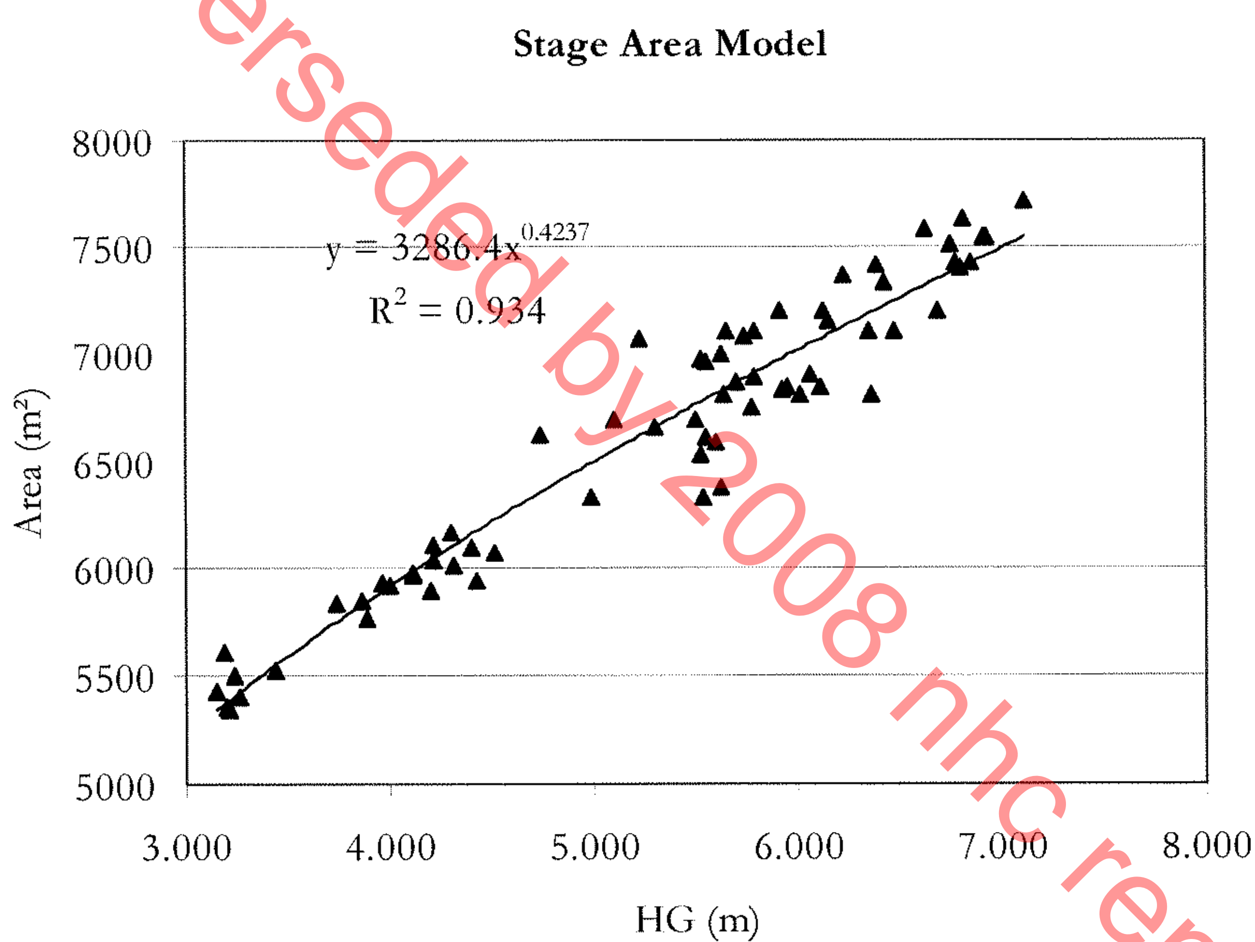


Figure 2. Stage area model

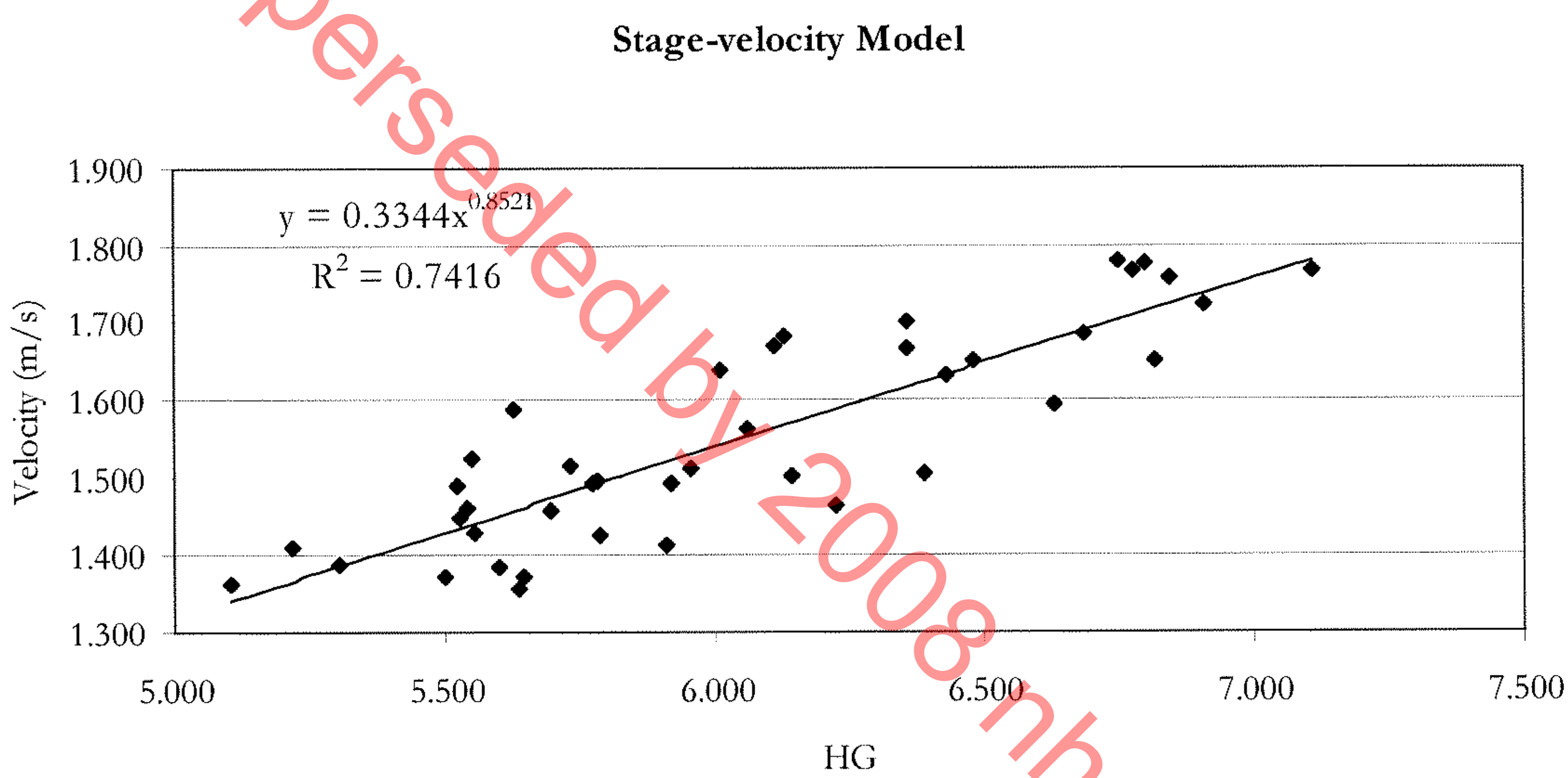


Figure 3. Stage velocity model

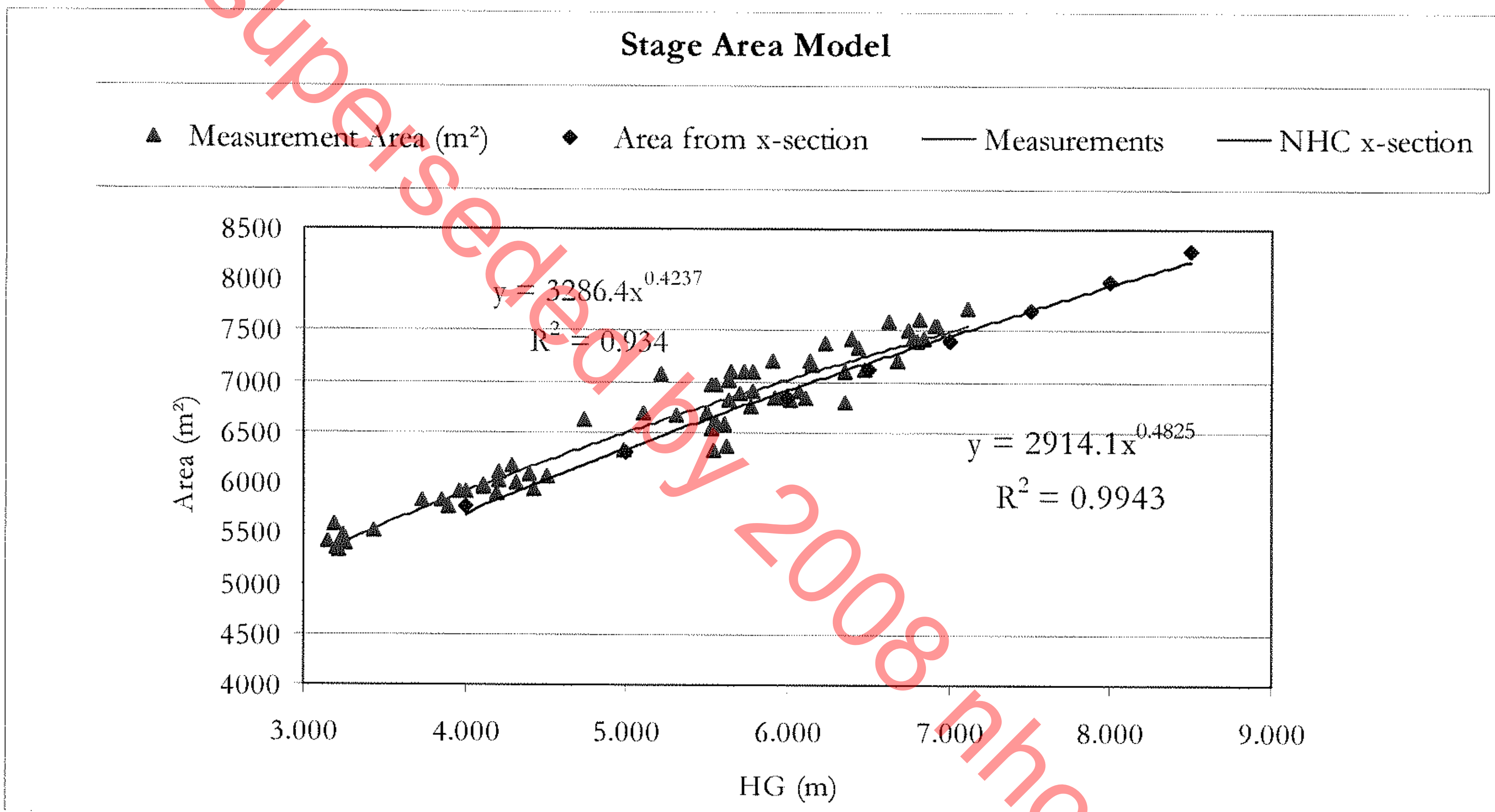


Figure 4. Comparison of stage area models

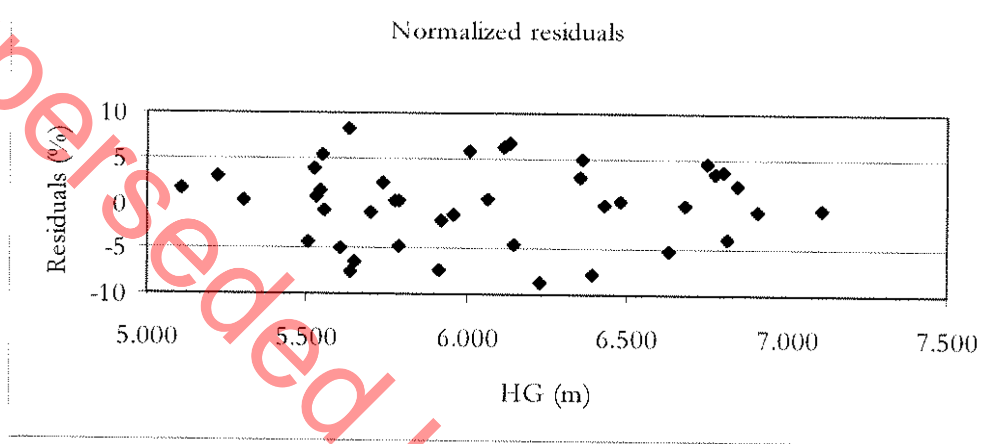


Figure 5. Normalized residuals of the stage velocity model

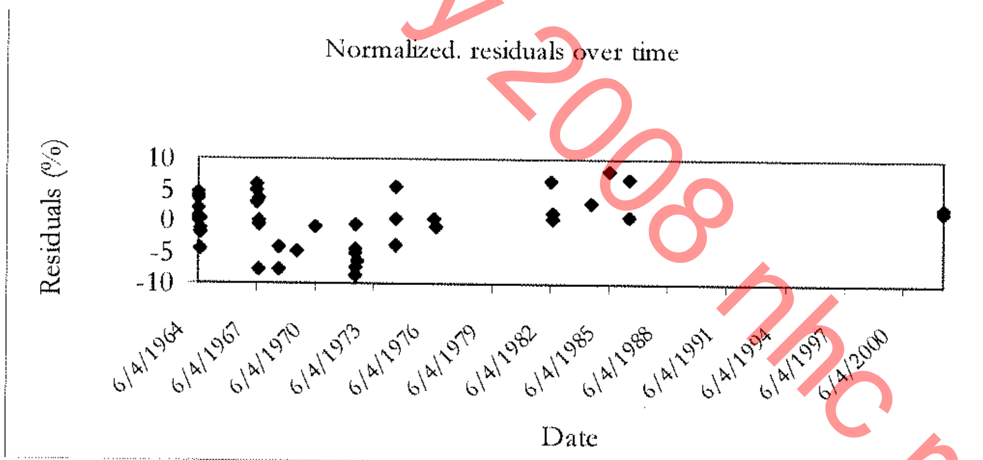


Figure 6. Normalized residuals of the stage-velocity model over time

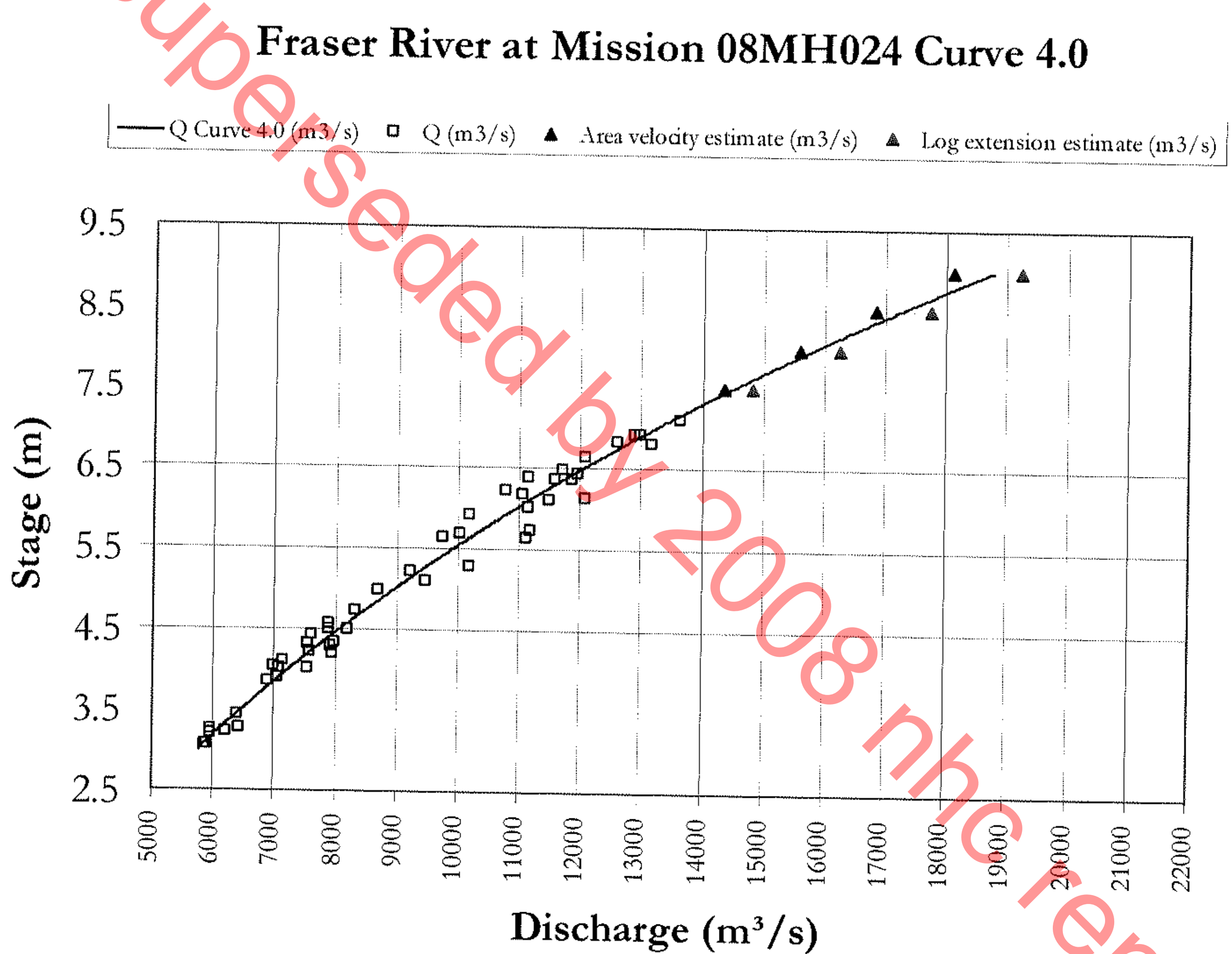


Figure 7. WSC Rating curve 4.0 and discharge measurements.

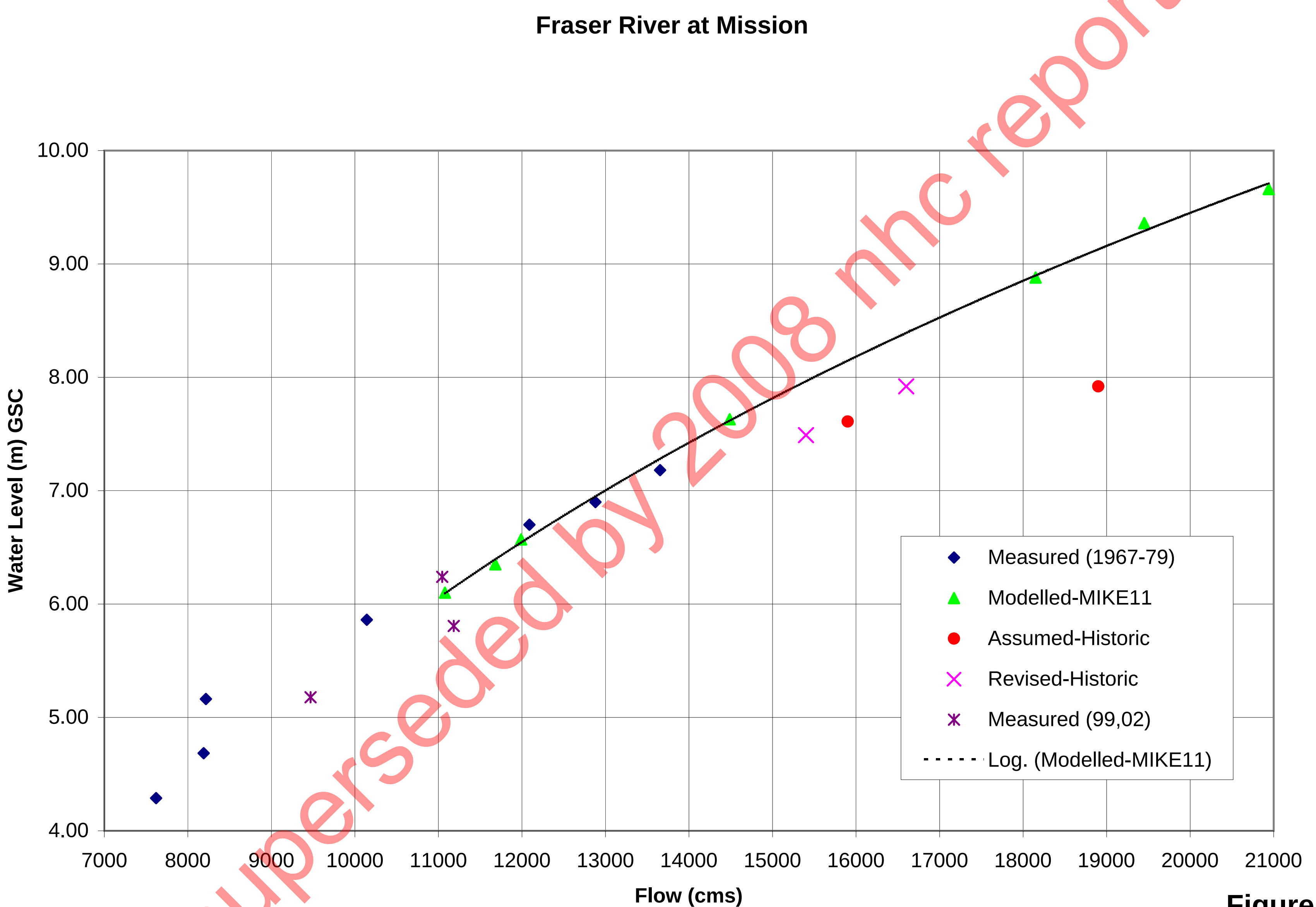

Fraser River at Mission
Water Level (m) GSC
10.00
9.00
8.00
7.00
6.00
5.00
4.00
7000
8000
9000
10000
11000
12000
13000
14000
15000
16000
17000
18000
19000
20000
21000
Flow (cms)
Measured (1967-79)
Modelled-MIKE11
Assumed-Historic
Revised-Historic
Measured (99,02)
Log. (Modelled-MIKE11)

Figure 2

Air Photography Showing
Extent of 1948 Flood Downstream of Mission

1948 Flood – Silverdale Creek to Mission
Source: BC Gov't Special Project

1948 Flood – McMillan Island to Silverdale Creek
Source: BC Gov't Special Project

1948 Flood – Barnston Island to McMillan Island
Source: BC Gov't Special Project

1948 Flood – Port Mann to Barnston Island
Source: BC Gov't Special Project

APPENDIX E
DESIGN WINTER OCEAN LEVELS

Ocean Water Levels and Downstream Boundary Conditions

2006 October 31

Prepared For:

Fraser Basin Council

British Columbia

Prepared By:

Michael Tarbotton, P.Eng. and Max Larson, P.Eng.

Triton Consultants Ltd.

Vancouver, BC

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TABLE OF CONTENTS

1. INTRODUCTION 1

1.1 Background 3

1.1.1 Vertical Datum 3

1.1.2 Previous Studies 7

1.1.3 Long Term Sea-Level Rise 9

1.1.4 Existing Sea Dyke Elevations 10

2. METHOD 11

2.1 Existing Water Level Measurements 11

2.2 Tidal Constituent Database 12

2.3 Historical Tide Height Predictions 13

2.4 Historical Storm Surge Events 13

2.5 Tide Height Statistics by Month 14

2.6 Storm Surge Statistics by Month 14

2.7 Combining Tide and Storm Surge 15

2.7.1 Maximum Elevation 17

2.7.2 Time Series 17

3. RESULTS BASED ON HISTORICAL DATA 18

3.1 Total Water Level Measurements 18

3.2 Tidal Constituent Database 18

3.3 Tide Height Statistics by Month 21

3.4 Storm Surge Statistics 26

3.5 Combining Tide and Storm Surge 34

3.5.1 Maximum Elevation 34

3.6 Time Series 39

4. SUMMARY AND RECOMMENDATIONS 41

4.1 Future Analysis Requirements and Refinements 41

5. REFERENCES 42

LIST OF FIGURES

Figure 1: Model Boundary Points 1
Figure 2: Relevant CHS Benchmarks 6
Figure 3: Variation of Amplitude and Phase – Semi-Diurnal Effect of the Moon (M2) 13
Figure 4: Storm Surge Calculation Definition Sketch 14
Figure 5: Storm Surge Definition Sketch 15
Figure 6: Moving Window Definition Sketch 16
Figure 7: Tidal Elevation Histogram – January, February and March 22
Figure 8: Tidal Elevation Histogram – April, May and June 23
Figure 9: Tidal Elevation Histogram – July, August and September 24
Figure 10: Tidal Elevation Histogram – October, November and December 25
Figure 11: Point Atkinson Tidal Elevation Histogram – WL ≥ 1.5 m CGD 26
Figure 12: Surge Duration Distribution - January, February and March 30
Figure 13: Surge Duration Distribution – April, May and June 31
Figure 14: Surge Duration Distribution - July, August and September 32
Figure 15: Surge Duration Distribution - October, November and December 33
Figure 16: Combined Water Level Exceedance Graph - January, February and March 35
Figure 17: Combined Water Level Exceedance Graph – April, May and June 36
Figure 18: Combined Water Level Exceedance Graph – July, August and September 37
Figure 19: Combined Water Level Exceedance Graph – October, November and December ...38
Figure 20: Winter, 200-Year Return Period Design Event 40

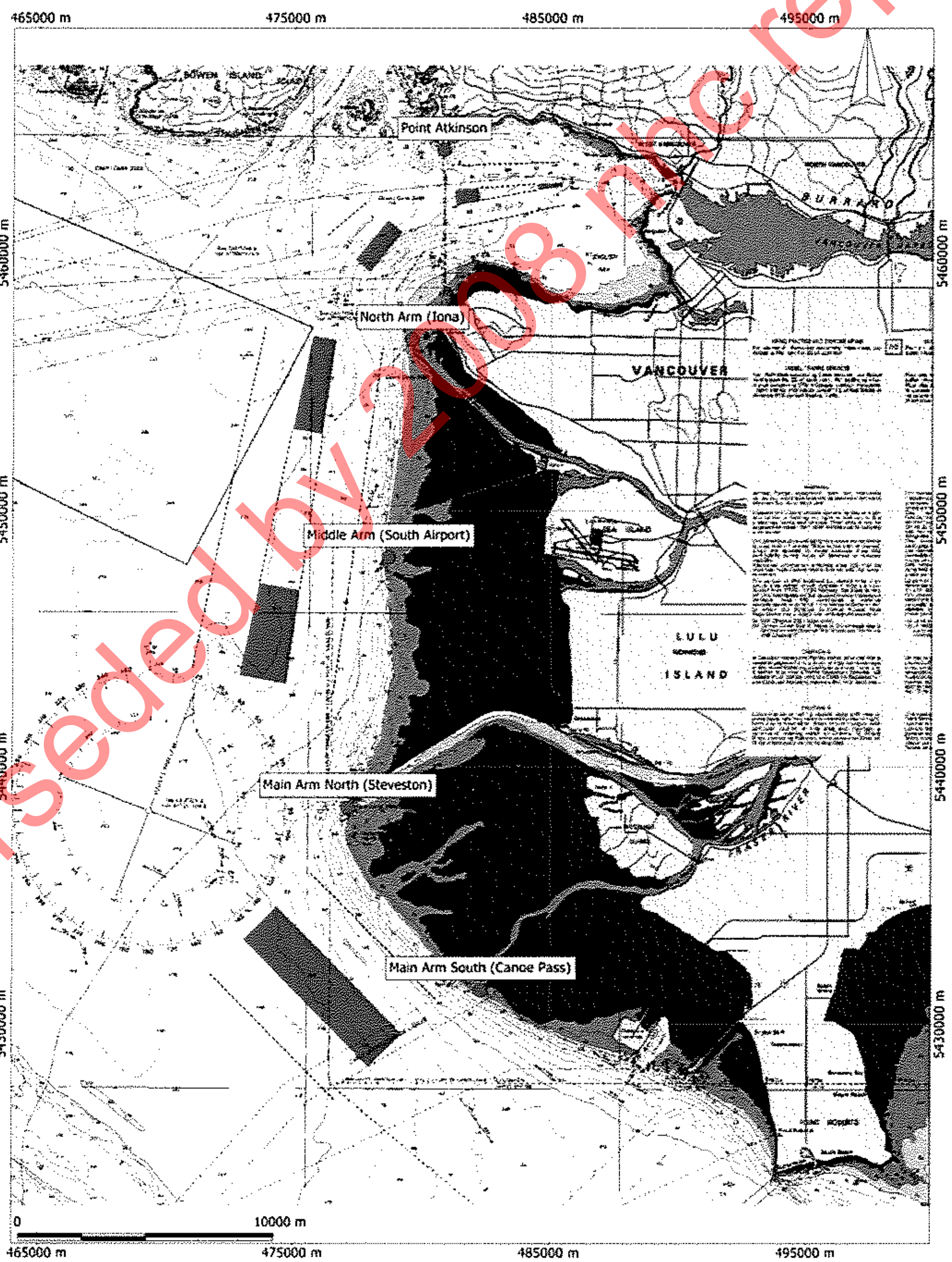
LIST OF TABLES

Table 1: Boundary Point Coordinates 2
Table 2: Relevant CHS Benchmarks 4
Table 3: Summary of Estimates of Extreme Flood Levels (Liu, 2002) 7
Table 4: Annual Maximum Water Levels at Point Atkinson (Lui, 2002) 8
Table 5: Storm Surge Predictive Equations – Point Atkinson (Neil, 2001) 9
Table 6: Sea-Level Rise on the Fraser Delta 10
Table 7: Existing Dyke or Design Elevations 10
Table 8: Historical Water Level Stations 12
Table 9: Point Atkinson Statistics based on Total Water Level (Mean Estimate) 18
Table 10: Tidal Height Constituents - Amplitudes (m) 19
Table 11: Tidal Height Constituents - Phase (deg UTC) 20
Table 12: Chronological Listing of Historical Storm Surge Events 26
Table 13: Historical Point Atkinson Surge Statistics 29
Table 14: Point Atkinson Statistics based on EST Analysis (Mean Estimate) 39
Table 15: Point Atkinson Statistics based on EST Analysis (95%CI) 39

1. INTRODUCTION

This report describes the analyses undertaken by Triton Consultants Ltd. (Triton) to support Northwest Hydraulic Consultants Ltd. (NHC) in developing a calibrated one-dimensional model of the Fraser River (the Model) for the Fraser Basin Council (FBC). Triton's role in the study was to provide appropriate downstream boundary conditions at the mouths of the Fraser River. The four boundary locations that define the seaward ends of each conveyance channel (branch) in the Model are shown in Figure 1.

Figure 1: Model Boundary Points



The WGS84 geographic and UTM Zone 10 grid coordinates of each of these points are shown in Table 1.

Table 1: Boundary Point Coordinates

Point Description	Latitude (°N)	Longitude (°W)	Northing (m)	Easting (m)
North Arm (Iona)	49.2624	123.2807	479580	5456668
Middle Arm (Airport South)	49.1860	123.2926	478680	5448176
Main Arm North (Steveston)	49.0983	123.3134	477122	5438436
Main Arm South (Canoe Pass)	49.0346	123.2421	482303	5431335

In developing downstream boundary conditions for the model, the effects of tide (deterministic component) and non-tidal mechanisms (probabilistic component) were considered. The non-tidal mechanisms explicitly or implicitly include: storm surge (barometric and wind-induced), seasonal fluctuations (e.g., freshwater discharge, seasonal weather features) and other long-term variations in mean water level (e.g., El Niño, La Niña, global climate change). Non-tidal effects are secondary to tides but are of considerable importance. Storm surge in the Strait of Georgia is in the order of one metre at the return periods of interest to this study. Wave-induced effects such as setup and the super-elevation of water levels by littoral currents are considered to be secondary and were not evaluated. Furthermore, these effects are especially small in the deep water in which the downstream boundaries of the Model are located.

There are many ways of approaching the combined water level issue, each with its own strengths and weaknesses. For example, one approach would be to develop a time-stepping two-dimensional numerical hydrodynamic model of tides and storm surge in the Strait of Georgia, calibrate the model against historical data, then use the model to simulate many years of data such that the extreme design event might be selected directly from the simulated record. This has the advantage of explicitly accounting for the hydrodynamics of the problem that might vary significantly should a storm surge event occur at high rather than low tide. It also has the ability to take the effect of global climate change explicitly into account provided suitable atmospheric predictions can be obtained and used to drive the simulation. It has, however, the major limitation of being reliant upon questionable atmospheric predictions and is so computationally intensive that it may be economically impractical.

A modification of the above approach would be to develop a two-dimensional numerical hydrodynamic model of storm surge in the Strait of Georgia and to investigate the response of the model to forcing parameters such as atmospheric pressure and two-dimensional wind fields. Following calibration to a number of documented surge events, model results would be used to generate empirical relationships between storm surge, atmospheric pressure and wind speed and direction. These empirical relationships could then be used to predict future water levels. This method is computationally practical and was used successfully by the present authors in storm surge studies of the Chukchi (Alaska) and Beaufort Seas.

A third approach would be to perform purely statistical analyses on historical measurements of total water level without distinguishing between tide and surge components. A statistical distribution is fit to the data, which is then used to estimate return periods of extreme events. Its advantage is simplicity and robustness even though it is predicated on the theoretically-unsound practice of creating statistics from a parameter that includes both deterministic (tide) and probabilistic (surge) components. This method cannot account for global climate change in a non-arbitrary way.

A fourth approach is to undertake a tidal analysis of historical total water level observations so that the deterministic tidal component can be inferred and removed from the record leaving only the storm surge component. Statistical analyses can then be performed on this purely probabilistic component to estimate return periods of extreme storm surge events. Design total water level events are then obtained by artificially selecting a tide level to coexist with the design storm surge and recombining to yield total water level. As above, this method cannot account for global climate change in a non-arbitrary way.

The third and fourth approaches were followed in the present study. The primary condition of interest for this study was prescribed by the Client to be the 200-year event; such an event has a 10% probability of being exceeded in a 20-year period. We note that much longer dyke design return periods (e.g., 1,000 to 10,000 years) are common in other jurisdictions.

1.1 BACKGROUND

A brief review of previous storm surge studies of Georgia Basin was made including a review of recent investigations funded by the Canadian Climate Action Fund (CCAF) and the University of British Columbia. As well, the Pacific Storm Surge Forecasting and Modelling Workshop sponsored by the BC Ministry of Environment (Delta Vancouver Airport Hotel on November 22, 2005) was attended. Key findings from these studies and other background information are presented in the following sections.

1.1.1 Vertical Datum

There are two commonly used vertical datums of relevance to this study:

- Chart Datum (CD), and
- Canadian Geodetic Datum (CGD).

Chart Datum is used by the Canadian Hydrographic Service on CHS nautical charts to indicate an elevation that water levels are unlikely to fall below. It is set to coincide with Lower Low Water Large Tides which is the average of the lowest low waters, one from each of 19 years of tidal predictions.

Canadian Geodetic Datum (CGD) is the reference surface to which Geodesy Canada refers elevations. CGD is often called "sea-level datum" because it is an approximation to the mean sea level geoid. It is based on a 1928 adjustment of the Canadian levelling network in which the mean water levels at gauging stations in Halifax, Yarmouth, Point-au-Père, Vancouver and Prince Rupert were all fixed at zero. CGD is the vertical datum used by Northwest Hydraulic Consultants in the present study, hence all source data were converted to this reference level.

Table 2 show the difference between Chart Datum and Canadian Geodetic Datum at various locations in the study region. Values shown in bold are those selected by the author as being most indicative of the conversion at that location.

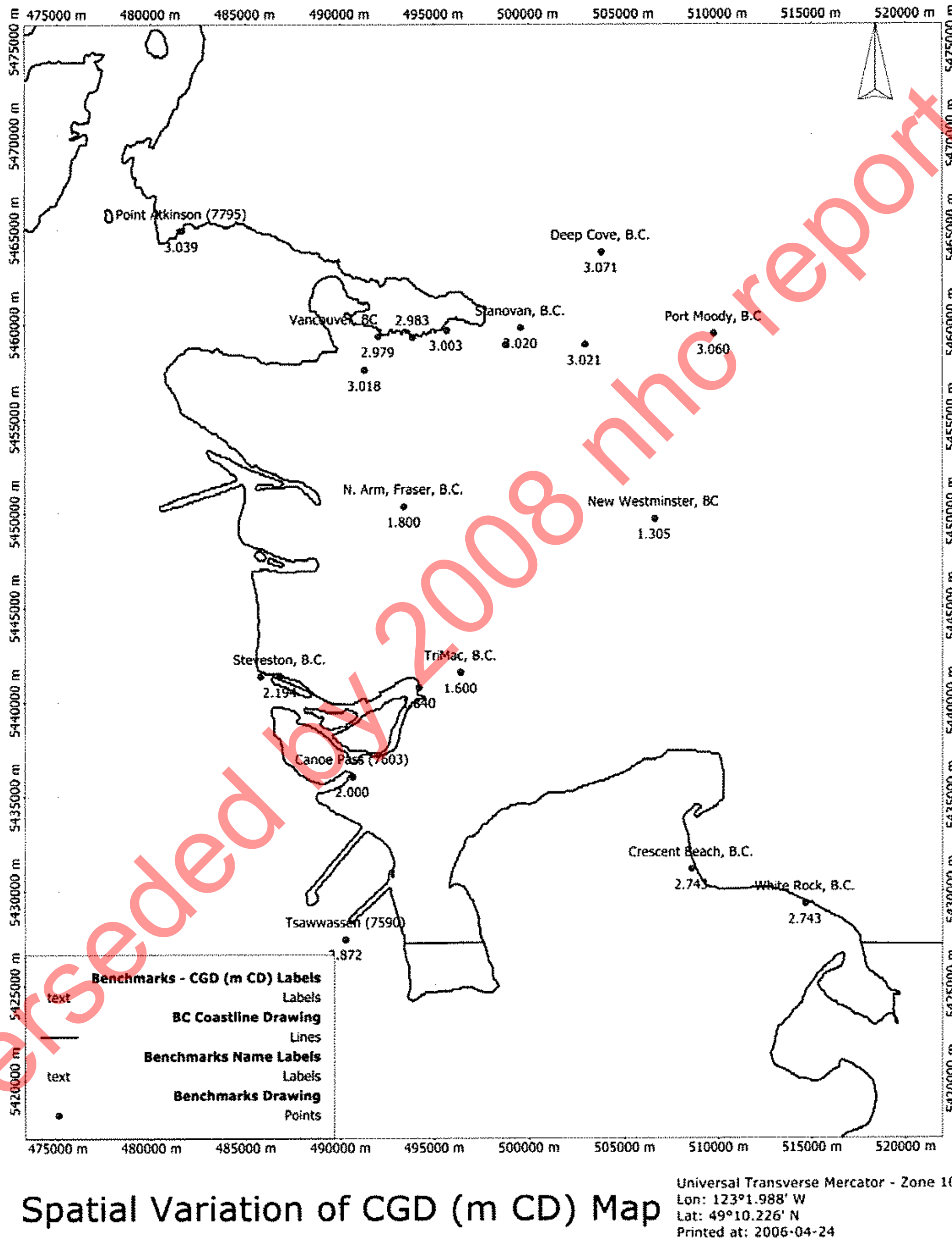
Table 2: Relevant CHS Benchmarks

Location	Lat.	Lon.	BM	CD	GSC	Δ	Reference
Point Atkinson (7795)	49.3370	-123.2530	118-1950	6.501	3.463	3.038	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7795
			128-1950	7.341	4.303	3.038	
			213-J-2	4.532	1.489	3.043	
			213-J-3	6.937	3.898	3.039	
			LS 286-56	7.404	4.367	3.037	
			Average			**3.039**	
Sand Heads (7594)	49.1250	-123.1950	NA	5.46	2.6	2.860	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7594
						3.000	"Effective Jan. 1/78 chart datum will be 3.00 m below Geodetic"
Tsawwassen (7590)	49.0000	-123.1333	77C010	6.478	3.606	**2.872**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7590
				6.498	3.606	2.892	"BM is unstable. 1978 leveling from BM 19-1959 elevation = 6.498m"
Canoe Pass (7603)	49.0775	-123.1285	77C034	4.843	2.843	**2.000**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7603
Woodward's Landing	49.1200	-123.0800	77-C-011	5.074	3.234	**1.840**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7610
Steveston	49.1250	-123.1810	77C008	3.29	1.09	2.200	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7607
			BM 1248J	3.623	1.435	2.188	
			Average			**2.194**	
N. Arm Fraser	49.2060	-123.0910	14-1959	3.686	1.886	**1.800**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7640
Crescent Beach	49.0333	-122.8833	16-J *	6.075	3.332	**2.743**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7579
False Creek	49.2710	-123.1200	BM1236-J1974	19.306	16.288	**3.018**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7710
Vancouver	49.2870	-123.1100	I-J	16.298	13.319	**2.979**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7735
North Vancouver	49.2865	-123.0846	BM2-1956	11.894	8.911	2.983	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7729
			Geod 210-J	19.129	16.146	2.983	
			Average			**2.983**	
Terminal Dock, Van.	49.2900	-123.0600	11-1954	7.754	4.751	**3.003**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7739

Location	Lat.	Lon.	BM	CD	GSC	Δ	Reference
New Westminster	49.2000	-122.9100	13J (59)	5.373	4.054	1.319	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7654
			6-1959	5.261	3.961	1.300	
			7-1959	5.272	3.972	1.300	
			77-C-072	4.69	3.39	1.300	
			Average			**1.305**	
Second Narrows	49.2830	-123.0170	1232-J	7.539	4.519	**3.020**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7744
Stanovan	49.2910	-123.0060	BC-6-10683	7.759	4.739	**3.020**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7747
Shellburn	49.2830	-122.9600	9-1954	7.202	4.181	**3.021**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7751
White Rock	49.0167	-122.8000	18-J	5.187	2.444	**2.743**	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7577
Deep Cove	49.3270	-122.9480	10-1964	5.295	2.226	3.069	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7765
			11-1964	5.941	2.863	3.078	
			43-1959	5.144	2.077	3.067	
			Average			**3.071**	
Port Moody	49.2880	-122.8660	24, 1961	7.737	4.677	3.060	http://www.meds-sdmm.dfo-mpo.gc.ca/meds/prog_nat/benchmark/public/station_e.asp?T1=7755
			25-1961	6.14	3.08	3.060	
			87C9765	6.171	3.111	3.060	
			Average			**3.060**	

This same information is plotted in Figure 2 to better illustrate the spatial differences.

Figure 2: Relevant CHS Benchmarks



It is clear that there are significant differences in the CGD to CD offset in the study area. For the purposes of this study, the value at Point Atkinson has been assumed to be constant throughout the study area (**CGD at 3.04 m CD**) due to the reliability of that station and this study's heavy reliance on water level data derived from that location.

1.1.2 Previous Studies

An investigation of water levels in the Strait of Georgia (Point Atkinson) is included in a 2002 Master of Science thesis by University of British Columbia student, Ms. Joan Lui. The thesis compares various means for estimating the extreme flood levels; particularly the Direct Joint Probability method, the Annual Maxima method, and the Simple Addition method. The results of Ms. Lui's analysis are shown in Table 3.

Table 3: Summary of Estimates of Extreme Flood Levels (Liu, 2002)

Return Period (years)	Direct Joint Probability Method	Annual Maxima Method	Simple Addition Method
200	2.25 m CGD	2.60 m CGD	3.10 m CGD
1250	2.45 m CGD	2.68 m CGD	3.20 m CGD

The thesis contains a detailed discussion of the relative merits of the various methods that won't be repeated here. Table 4 is a listing from the thesis indicating the annual maximum water levels at Point Atkinson in the over 60-year period of record. The three largest water level events that are highlighted in yellow correspond to total water levels of 2.56 m, 2.53 m and 2.48 m CGD. In the simplest possible interpretation, this implies that the 50-year event is approximately 2.5 m which implies that the Direct Joint Probability Method as applied is non-conservative.

Table 4: Annual Maximum Water Levels at Point Atkinson (Lui, 2002)

Year	Month	Day	Time	Annual Max (m, above CHAR datum)	Year	Month	Day	Time	Annual Max (m, above CHAR datum)
1914	11	22	10	4.930	*1968*	12	24	10	5.440
1915	12	8	7	5.070	*1969*	12	11	8	5.310
1916	1	6	7	5.240	*1970*	12	7	12	5.350
1917	12	31	8	5.070	*1971*	1	15	9	5.210
1918	1	14	7	5.120	*1972*	12	22	8	5.330
1919	12	24	7	5.050	*1973*	12	13	9	5.280
1921	12	21	11	5.120	*1974*	1	14	10	5.380
1922	2	16	8	4.900	*1975*	12	26	11	5.220
1927	12	12	9	4.970	*1976*	2	18	7	5.060
1932	12	22	12	5.370	*1977*	12	15	10	5.450
1933	12	19	8	5.340	*1978*	2	9	7	5.460
1939	12	14	9	5.150	*1979*	12	24	10	5.320
1944	1	1	10	5.440	*1980*	12	26	9	5.220
1947	12	18	10	5.100	*1981*	11	14	8	5.280
1948	1	1	10	5.370	*1982*	12	16	7	5.600
1950	1	10	10	5.220	*1983*	1	27	5	5.490
1951	12	1	8	5.400	*1984*	11	27	10	5.230
1952	12	30	6	5.430	*1985*	2	11	10	5.040
1953	1	20	9	5.340	*1986*	11	18	8	5.050
1954	1	7	8	5.210	*1987*	1	3	9	5.520
1955	12	6	11	5.120	*1988*	11	22	15	5.200
1956	1	5	11	5.210	*1989*	3	11	7	4.970
1957	12	24	8	5.430	*1990*	12	4	8	5.250
1958	1	10	9	5.250	*1991*	2	2	8	5.250
1961	2	15	6	5.060	*1992*	1	25	9	5.280
1962	2	8	8	5.230	*1993*	12	13	6	5.240
1963	1	1	10	5.150	*1994*	12	19	7	5.200
1964	12	22	9	5.330	*1995*	11	29	11	5.240
1965	12	28	10	5.150	*1996*	2	20	7	5.340
1966	12	4	11	5.330	*1997*	1	1	10	5.240
1967	12	5	9	5.570					

An alternative approach to predicting storm surge was taken by Mr. Laurie Neil (Environment Canada, 2001) although he did not generate extreme water level estimates. Table 5 shows the results of his correlation analysis that could be useful for forecasting storm surge in the Strait at a future project stage.

Table 5: Storm Surge Predictive Equations – Point Atkinson (Neil, 2001)

Season	Best Predictive Equation	R	R^2	X (m)	σ	$\mathcal{E}$ (m)
Nov. Dec. Jan.	$\eta = IB + \Delta C + 2.564 + 0.233{*}T_{alongshore24} + 0.910{*}T_{crossshore24} - 0.00248{*}P^{**}$	0.835	0.696	0.057	0.049	0.072
Oct. Feb. Mar.	$\eta = IB + \Delta C + 2.497 + 0.204{*}T_{alongshore24} + 0.530{*}T_{crossshore24} - 0.0025{*}P^{**}$	0.763	0.582	-0.035	0.038	0.094

where

η	water level residual
P^{**}	pressure (mb) observed at YVR
IB	inverse barometer effect (-00948 (P – 1013.3))
ΔC	long term correction factor
R	multiple correlation coefficient.
R^2	coefficient of determination
X	statistical mean value of residual water level calculated with the corresponding equation
σ	standard deviation of residual water level calculated with the corresponding equation
e	average error of the actual water level minus predicted water level

1.1.3 Long Term Sea-Level Rise

NHC (2004) report on Lower Fraser River Flood Profile – Hydraulic Model Scoping Study contains a brief review of studies relevant to long term sea-level rise. The following is an excerpt from that reference:

> *A wide array of studies and investigations has been carried out by research organizations and government agencies to assess potential changes in global sea-level over the next century. Many of these findings have been reviewed and compiled by the Intergovernmental Panel on Climate Change (IPGCC, 2001), and Whitfield (2000) has summarized recent climate variation in Canada. According to published reports, global sea level has risen at around 2 mm/year during the last century. Most recent studies have concluded that sea level will rise at a faster rate than in the last century due to the effects of global warming. However, the range in predictions is very large. For example, the IPGCC reported that sea level could rise between 9 and 88 cm by the year 2100. The US Environmental Protection Agency has provided estimates for various future scenarios. For the median case, sea level was estimated to rise 0.5 m over the next century (5 mm/year) and there was a 1% chance of a rise of 110 cm. The two main processes contributing to the sea-level rise were the increase of seawater volume by melting ice sheets and thermal expansion of seawater. In*

addition to these solely eustatic sea-level changes, there is the potential effect of land subsidence. For example in some deltaic locations (Venice Italy, Sacramento Delta California, Nile Delta Egypt, Pampanga Delta Philippines) ongoing land subsidence can induce apparent increases in sea-level that far exceed any potential impacts from global warming. Church (2002) reviewed regional trends in land movements in the Fraser Delta and provided an overall assessment for sea-level rise on the Fraser delta in the 21st century, as summarized below.

Table 6: Sea-Level Rise on the Fraser Delta

Factor	*Low Rate (mmlyear)*	*High Rate (mmlyear)*
Increase of seawater volume by glacier melt	*1.1*	*2.2*
Thermal expansion of seawater	*1.0*	*2.0*
Land sinking (net effect of sedimentation + tectonics)	*0.7*	*1.5*
Total	*2.8*	*5.7*

These figures project sea-level to rise by between 28 and 57 cm over the coming century. These values are within the range of scenarios provided by the IGPCC. It should be emphasized that none of the current predictions of sea-level rise are considered reliable since the current predictive models are incomplete and oversimplified. Although the estimates may seem significant, in comparison, the freeboard on most sea dikes in the Fraser delta typically amounts to 1.0 m.

1.1.4 Existing Sea Dyke Elevations

Existing dyke or dyke design elevations are shown in Table 7. These values are understood to include an allowance for wave run-up and freeboard.

Table 7: Existing Dyke or Design Elevations

Location	Crest Elevation (m GSC)
Richmond Sturgeon Bank	3.4
Westham Island	2.9 to 3.3
Ladner to Canoe Pass	3.3
Boundary Bay	3.6

2. METHOD

The method used in the present investigation required completion of the following tasks:

- Assimilation of existing water level measurements in the southern Strait of Georgia;
- Development of a tidal constituent database for each of the locations shown in Figure 1;
- Development of a database of historical tide height predictions;
- Development of a database of historical storm surge events;
- Development of tide height statistics by month;
- Development of storm surge statistics by month; and,
- Development and implementation of a method for combining tide and storm surge for the production of total water level time series suitable for use in the model.

Each of these tasks is described in the following sections.

2.1 EXISTING WATER LEVEL MEASUREMENTS

Triton's existing database of historical measurements was freshened with new information from CHS, the Marine Environmental Data Service (MEDS), and the US National Oceanic and Atmospheric Administration (NOAA) databases. There are over 100 historical tidal observation stations within a 250 km radius of the river mouth. Of these, only a few have a period of record long enough to produce meaningful statistics on non-tidal effects. Table 8 lists the stations with more than 10 years of historical data. Those stations shown in bold font were analyzed in detail[1].

[1] Note that Cherry Point was identified for detailed evaluation in the study proposal, however the necessary information was unavailable from usual US NOAA channels.

Table 8: Historical Water Level Stations

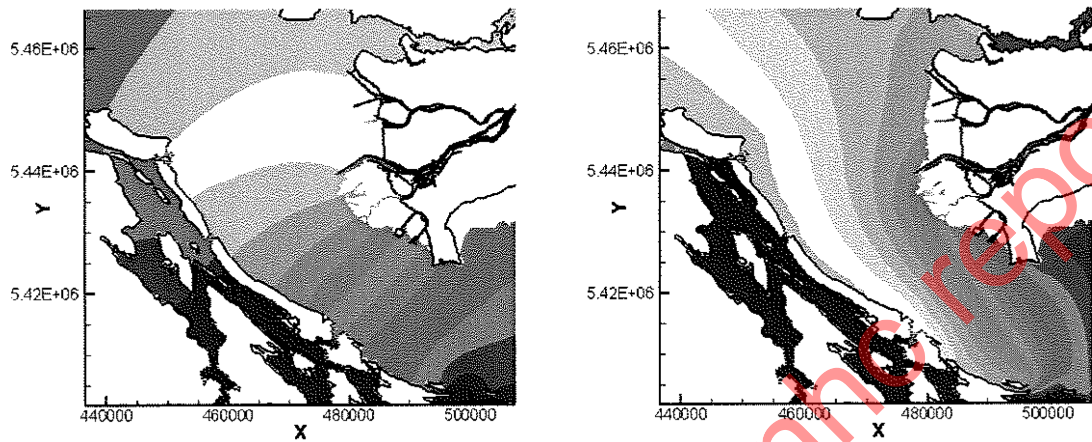
Station	Name	Lon	Lat	Years
9447130	**Seattle, WA**	**-122.34**	**47.61**	**105**
7795	**Point Atkinson, BC**	**-123.25**	**49.34**	**92**
9443090	**Neah Bay, WA**	**-124.62**	**48.37**	**72**
7735	Vancouver, BC	-123.11	49.29	66
8074	**Campbell River, BC**	**-125.25**	**50.04**	**48**
7120	**Victoria Harbour, BC**	**-123.37**	**48.42**	**42**
7330	Fulford Harbour, BC	-123.45	48.77	41
8525	Port Renfrew, BC	-124.42	48.56	30
7277	Patricia Bay, BC	-123.45	48.65	30
7607	**Steveston, BC**	**-123.18**	**49.13**	**30**
7993	Little River, BC	-124.92	49.74	28
7935	Winchelsea Is., BC	-124.09	49.30	16
7020	Sooke , BC	-123.73	48.37	14
7107	Esquimalt Lagoon, BC	-123.47	48.43	13
7510	Tumbo Channel, BC	-123.11	48.79	12
7590	**Tsawwassen, BC**	**-123.13**	**49.00**	**12**
9449424	Cherry Point, WA	-122.76	48.86	10
9449880	Friday Harbor, WA	-123.01	48.55	10
9444900	Port Townsend, WA	-122.76	48.11	10
9444090	Port Angeles, WA	-123.44	48.13	10

Following review of the various datasets, it was decided to base the analysis on measurements at Point Atkinson due to relatively long period of record, reliability of the data and proximity to the site.

2.2 TIDAL CONSTITUENT DATABASE

Independent tidal predictions at each of the mouths of the Fraser are required to reflect local differences in tidal amplitude and phase. These differences are important as they affect how the tide propagates into the lower Fraser River. A tidal constituent database was compiled using tidal constituent information in the current Canadian Hydrographic Service (CHS) database of reference and secondary tidal stations in the Strait of Georgia as a primary input. Spatial interpolation between these stations was done with the assistance of Triton's existing two-dimensional finite-element tidal harmonic model of the Strait of Georgia/Fraser Estuary. This model accurately describes the relative spatial variation of tidal height amplitude and phase. For example, Figure 3 shows the spatial variation of tide height amplitude (left figure) and phase (right figure) due to the tidal constituent M2 (twice-a-day influence of the moon). Similar results are available for the other large-amplitude constituents that together provide enough information to estimate the majority of the local tidal signal and to infer the probable response of the minor constituents that were not explicitly modelled.

Figure 3: Variation of Amplitude and Phase – Semi-Diurnal Effect of the Moon (M2)



Due to the close proximity the study area to Point Atkinson and the high reliability of that station, the Triton model results were adjusted to match perfectly the official CHS tidal constituents at Point Atkinson (CHS Station 7795). The Triton model was then probed at the boundary points shown in Figure 1 to yield tidal constituents specific to each location. The remaining Point Atkinson constituents that were not explicitly modelled were pro-rated using the trends evident in the measured and modelled data.

2.3 HISTORICAL TIDE HEIGHT PREDICTIONS

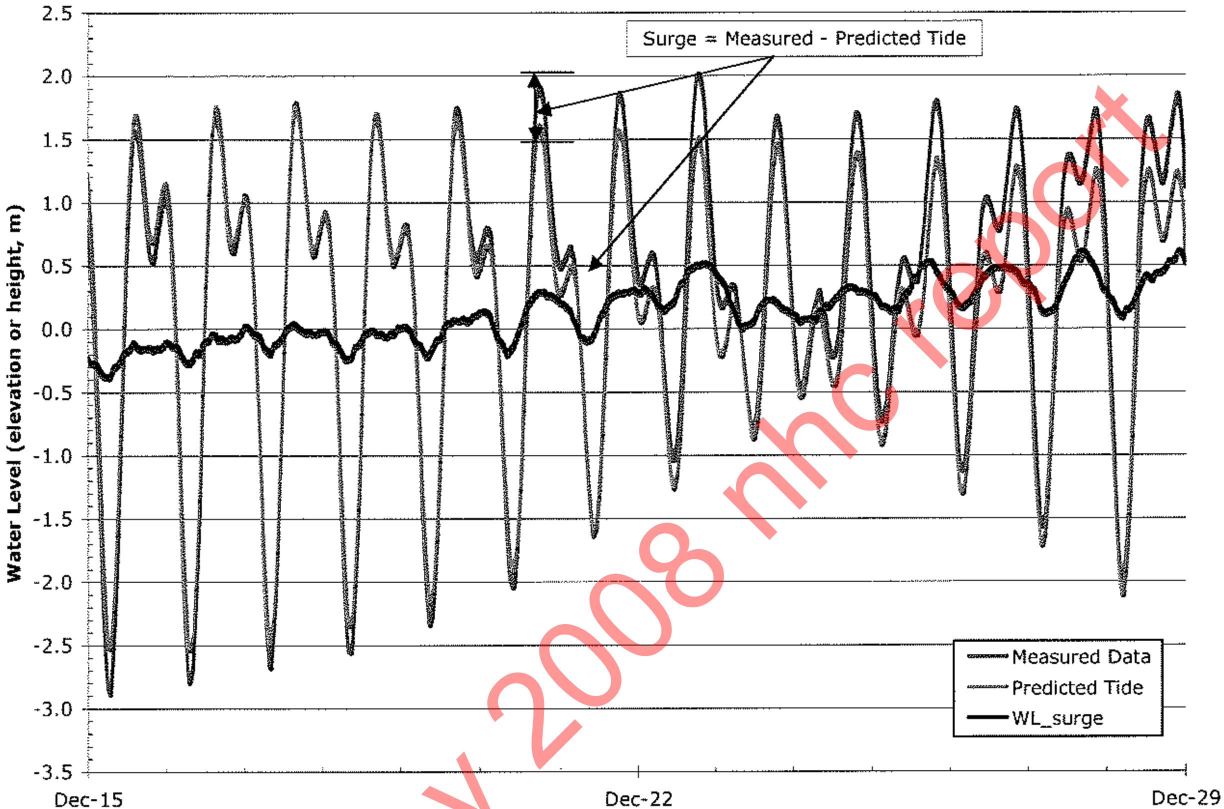
The tidal constituent definitions in the database described in Section 2.2 are consistent with those used in the Canadian Hydrographic Service (CHS) tide prediction and analysis program suite (Foreman, 1977). The tide height prediction module of the suite was used to generate hourly tide heights for 19 years which is the practical period over which tidal parameter are considered to repeat.

Tidal predictions were made for the entire time period overlapping Point Atkinson water level measurement period of record (1914 to present).

2.4 HISTORICAL STORM SURGE EVENTS

Tidal predictions for the historical period of record were made as described in Section 2.3. Differences between the measured and predicted water levels (residuals) were attributed to storm surge and other non-tidal effects (see Figure 4).

Figure 4: Storm Surge Calculation Definition Sketch



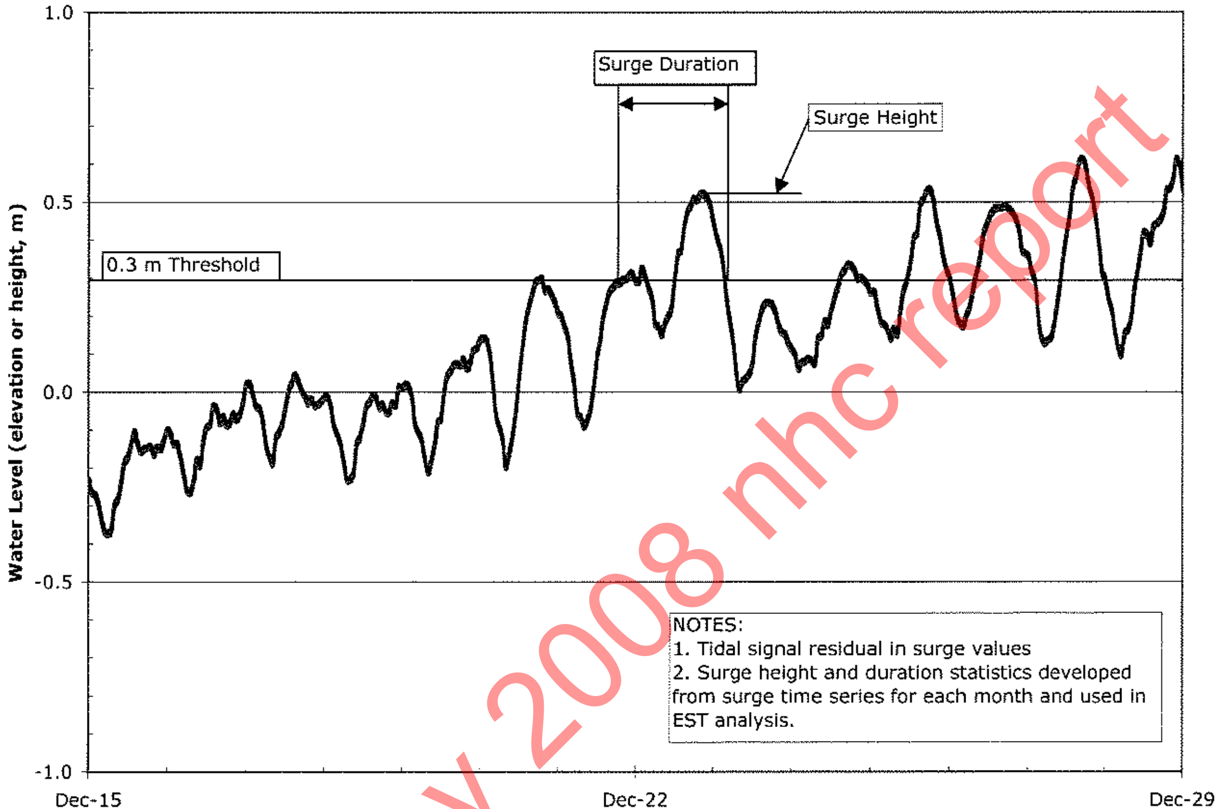
2.5 TIDE HEIGHT STATISTICS BY MONTH

Tide height statistics were developed using a subset of the historical tide height prediction dataset that corresponds to the US National Tidal Datum Epoch 1960-1978. This subset was selected to ensure that only a single full 19-year tidal repeat interval was used in computing the tidal statistics. The statistical analysis was undertaken using Triton's proprietary software package CEAPack (Coastal Engineering Analysis Package).

2.6 STORM SURGE STATISTICS BY MONTH

The storm surge time series was analysed to produce a summary of storm surge heights and durations (see).

Figure 5: Storm Surge Definition Sketch



A statistical analysis of the residuals was undertaken to develop surge height and duration statistics by month such that return periods of various storm surge events could be estimated. For the present work, differences in timing and amplitude of these non-tidal effects between Point Atkinson and the various river mouths were neglected.

2.7 COMBINING TIDE AND STORM SURGE

As noted in Section 2.6 above, the non-tidal component was treated as probabilistic with due consideration for monthly biases. The non-tidal component was added to the deterministic tide to yield the total combined water level using a method known as the Empirical Simulation Technique (EST). EST is the presently preferred method of the US Army Corps of Engineers and US Federal Emergency Management Agency (FEMA) for dealing with combined risks such as hurricane waves, water levels and tide.

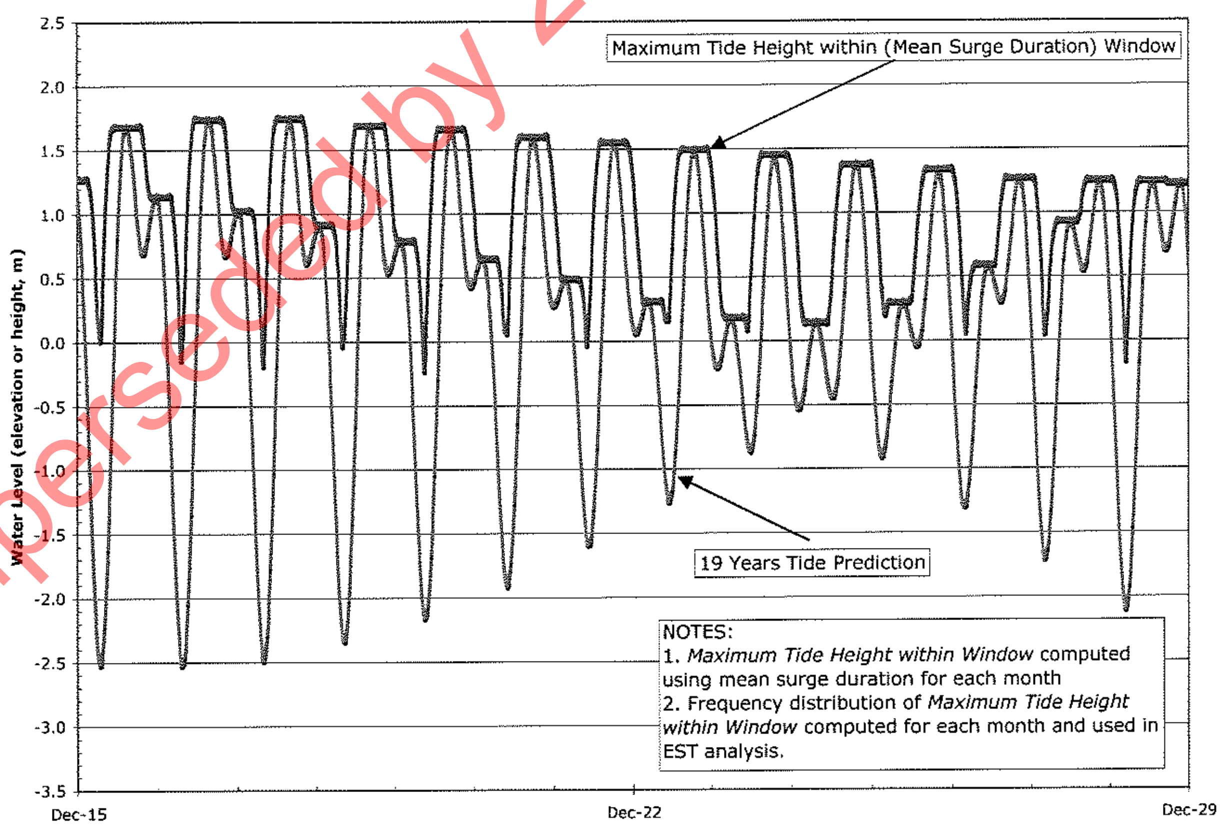
The Empirical Simulation Technique is a procedure for simulating multiple life-cycle sequences of non-deterministic multi-parameter systems such as storm events and their corresponding environmental impacts. EST is based on a *Bootstrap* re-sampling-with-replacement, interpolation, and subsequent smoothing technique in which a random sampling of a finite length database is used to generate a larger database. The only assumption is that future events will be statistically similar in magnitude and frequency to past events. EST begins with an analysis of historical events – in this case storm surge events. This database of historical events is known as the *Training Set* which is parameterized to define the characteristics of the event (e.g., maximum surge height, duration). The impacts of events may be known or may be simulated by other models (e.g., hurricane events can be characterized by parameters such as central pressure, forward speed, etc. and their impact may be simulated with appropriate hydrodynamic models – in this case that impact is measured by the combined tide and surge water level).

Parameters that describe an event are referred to as *Input Parameters* or *Input Vectors*. *Response Parameters* or *Response Vectors* define event-related impacts such as storm surge elevation, inundation, shoreline and dune erosion, etc. These Input Parameters and Response Parameters are then used as a basis for generating life-cycle simulations of event activity with corresponding impacts.

For the present application, 1000 scenarios or *realizations* of the next 1000 years were generated. EST uses these realizations to generate statistics such as those shown in Section 3.5. The following steps were followed to generate these realizations:

- The highest 50 to 100 storm surge events in each month were extracted from the Point Atkinson database and the mean event durations computed;
- Tidal level statistics were generated for each calendar month using a moving window that selects the maximum water level within the window (see Figure 6); the window duration was chosen to be equal to the mean event duration in each particular month. This ensures that the storm surge event will be paired with the maximum tidal water level that might occur during a typical event

Figure 6: Moving Window Definition Sketch



- Each historical event is assumed to be more likely to be coincident with a high probability tide level than a low one as dictated by the tide level statistics.
- EST reads the historical surge data and tide level statistics and randomly determines the number and magnitude of combined storm surge plus tide events in each of the 1000 years of prediction.
- The above is repeated 1000 times such that a relatively smooth statistical distribution is obtained

Because of the nature of the method, it is theoretically possible to predict combined water levels that are lower than the maximum due to tide alone at low return periods of limited interest to this study. For this reason, a secondary consideration was introduced that precludes selecting a combined water level estimate that is less than that due to tide alone.

2.7.1 Maximum Elevation

Maximum combined (surge plus tide) water levels are the direct output of the EST method.

2.7.2 Time Series

The Fraser Basin Model uses a time-stepping solution method, therefore it was necessary to present the design water level in the context of a realistic two-week time period. Since the Fraser River discharge analysis does not distinguish between individual winter months, January was selected for use in the Model because that month exhibits the highest tide and surge combined water level. Because there are an infinite number of possible combinations of tide and surge that will yield the desired design water level (e.g., 200 years), it was arbitrarily assumed that the peak surge would coincide with maximum January tide at Point Atkinson in the year 2000, and be whatever magnitude is necessary to yield the desired combined value.

The design winter surge event was assumed to have a triangular shape that rises from 0.3 m to the target peak value; the falling limb is the mirror image of the rising limb. The duration of the event was taken to correspond to the 95% confidence interval surge duration; this surge event was assumed to be identical at each boundary node of the model, hence it was added without modification to the individual tide predictions at each node.

The limitations of this approach are:

- This is only a single realization of a condition that yields the target 200-year tide plus surge water level at Point Atkinson; there are an infinite number of alternate realizations that could produce a worse flood condition on the river. NHC have undertaken sensitivity testing to determine whether this is likely.
- Global sea level rise is not implicitly included. For a 200-year design condition, adding the mean rise estimate at 100 years (say 4 mm/year x 100 years = 0.4 m) should really be the base case and not a "sensitivity" run.
- The approach does not contain an allowance for the effect of global climate change in terms of potential future changes in *storminess*. Current research indicates that the frequency of storms in the future is unlikely to change, however the intensity is expected to increase but this has not yet been quantified.

3. RESULTS BASED ON HISTORICAL DATA

3.1 TOTAL WATER LEVEL MEASUREMENTS

Table 9 shows estimates of various return period events based on historical annual maximum total water level measurements and ignoring the fact that the data are not purely probabilistic (i.e., they implicitly include the deterministic tidal component). Table 9 uses maximum water level data from each year where there is more than 80% hourly data coverage, a total of 55 years, and is based on a simple Gumbel analysis of total water level. This table is provided as a reasonableness check on the more sophisticated EST analyses described later.

Table 9: Point Atkinson Statistics based on Total Water Level (Mean Estimate)

Month	10 Year (m CGD)	50 Year (m CGD)	100 Year (m CGD)	200 Year (m CGD)	500 Year (m CGD)	1250 Year (m CGD)
January	2.29	2.51	2.60	2.69	2.82	2.94
February	2.13	2.34	2.43	2.52	2.63	2.76
March	1.97	2.15	2.23	2.30	2.40	2.51
April	1.78	1.93	1.99	2.06	2.14	2.22
May	1.83	1.96	2.01	2.06	2.14	2.21
June	1.88	2.00	2.05	2.09	2.16	2.22
July	1.83	1.93	1.97	2.01	2.07	2.12
August	1.75	1.86	1.90	1.95	2.01	2.06
September	1.73	1.85	1.90	1.96	2.02	2.09
October	1.87	2.95	2.12	2.19	2.29	2.39
November	2.15	2.34	2.42	2.51	2.61	2.72
December	2.38	2.62	2.70	2.80	2.93	3.05
Annual	2.40	2.59	2.67	2.75	2.86	2.96

Note: Maximum water level estimates for return periods greater than 200 years were not included within the original scope of this study. Longer return period estimates have been added to reflect the probable adoption by the City of Richmond of maximum design water level return periods of greater than 1000 years.

3.2 TIDAL CONSTITUENT DATABASE

The tidal constituents developed in this analysis are summarized in Table 10

Table 10: Tidal Height Constituents - Amplitudes (m)

Constituent	Point Atkinson	North Arm	Middle Arm	Main Arm North	Main Arm South
Z0	3.090	3.090	3.090	3.090	3.090
O1	0.483	0.483	0.482	0.480	0.474
K1	0.862	0.861	0.859	0.853	0.840
M2	0.918	0.914	0.906	0.887	0.843
S2	0.229	0.228	0.226	0.221	0.210
M4	0.007	0.007	0.007	0.007	0.006
MF	0.021	0.021	0.021	0.021	0.021
N2	0.184	0.183	0.182	0.178	0.169
SA	0.055	0.055	0.055	0.055	0.055
MM	0.034	0.034	0.034	0.034	0.034
SIG1	0.013	0.013	0.013	0.013	0.013
Q1	0.077	0.077	0.077	0.077	0.076
J1	0.047	0.047	0.047	0.046	0.046
OO1	0.027	0.027	0.027	0.027	0.026
MU2	0.020	0.020	0.020	0.019	0.018
L2	0.037	0.037	0.037	0.036	0.034
NO1	0.045	0.045	0.045	0.045	0.044
SSA	0.037	0.037	0.037	0.037	0.037
2Q1	0.018	0.018	0.018	0.018	0.018
RHO1	0.024	0.024	0.024	0.024	0.024
P1	0.268	0.268	0.267	0.265	0.261
PHI1	0.019	0.019	0.019	0.019	0.019
2N2	0.018	0.018	0.018	0.017	0.017
NU2	0.037	0.037	0.037	0.036	0.034
LDA2	0.010	0.010	0.010	0.010	0.009
K2	0.062	0.062	0.061	0.060	0.057
TAU1	0.022	0.022	0.022	0.022	0.022
M6	0.009	0.009	0.009	0.008	0.007
2MS6	0.008	0.008	0.008	0.007	0.007
MSF	0.034	0.034	0.034	0.034	0.034
MS4	0.003	0.003	0.003	0.003	0.003
MN4	0.003	0.003	0.003	0.003	0.003
M8	0.001	0.001	0.001	0.001	0.001
M3	0.002	0.002	0.002	0.002	0.002
EPS2	0.003	0.003	0.003	0.003	0.003
ETA2	0.004	0.004	0.004	0.004	0.004
2SM2	0.004	0.004	0.004	0.004	0.004
MO3	0.003	0.003	0.003	0.003	0.003
SO3	0.001	0.001	0.001	0.001	0.001
MK3	0.004	0.004	0.004	0.004	0.004
SK3	0.002	0.002	0.002	0.002	0.002
SN4	0.001	0.001	0.001	0.001	0.001

Constituent	Point Atkinson	North Arm	Middle Arm	Main Arm North	Main Arm South
MK4	0.004	0.004	0.004	0.004	0.003
2MN6	0.005	0.005	0.005	0.005	0.004
MSN6	0.002	0.002	0.002	0.002	0.002
2MK6	0.002	0.002	0.002	0.002	0.002
2SM6	0.002	0.002	0.002	0.002	0.002
MSK6	0.002	0.002	0.002	0.002	0.002

Table 11: Tidal Height Constituents - Phase (deg UTC)

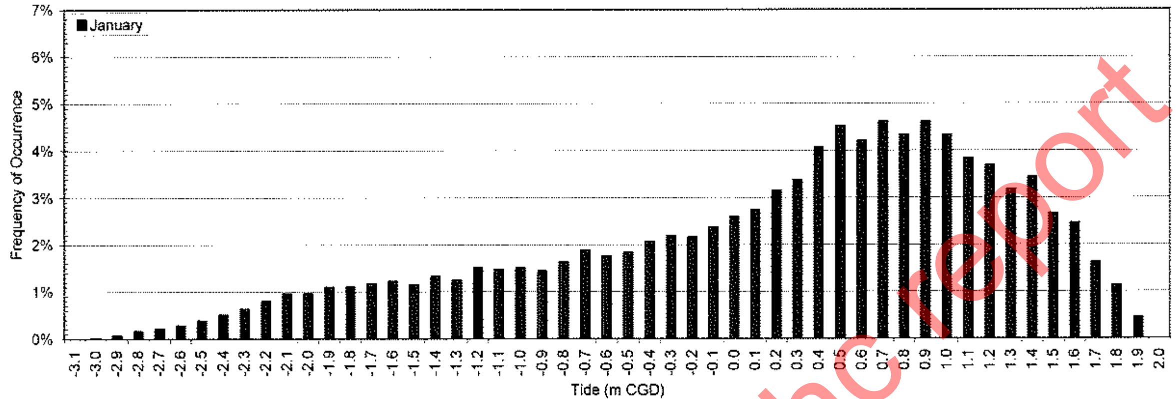
Constituent	Point Atkinson	North Arm	Middle Arm	Main Arm North	Main Arm South
Z0	0.0	0.0	0.0	0.0	0.0
O1	263.2	263.2	263.1	262.9	262.4
K1	286.1	286.0	286.0	285.7	285.2
M2	31.2	31.1	30.9	30.4	29.0
S2	59.9	59.8	59.6	59.1	57.6
M4	271.7	271.5	271.2	270.4	268.1
MF	76.6	76.6	76.6	76.6	76.6
N2	2.9	2.8	2.6	2.1	0.7
SA	344.3	344.3	344.3	344.3	344.3
MM	108.0	108.0	108.0	108.0	108.0
SIG1	282.6	282.6	282.5	282.3	282.0
Q1	258.8	258.8	258.7	258.5	258.1
J1	323.9	323.8	323.7	323.5	322.9
OO1	331.4	331.3	331.2	331.0	330.4
MU2	292.0	291.9	291.7	291.2	289.9
L2	89.8	89.7	89.5	89.0	87.6
NO1	288.0	287.9	287.9	287.6	287.2
SSA	215.7	215.7	215.7	215.7	215.7
2Q1	263.2	263.2	263.1	262.9	262.6
RHO1	248.8	248.8	248.7	248.5	248.1
P1	283.1	283.0	283.0	282.7	282.2
PHI1	288.1	288.0	288.0	287.7	287.2
2N2	343.7	343.6	343.4	342.9	341.6
NU2	11.7	11.6	11.4	10.9	9.5
LDA2	68.4	68.3	68.1	67.6	66.2
K2	59.9	59.8	59.6	59.1	57.6
TAU1	353.4	353.4	353.3	353.1	352.6
M6	59.3	59.1	58.7	57.7	54.9
2MS6	92.9	92.6	92.3	91.3	88.5
MSF	200.5	200.5	200.5	200.5	200.5
MS4	275.6	275.4	275.1	274.3	272.0
MN4	246.5	246.3	246.0	245.2	242.9

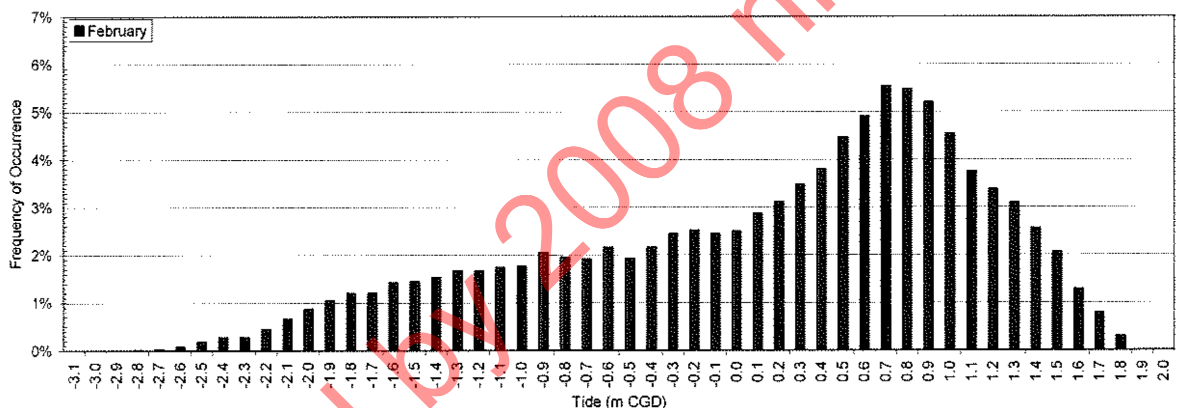
Constituent	Point Atkinson	North Arm	Middle Arm	Main Arm North	Main Arm South
M8	59.3	59.0	58.6	57.6	54.4
M3	231.3	231.1	230.9	230.2	228.3
EPS2	187.0	186.9	186.7	186.2	184.9
ETA2	270.0	269.9	269.7	269.1	267.7
2SM2	281.3	281.2	281.0	280.4	279.0
MO3	358.6	358.4	358.2	357.5	355.6
SO3	163.7	163.5	163.3	162.6	160.7
MK3	167.2	167.0	166.8	166.1	164.2
SK3	216.8	216.6	216.4	215.7	213.7
SN4	252.8	252.6	252.3	251.5	249.2
MK4	48.0	47.8	47.5	46.7	44.4
2MN6	31.2	31.0	30.6	29.6	26.8
MSN6	42.5	42.3	41.9	40.9	38.1
2MK6	106.7	106.4	106.1	105.1	102.3
2SM6	113.8	113.5	113.2	112.2	109.4
MSK6	95.1	94.8	94.5	93.5	90.7

3.3 TIDE HEIGHT STATISTICS BY MONTH

Figure 7 through Figure 10 show histograms of hourly tidal elevation for each month. Figure 11 shows a comparison between months of the frequency of occurrence of high water levels. Note that only November, December and January exhibit hourly tidal water levels of 1.9 m CGD with December having the highest frequency of this level.

Figure 7: Tidal Elevation Histogram – January, February and March





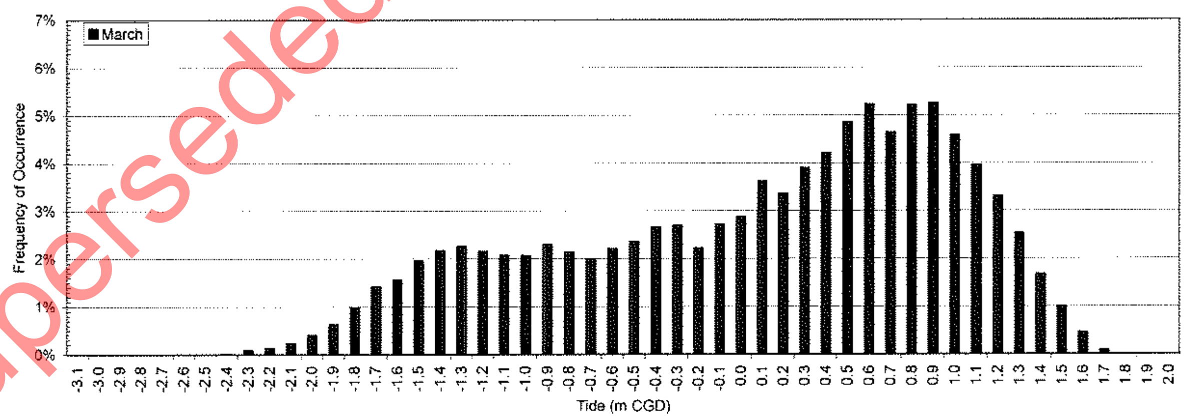


Figure 8: Tidal Elevation Histogram – April, May and June

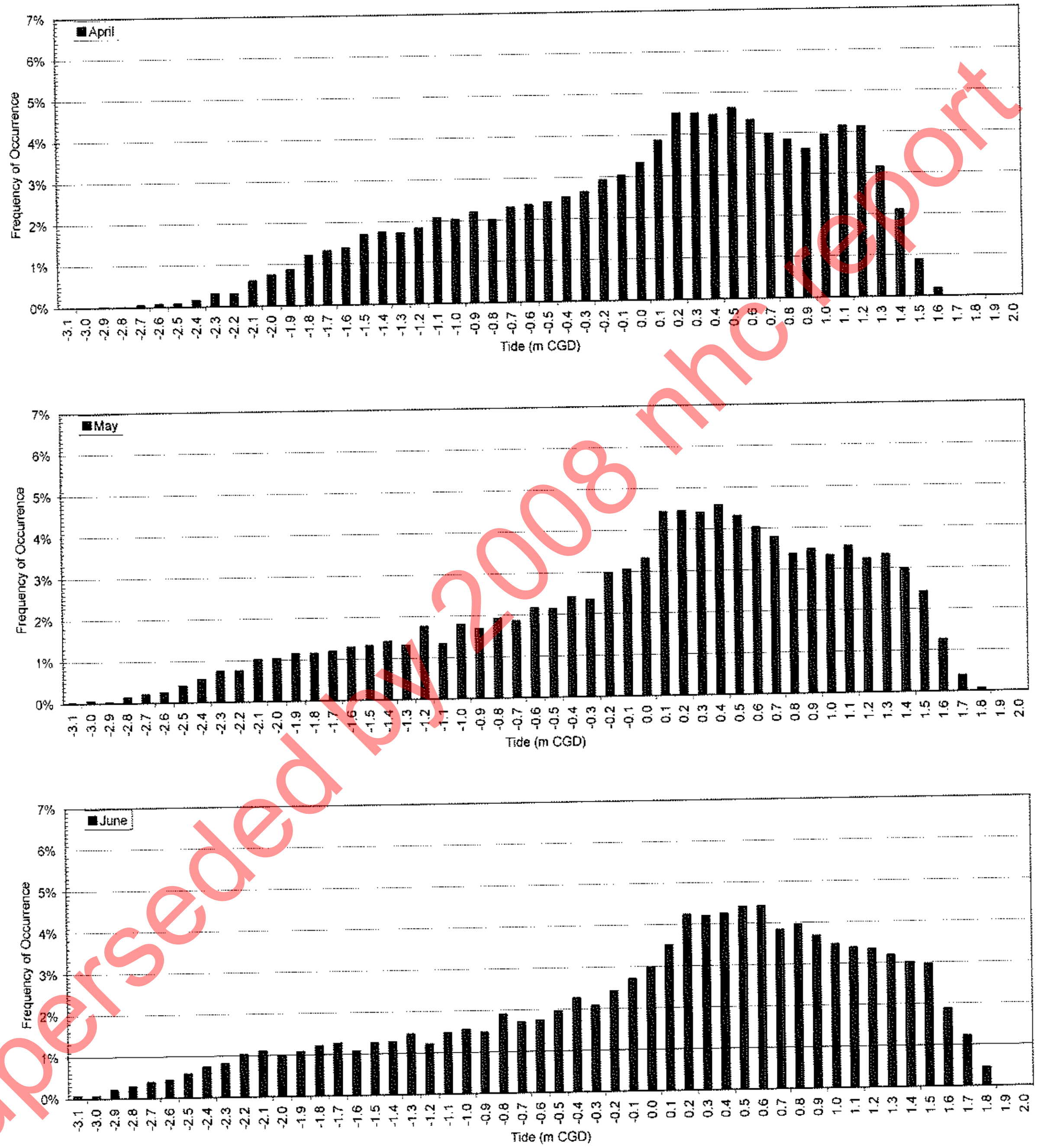


Figure 9: Tidal Elevation Histogram – July, August and September

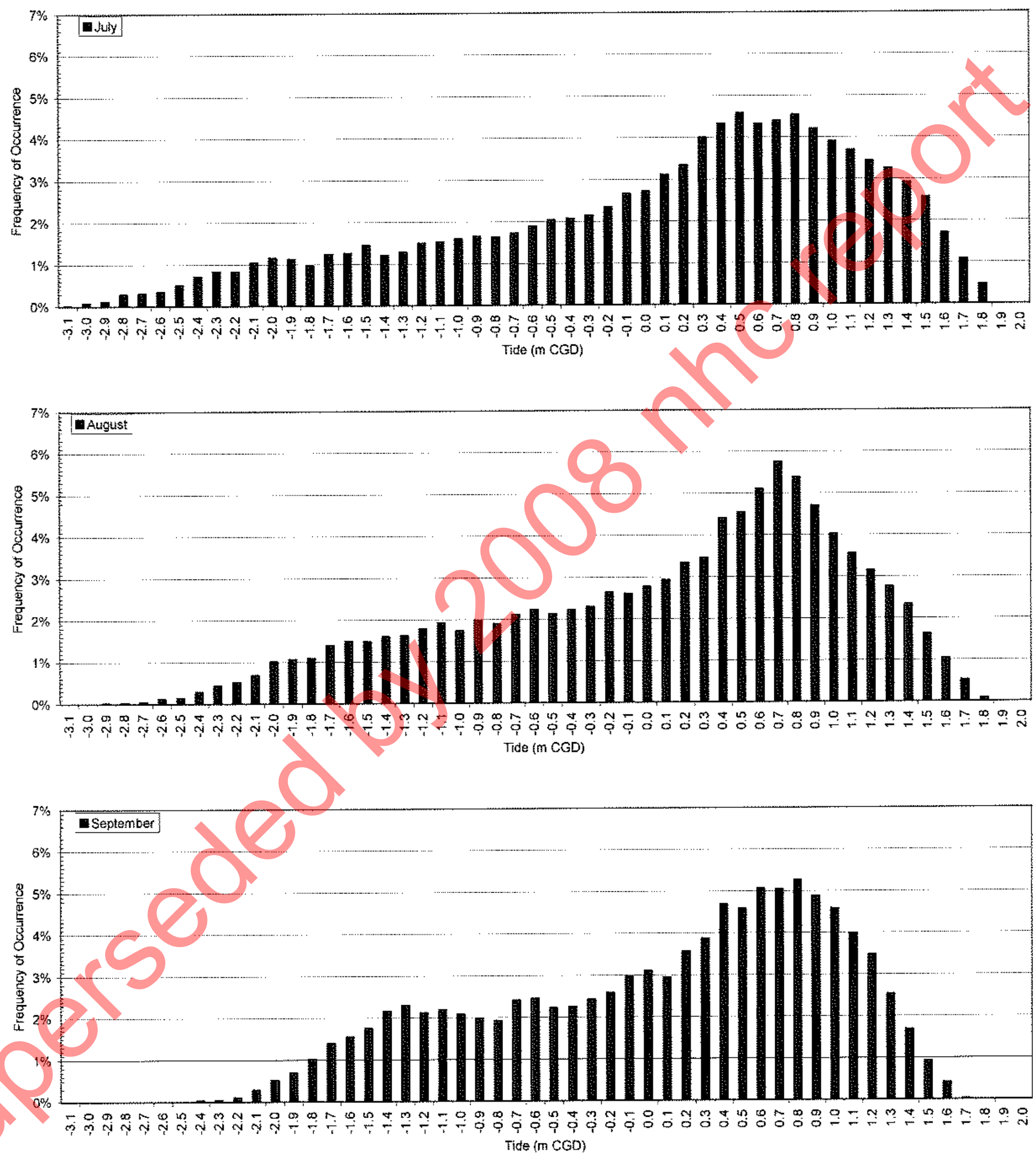


Figure 10: Tidal Elevation Histogram – October, November and December

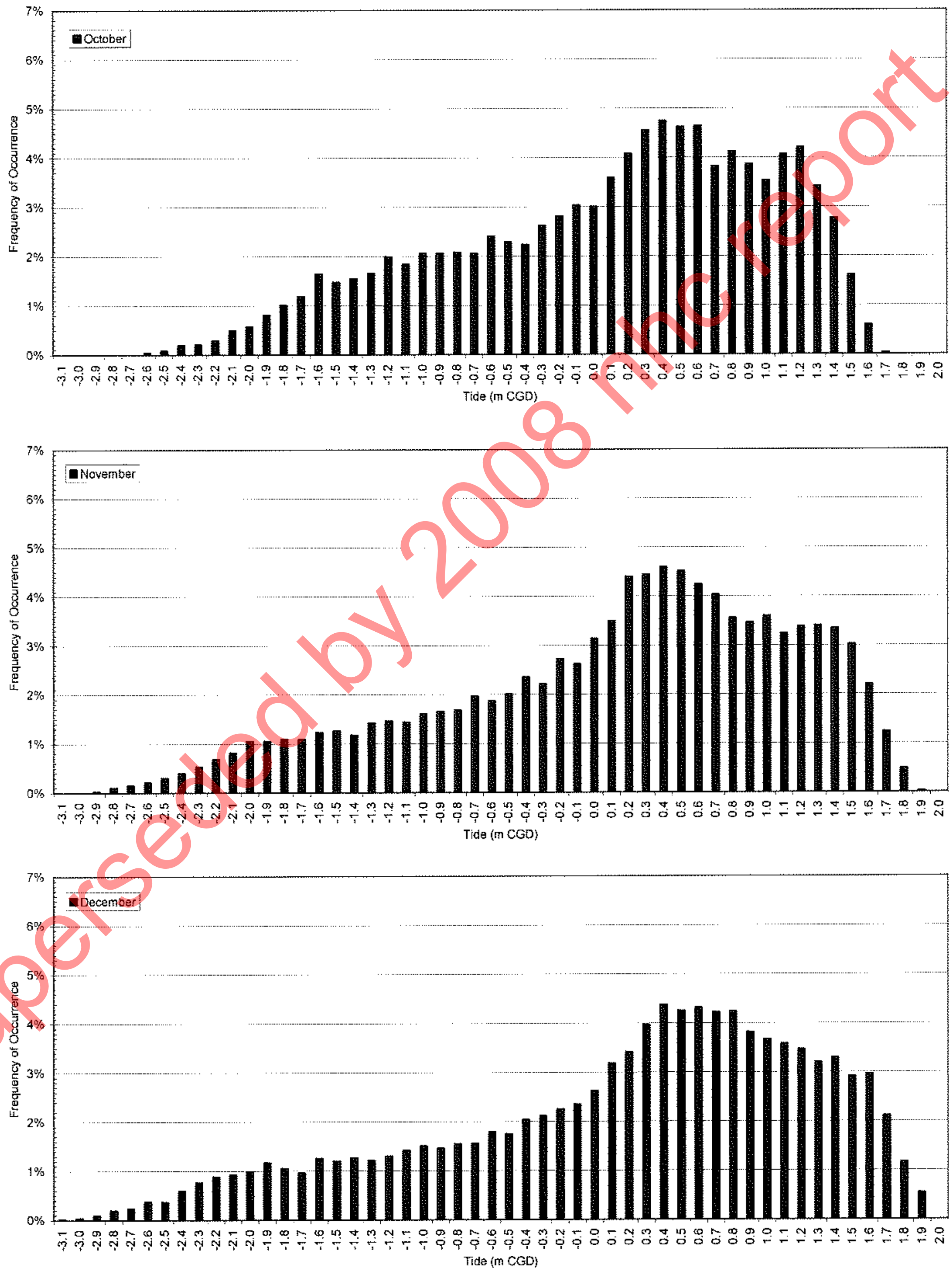
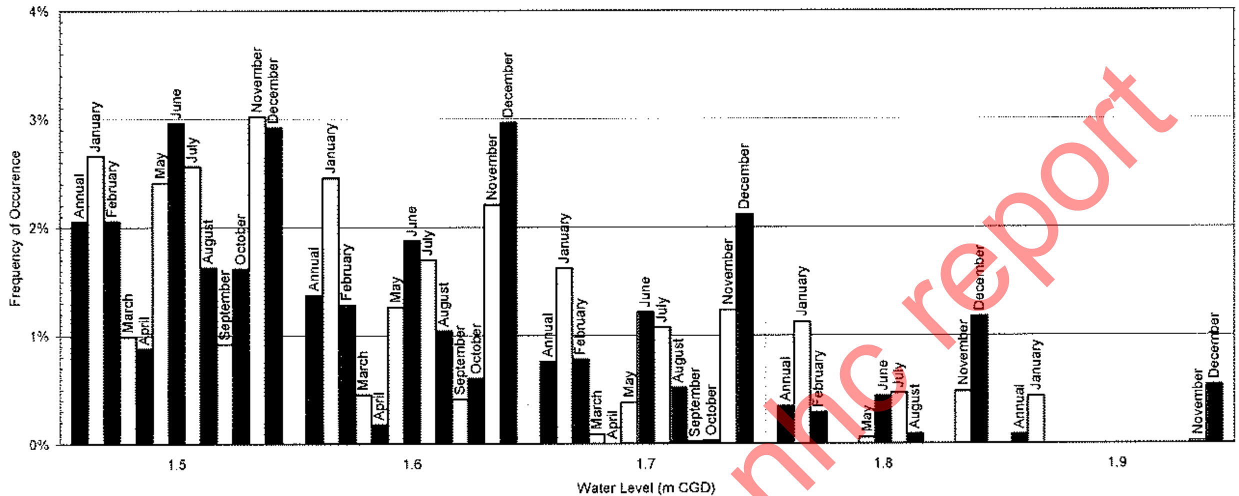


Figure 11: Point Atkinson Tidal Elevation Histogram – WL ≥ 1.5 m CGD



3.4 STORM SURGE STATISTICS

Table 12 contains a chronological listing of historical annual storm surge events.

Table 12: Chronological Listing of Historical Storm Surge Events

Number (-)	UTC Start Time YYYY-MM-DD HH:MM:SS	UTC End Time YYYY-MM-DD HH:MM:SSZ	Duration (h)	Surge (m)
1	1950-10-27 17:00	1950-10-27 17:00	1	0.66
2	1950-10-28 10:00	1950-10-28 13:00	4	0.67
3	1951-01-15 17:00	1951-01-15 20:00	4	0.71
4	1951-11-27 22:00	1951-11-28 04:00	7	0.77
5	1951-11-29 08:00	1951-11-29 09:00	2	0.74
6	1951-11-30 23:00	1951-12-01 07:00	9	0.81
7	1952-11-14 19:00	1952-11-14 23:00	5	0.72
8	1952-12-05 00:00	1952-12-05 01:00	2	0.69
9	1952-12-07 12:00	1952-12-07 20:00	9	0.86
10	1952-12-30 13:00	1952-12-30 18:00	6	0.71
11	1953-01-09 14:00	1953-01-09 19:00	6	0.76
12	1954-01-04 11:00	1954-01-04 11:00	1	0.65
13	1954-01-05 22:00	1954-01-05 22:00	1	0.65
14	1954-02-13 08:00	1954-02-13 09:00	2	0.67
15	1954-02-18 00:00	1954-02-18 01:00	2	0.68
16	1957-02-24 11:00	1957-02-24 11:00	1	0.65
17	1957-12-24 15:00	1957-12-24 16:00	2	0.69
18	1961-03-13 01:00	1961-03-13 03:00	3	0.68
19	1962-11-25 23:00	1962-11-26 05:00	7	0.78
20	1963-03-28 13:00	1963-03-28 13:00	1	0.69

Number (-)	UTC Start Time YYYY-MM-DD HH:MM:SS	UTC End Time YYYY-MM-DD HH:MM:SSZ	Duration (h)	Surge (m)
21	1963-10-24 12:00	1963-10-24 18:00	7	0.84
22	1964-01-20 02:00	1964-01-20 07:00	6	0.80
23	1965-01-03 09:00	1965-01-03 13:00	5	1.11
24	1967-12-03 10:00	1967-12-03 11:00	2	0.66
25	1967-12-05 13:00	1967-12-05 17:00	5	0.76
26	1969-12-11 23:00	1969-12-12 03:00	5	0.81
27	1969-12-23 04:00	1969-12-23 10:00	7	0.78
28	1973-01-17 08:00	1973-01-17 10:00	3	0.71
29	1973-01-19 09:00	1973-01-19 10:00	2	0.71
30	1973-01-21 01:00	1973-01-21 01:00	1	0.71
31	1973-12-13 01:00	1973-12-13 02:00	2	0.70
32	1974-01-15 13:00	1974-01-15 18:00	6	0.76
33	1975-11-15 10:00	1975-11-15 15:00	6	0.79
34	1977-11-01 21:00	1977-11-01 22:00	2	0.69
35	1978-01-05 22:00	1978-01-06 02:00	5	0.84
36	1978-02-07 23:00	1978-02-08 05:00	7	0.78
37	1978-02-09 04:00	1978-02-09 15:00	12	0.69
38	1979-02-13 12:00	1979-02-13 20:00	9	0.76
39	1980-01-12 11:00	1980-01-12 18:00	8	0.77
40	1980-03-13 09:00	1980-03-13 11:00	3	0.69
41	1980-11-29 14:00	1980-11-29 18:00	5	0.75
42	1981-11-14 20:00	1981-11-15 05:00	10	0.94
43	1981-12-05 14:00	1981-12-05 20:00	7	0.83
44	1982-11-30 19:00	1982-12-01 00:00	6	0.69
45	1982-12-03 21:00	1982-12-03 21:00	1	0.66
46	1982-12-16 08:00	1982-12-16 20:00	13	0.95
47	1982-12-17 09:00	1982-12-17 16:00	8	0.85
48	1982-12-19 03:00	1982-12-19 12:00	10	0.75
49	1982-12-22 13:00	1982-12-22 16:00	4	0.72
50	1983-01-26 06:00	1983-01-26 10:00	5	0.72
51	1983-01-27 02:00	1983-01-27 16:00	15	1.00
52	1983-02-11 13:00	1983-02-11 13:00	1	0.65
53	1983-11-15 18:00	1983-11-16 02:00	9	0.69
54	1983-11-17 13:00	1983-11-17 16:00	4	0.71
55	1983-11-18 03:00	1983-11-18 03:00	1	0.65
56	1983-11-24 23:00	1983-11-25 01:00	3	0.68
57	1984-10-12 21:00	1984-10-12 23:00	3	0.68
58	1984-11-02 20:00	1984-11-02 21:00	2	0.69
59	1986-01-18 15:00	1986-01-18 16:00	2	0.70

Number (-)	UTC Start Time YYYY-MM-DD HH:MM:SS	UTC End Time YYYY-MM-DD HH:MM:SSZ	Duration (h)	Surge (m)
60	1987-01-03 15:00	1987-01-03 16:00	2	0.65
61	1987-12-01 16:00	1987-12-01 19:00	4	0.74
62	1987-12-03 12:00	1987-12-03 14:00	3	0.72
63	1987-12-09 13:00	1987-12-09 18:00	6	0.82
64	1991-11-17 12:00	1991-11-17 15:00	4	0.69
65	1992-01-31 22:00	1992-02-01 06:00	9	0.83
66	1994-12-19 14:00	1994-12-19 23:00	10	0.74
67	1994-12-20 13:00	1994-12-20 14:00	2	0.68
68	1995-03-10 02:00	1995-03-10 08:00	7	0.92
69	1995-03-11 02:00	1995-03-11 08:00	7	0.73
70	1995-03-12 05:00	1995-03-12 05:00	1	0.66
71	1995-03-21 09:00	1995-03-21 16:00	8	0.84
72	1995-12-12 21:00	1995-12-13 11:00	15	0.80
73	1996-02-20 17:00	1996-02-20 18:00	2	0.67
74	1997-01-01 11:00	1997-01-01 19:00	9	0.98
75	1997-01-02 14:00	1997-01-02 16:00	3	0.68
76	1998-02-07 11:00	1998-02-07 14:00	4	0.68
77	1998-02-12 15:00	1998-02-13 01:00	11	0.70
78	1998-02-21 04:00	1998-02-21 14:00	11	0.82
79	1998-03-24 06:00	1998-03-24 06:00	1	0.65
80	1999-03-03 15:00	1999-03-03 21:00	7	0.81
81	2001-12-01 14:00	2001-12-02 04:00	15	0.82
82	2002-12-15 01:00	2002-12-15 01:00	1	0.65
83	2002-12-16 06:00	2002-12-16 18:00	13	0.81
84	2003-01-02 17:00	2003-01-02 21:00	5	0.72
85	2003-01-03 11:00	2003-01-03 14:00	4	0.78
86	2003-03-15 19:00	2003-03-15 19:00	1	0.66
87	2005-11-04 00:00	2005-11-04 01:00	2	0.67

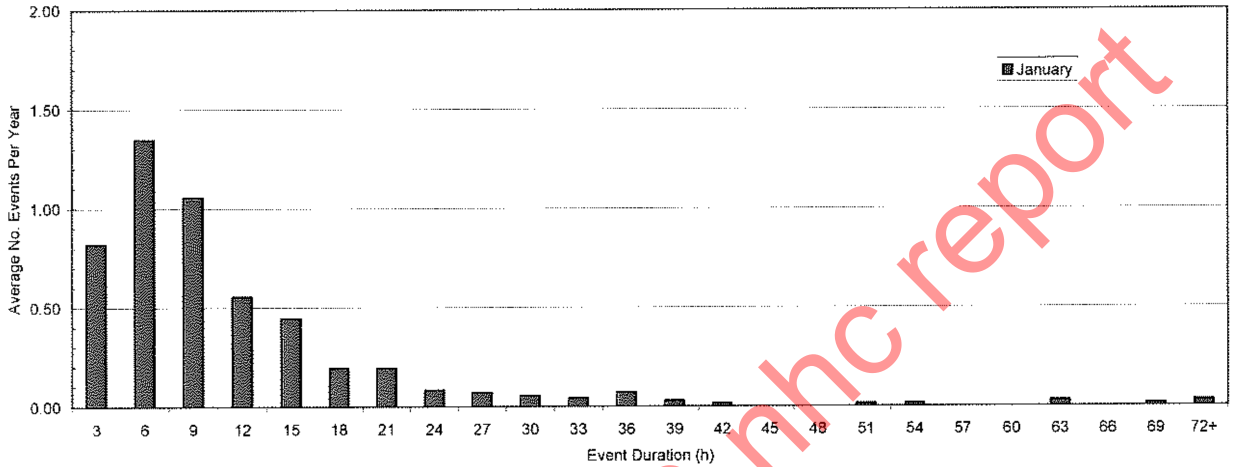
Table 12 does not include recent high water level events that occurred over New Year's 2006 and February 2006. During December and January, there were four occasions that measured total water levels exceeded 2.3 m CGD.

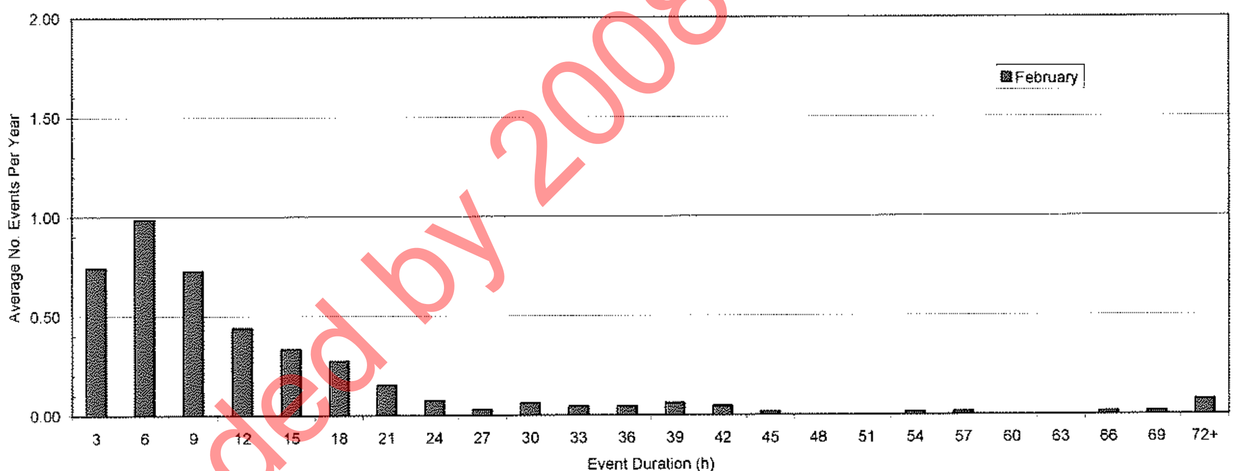
Table 13 shows the historical monthly Point Atkinson storm surge primary statistics in 68 partial years.

Table 13: Historical Point Atkinson Surge Statistics

Month	Max. Surge (m)	No. of Events > 0.3 m (-)	No. of Years (-)	Mean Events/Year (-)	Mean Event Duration (h)
January	1.11	365	72	5.1	10
February	0.84	275	66	4.2	11
March	0.92	278	66	4.2	9
April	0.61	144	67	2.1	7
May	0.61	100	68	1.5	4
June	0.58	96	67	1.4	4
July	0.54	58	65	0.9	4
August	0.49	36	64	0.6	3
September	0.68	60	65	0.9	5
October	0.82	195	65	3.0	8
November	0.93	277	65	4.3	11
December	0.96	379	66	5.7	11

Figure 12: Surge Duration Distribution - January, February and March





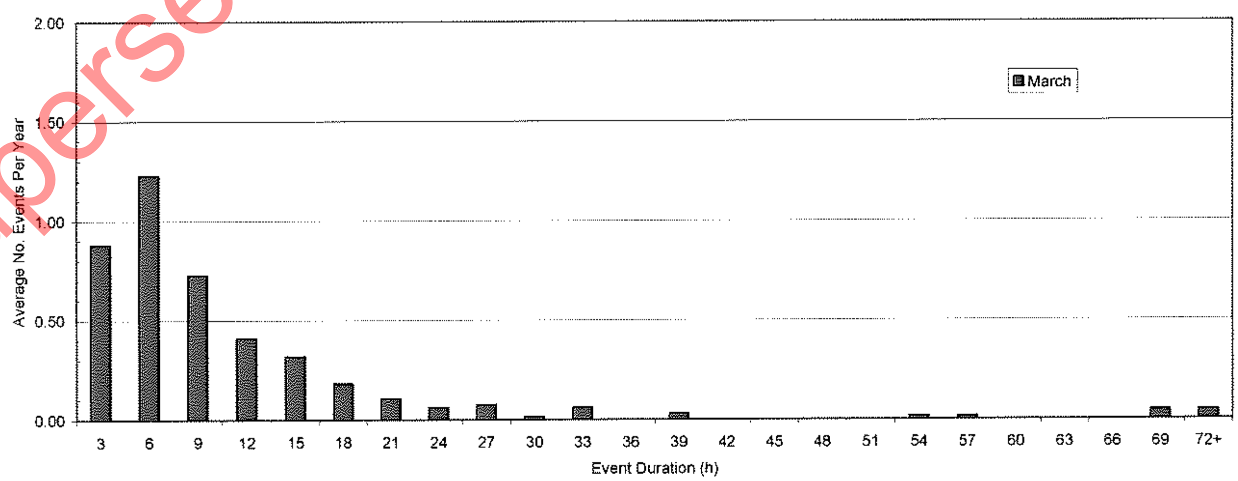
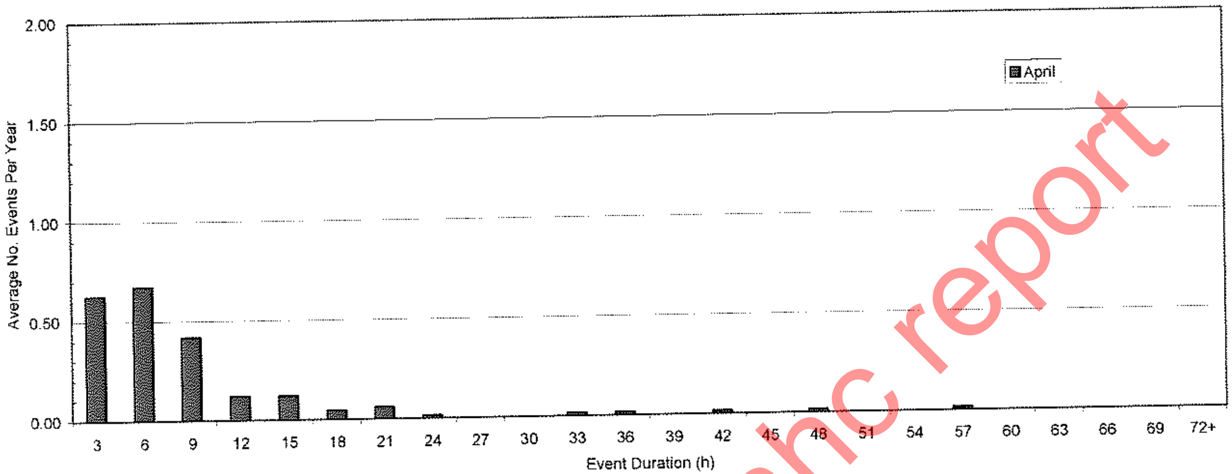


Figure 13: Surge Duration Distribution – April, May and June



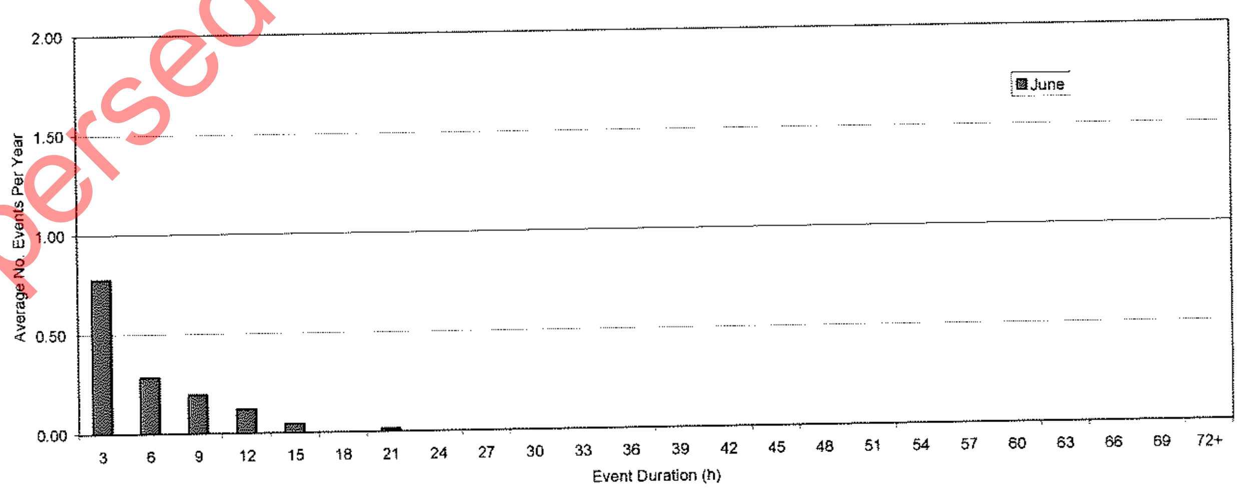


Figure 14: Surge Duration Distribution - July, August and September

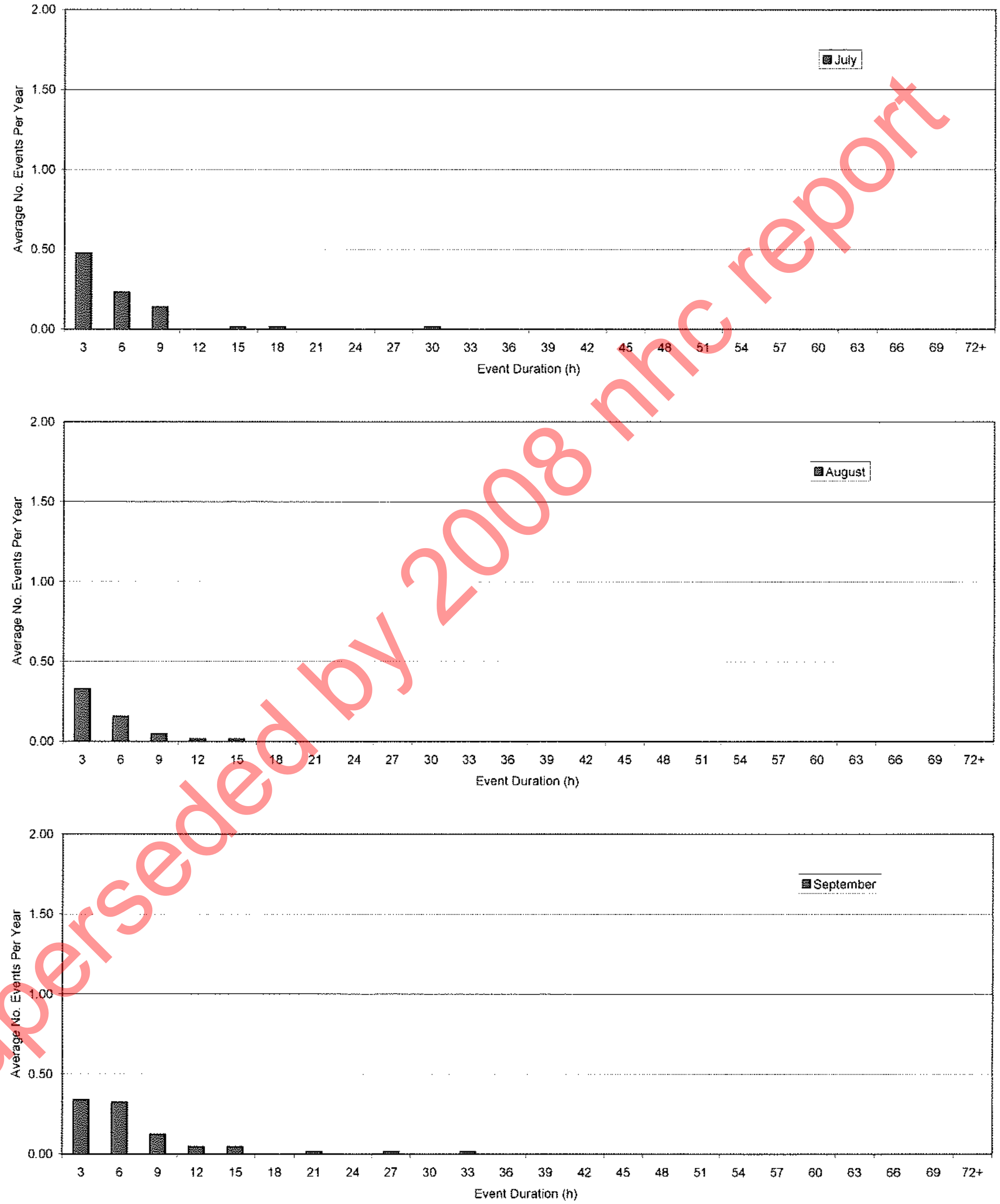
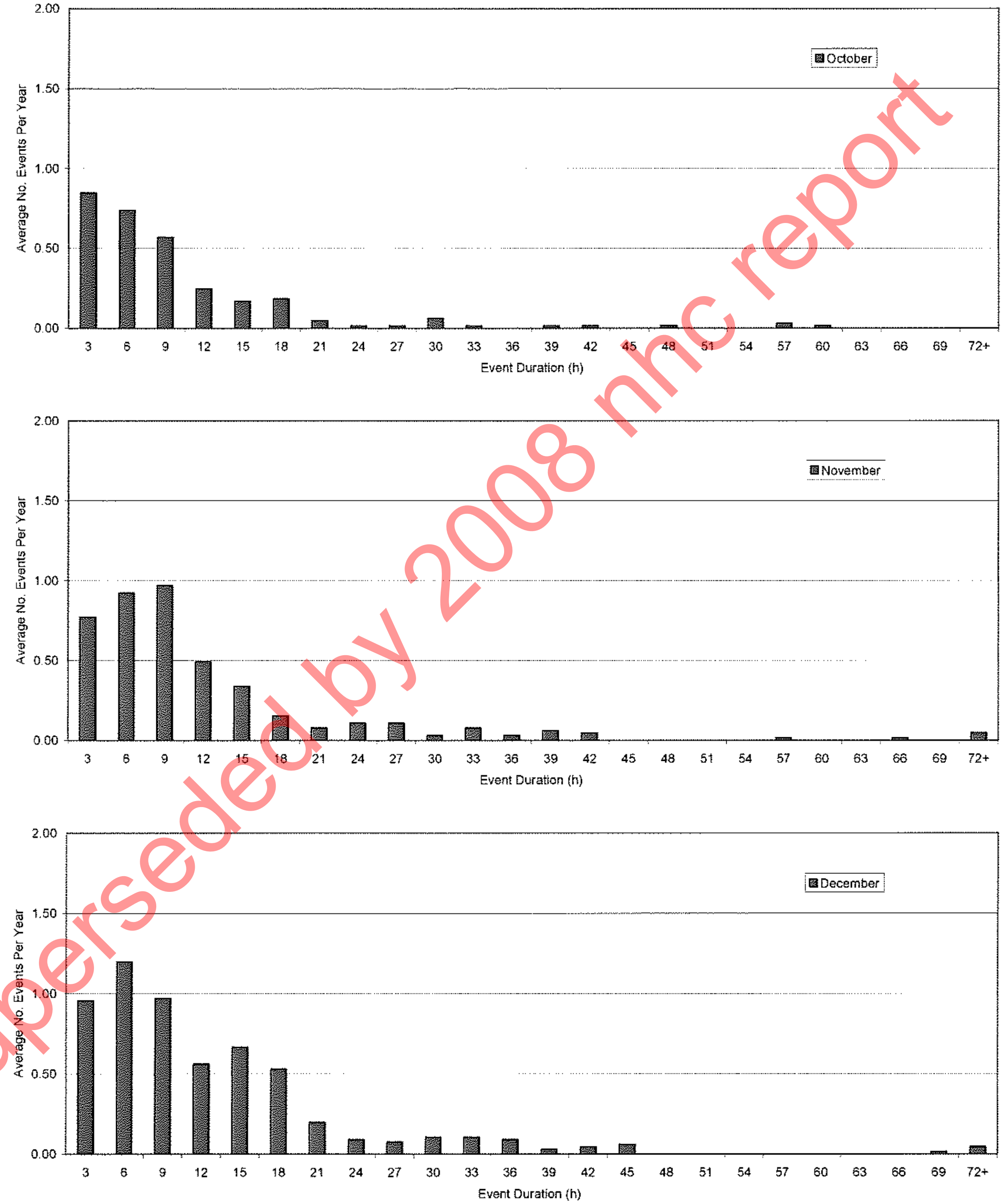


Figure 15: Surge Duration Distribution - October, November and December



3.5 COMBINING TIDE AND STORM SURGE

3.5.1 Maximum Elevation

The EST method described in Section 2.7 was used to generate the Figure 16 through Figure 19. Each figure shows the mean and ±95% confidence interval (CI) estimates of total water level at return periods between 10 and 1000 years.

It is recommended that the 95% CI values be used. In those cases in which the water level due to tides alone (shown in red) are greater than that due to tides-plus-surge, the tidal water level should be used.

Figure 16: Combined Water Level Exceedance Graph - January, February and March

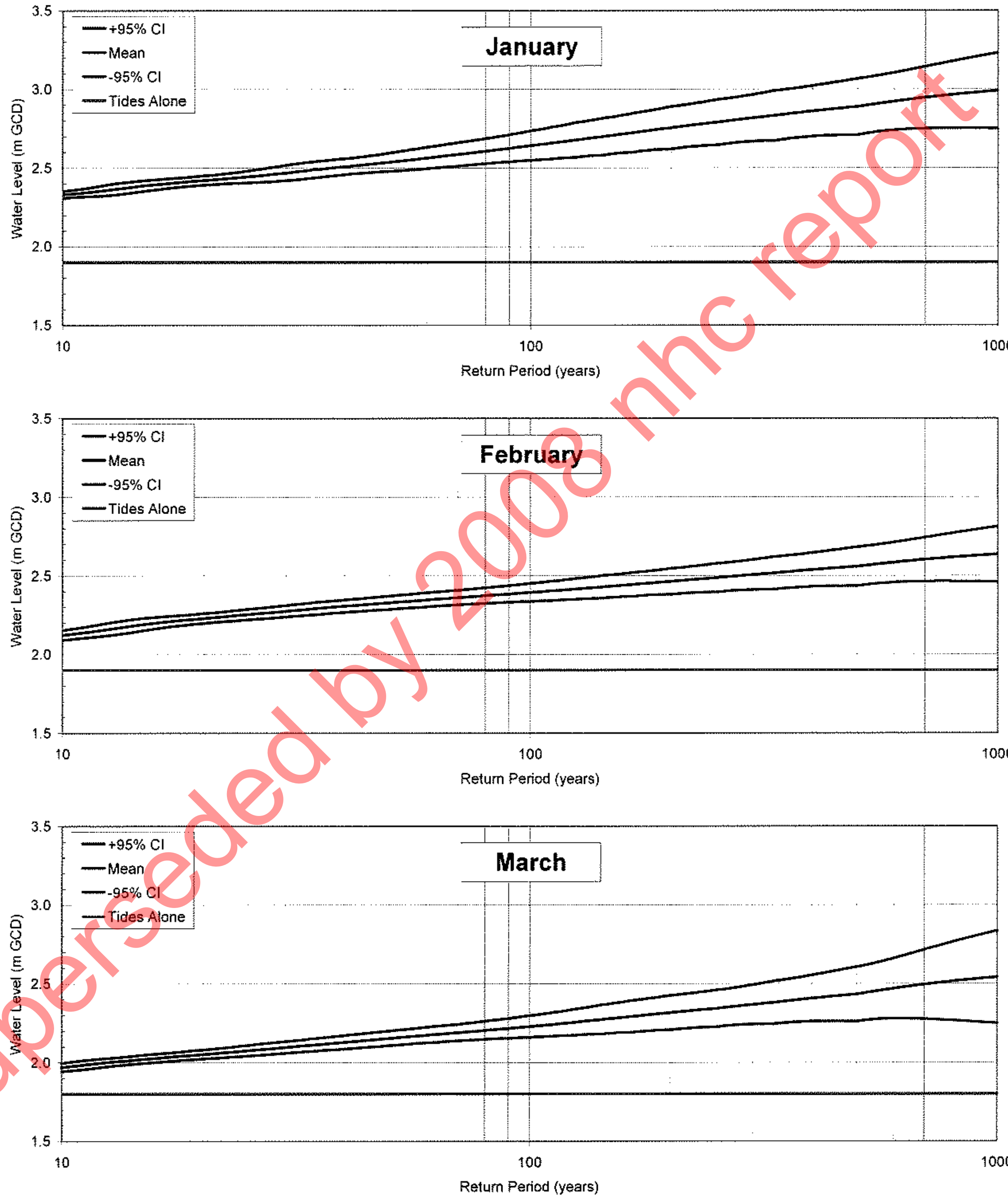


Figure 17: Combined Water Level Exceedance Graph – April, May and June

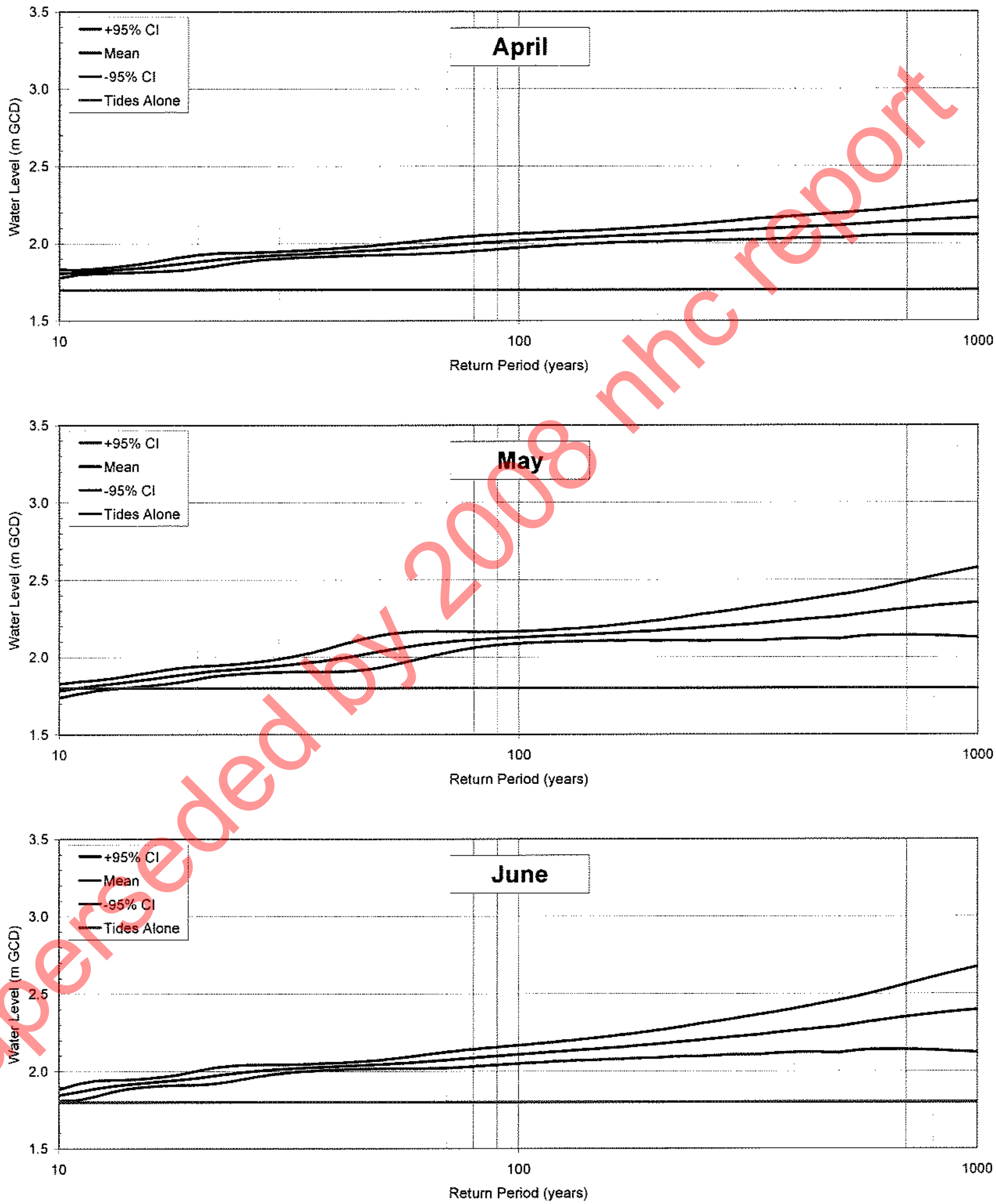


Figure 18: Combined Water Level Exceedance Graph – July, August and September

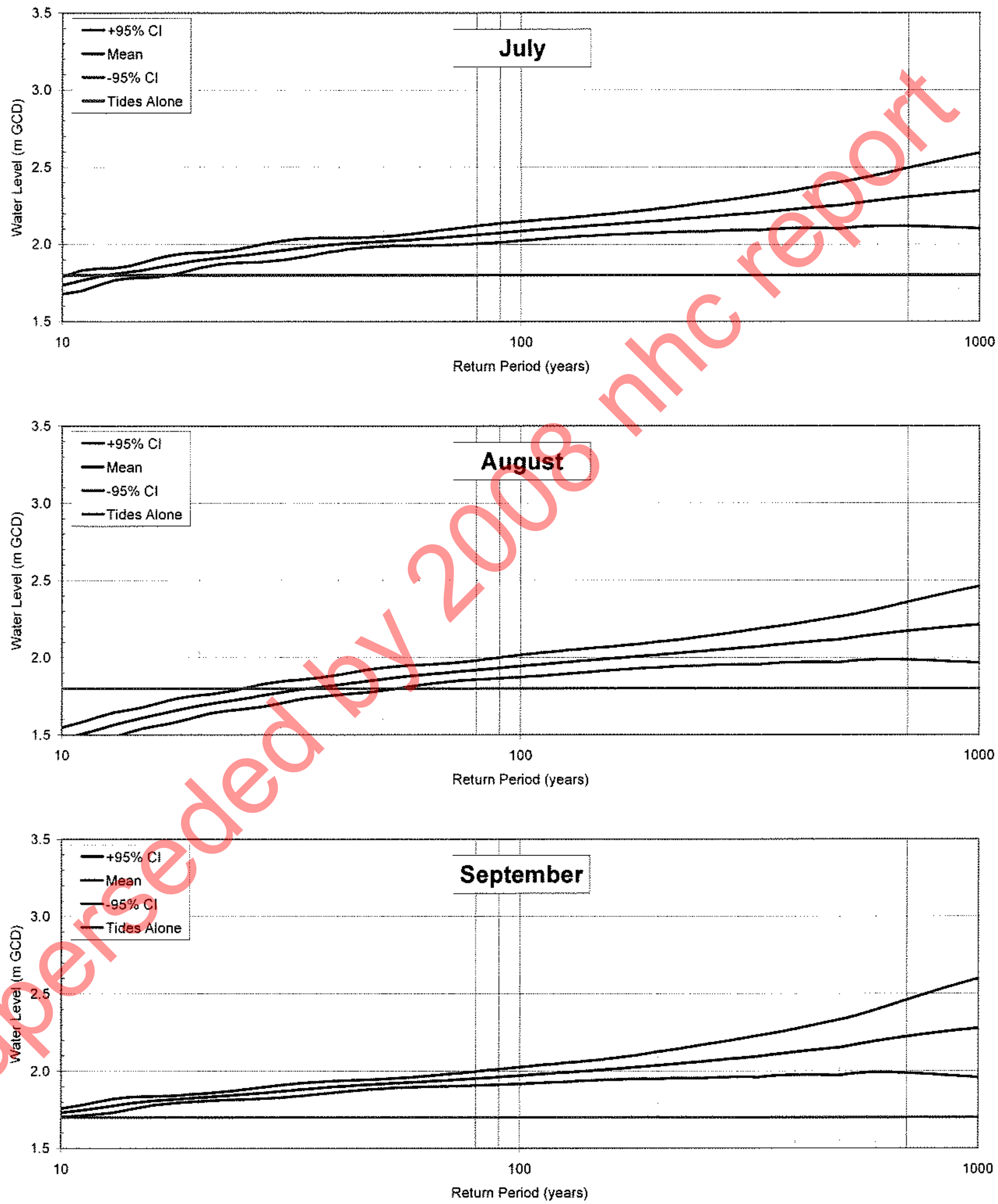
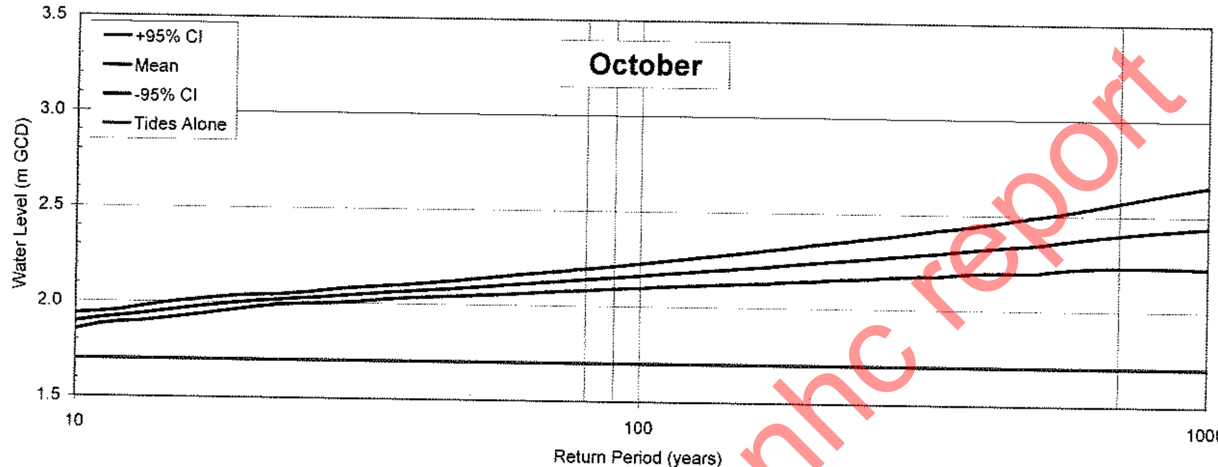
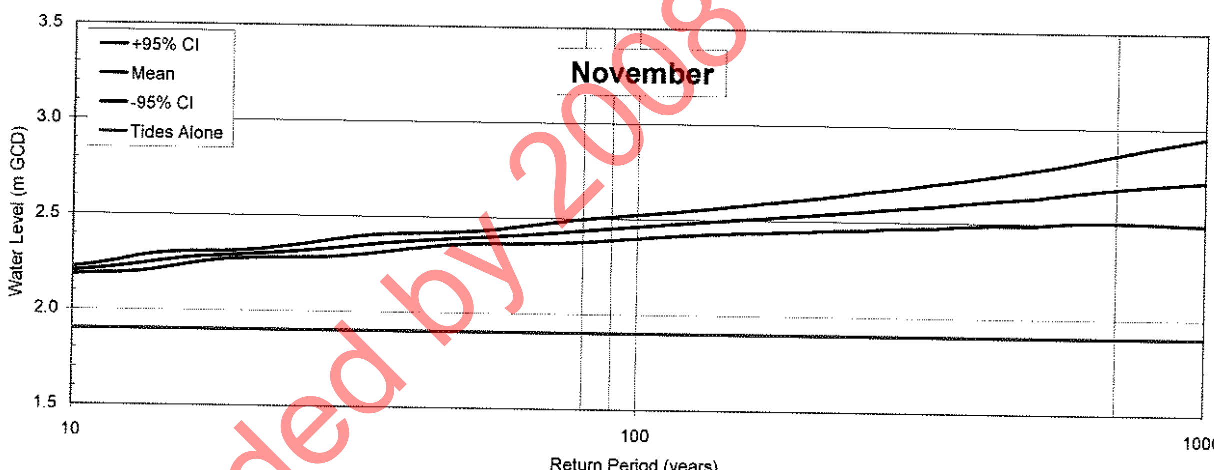


Figure 19: Combined Water Level Exceedance Graph – October, November and December





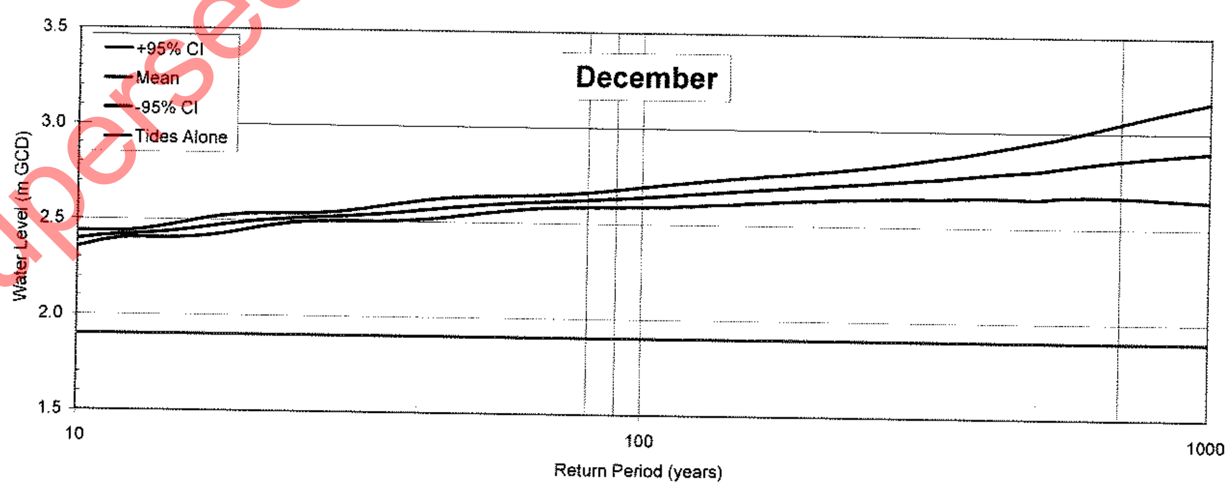


Table 15 and Table 15 summarize the key values from Figure 16 through Figure 19 based the mean and 95% CI; the 95% CI are preferred.

Table 14: Point Atkinson Statistics based on EST Analysis (Mean Estimate) (excluding impact of Global Climate Change)

Month	10 Year (m CGD)	50 Year (m CGD)	100 Year (m CGD)	200 Year (m CGD)	500 Year (m CGD)	1000 Year (m CGD)
January	2.33	2.54	2.64	2.76	2.89	2.99
February	2.12	2.33	2.39	2.46	2.56	2.64
March	1.97	2.15	2.23	2.32	2.43	2.54
April	1.81	1.96	2.02	2.06	2.12	2.16
May	1.78	2.04	2.13	2.17	2.26	2.35
June	1.85	2.04	2.11	2.17	2.29	2.40
July	1.85	2.04	2.11	2.17	2.29	2.40
August	1.46	1.86	1.95	2.02	2.12	2.21
September	1.73	1.92	1.97	2.04	2.15	2.28
October	1.89	2.09	2.16	2.24	2.35	2.44
November	2.20	2.39	2.46	2.53	2.63	2.72
December	2.40	2.59	2.64	2.71	2.80	2.90

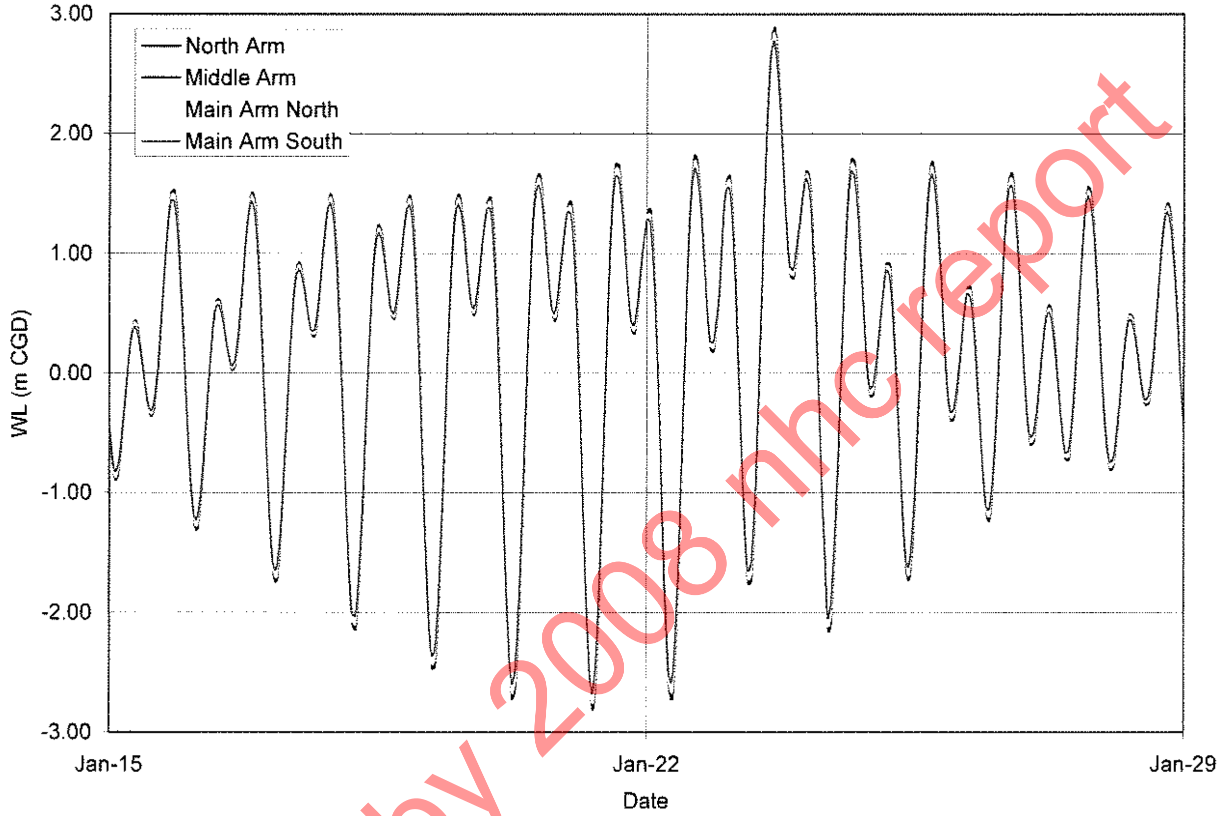
Table 15: Point Atkinson Statistics based on EST Analysis (95%CI) (excluding impact of Global Climate Change)

Month	10 Year (m CGD)	50 Year (m CGD)	100 Year (m CGD)	200 Year (m CGD)	500 Year (m CGD)	1000 Year (m CGD)
January	2.35	2.60	2.74	2.89	3.07	3.23
February	2.15	2.37	2.45	2.55	2.68	2.81
March	2.00	2.20	2.30	2.43	2.61	2.84
April	1.83	1.99	2.07	2.11	2.20	2.27
May	1.83	2.14	2.17	2.24	2.40	2.58
June	1.88	2.07	2.17	2.26	2.46	2.67
July	1.88	2.07	2.17	2.26	2.46	2.67
August	1.80	1.93	2.02	2.10	2.27	2.46
September	1.76	1.95	2.03	2.12	2.33	2.60
October	1.94	2.14	2.23	2.33	2.49	2.66
November	2.22	2.43	2.52	2.62	2.78	2.95
December	2.44	2.63	2.69	2.78	2.95	3.16

3.6 TIME SERIES

Figure 20 shows the water level time series corresponding to the winter design event.

Figure 20: Winter, 200-Year Return Period Design Event



4. *SUMMARY AND RECOMMENDATIONS*

This report summarizes the method and results obtained in developing downstream boundary conditions for the Fraser River Model. The final estimate of total water level at various return periods of interest are shown in Table 14 and Table 15. Figure 20 shows one example of a winter water level design event time profile with approximately a 200-year return period. Based on the analysis performed to date, it is recommended that a water level of about 2.90 m (CGD) (Table 15) should be considered for a 200-year return period event in the winter.

It is extremely important to note that the effects of **global climate change** such as **rising sea levels, increased storminess are not included** in these estimates nor are the impacts of **tsunamis** locally generated within the Strait of Georgia. Moreover, wave run-up, freeboard and settlement allowances must be added to these values prior to use in any engineering application.

4.1 FUTURE ANALYSIS REQUIREMENTS AND REFINEMENTS

- Evaluate the impact of including a "maximum possible" surge event in the EST data input dataset. This will require surge modelling in Georgia Strait.
- Evaluate the importance of including the "statistical tail" in the EST analysis. The present analysis does not include the "statistical tail".
- Estimate the accuracy of the predicted tide by calculating tidal constituents for a number of subsets of the recorded dataset.
- Estimate the impact of storm systems on the phasing of the tide in Georgia Strait. This could increase or decrease the surge used in the present analysis.
- Evaluate how storm systems (atmospheric pressure, wind speed and direction and duration) will be modified by Global Climate Change and how these changes will affect the frequency and magnitude of water level surges.
- Review the possible maximum water level magnitudes that could be generated by a landslide (above or below water) tsunami in Georgia Strait.

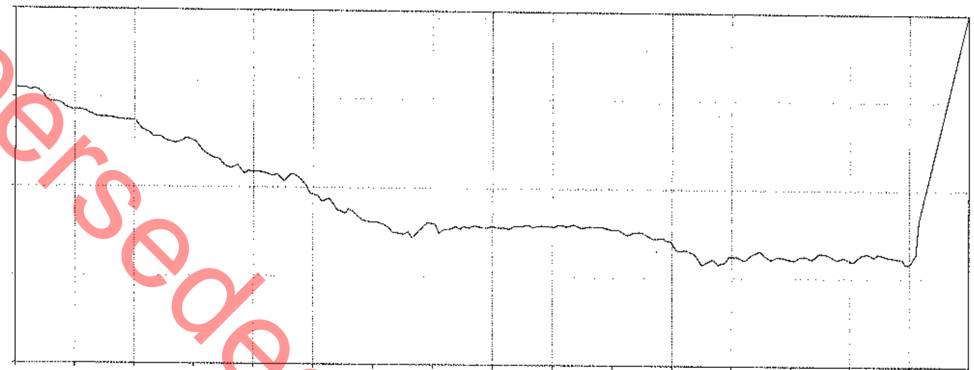
5. REFERENCES

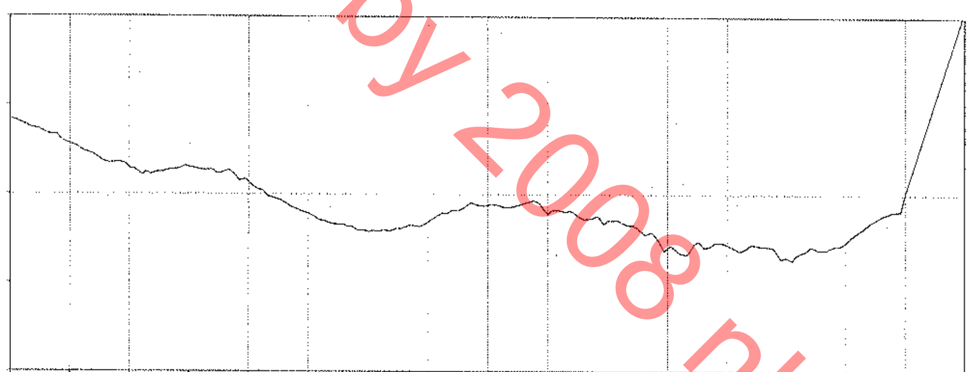
Liu, Joan C.H., 2004. The Direct Joint Probability Method for Estimating Extreme Sea Levels. University of British Columbia, Civil Engineering. M. Sc. Thesis. 180 pp.

Neil, Laurie, 2001. Applying Results from the CCAF Storm Surge/Coastal Erosion Project along the BC Coast. Meteorological Service of Canada (Pacific and Yukon Region Internal eReport 2001-03x

Scheffner, N.W., et al., 1999. Use and Application of the Empirical Simulation Technique: User's Guide. US Army Corps of Engineers, Engineering Research and Development Center Technical Report CHL-99-21. December. 194 p.

APPENDIX F
HISTORIC RIVER CROSS-SECTIONAL CHANGES





NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND
Historic Cross Section (1946 or 1951)
2005 Annual Survey

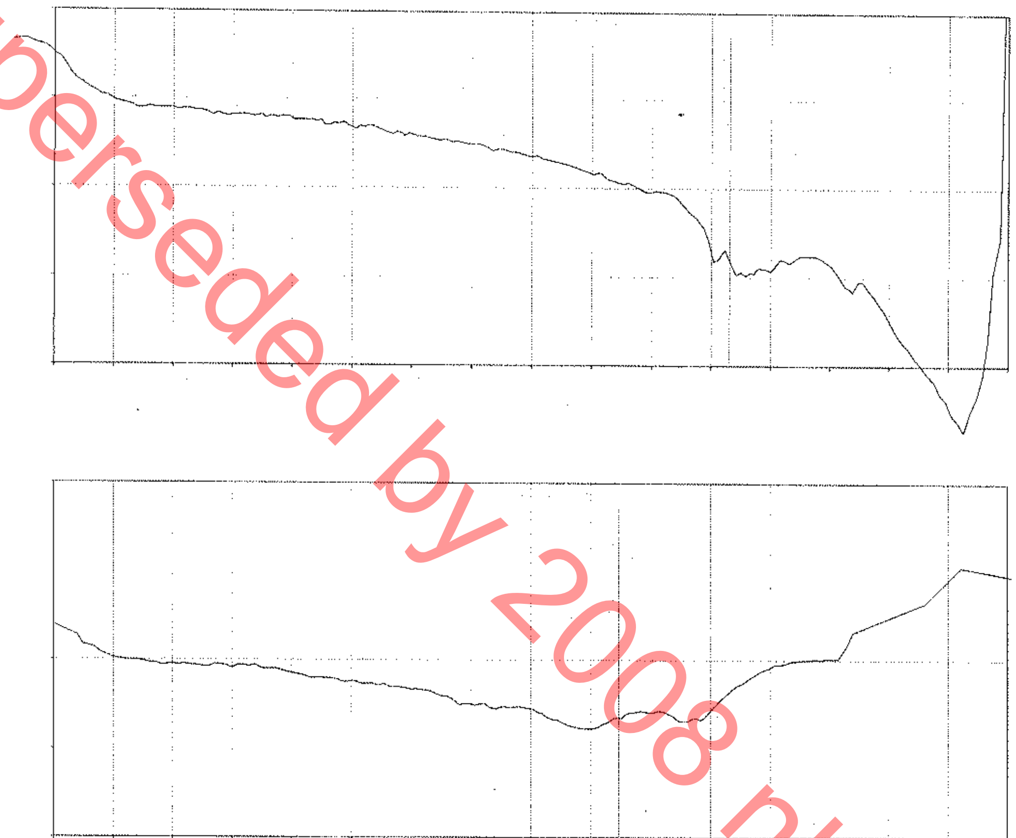
Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-620	REV NO. 0	19 Jun 06

northwest hydraulic consultants

FIGURE F-1



NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND

Historic Cross Section (1946 or 1951)
2005 Annual Survey

Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-621 | REV. NO. 0 | 19 Jun 06

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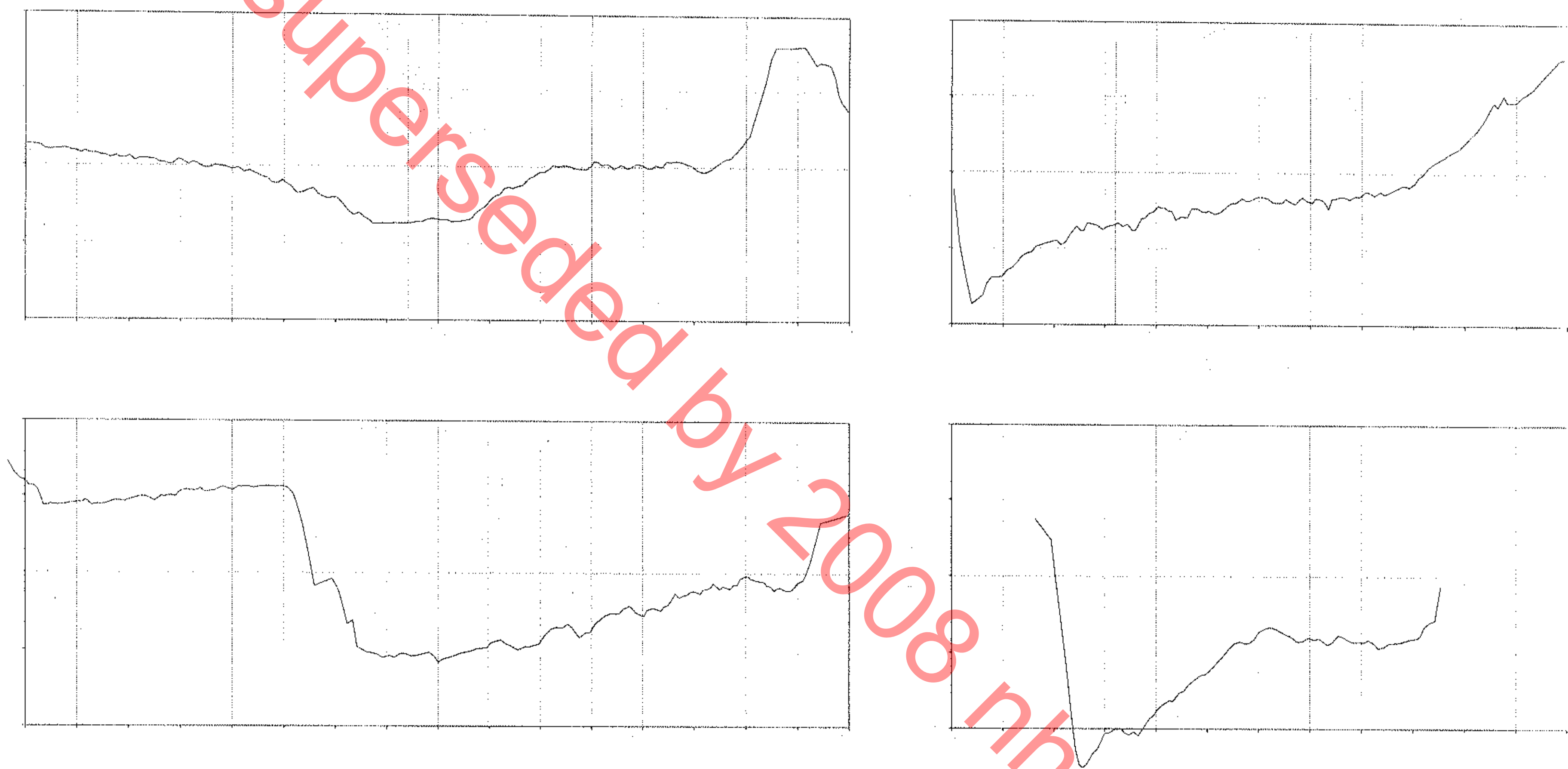
FIGURE F-2



NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND
Historic Cross Section (1946 or 1951)
2005 Annual Survey

Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-622	REV. NO. 0	19 Jun 06

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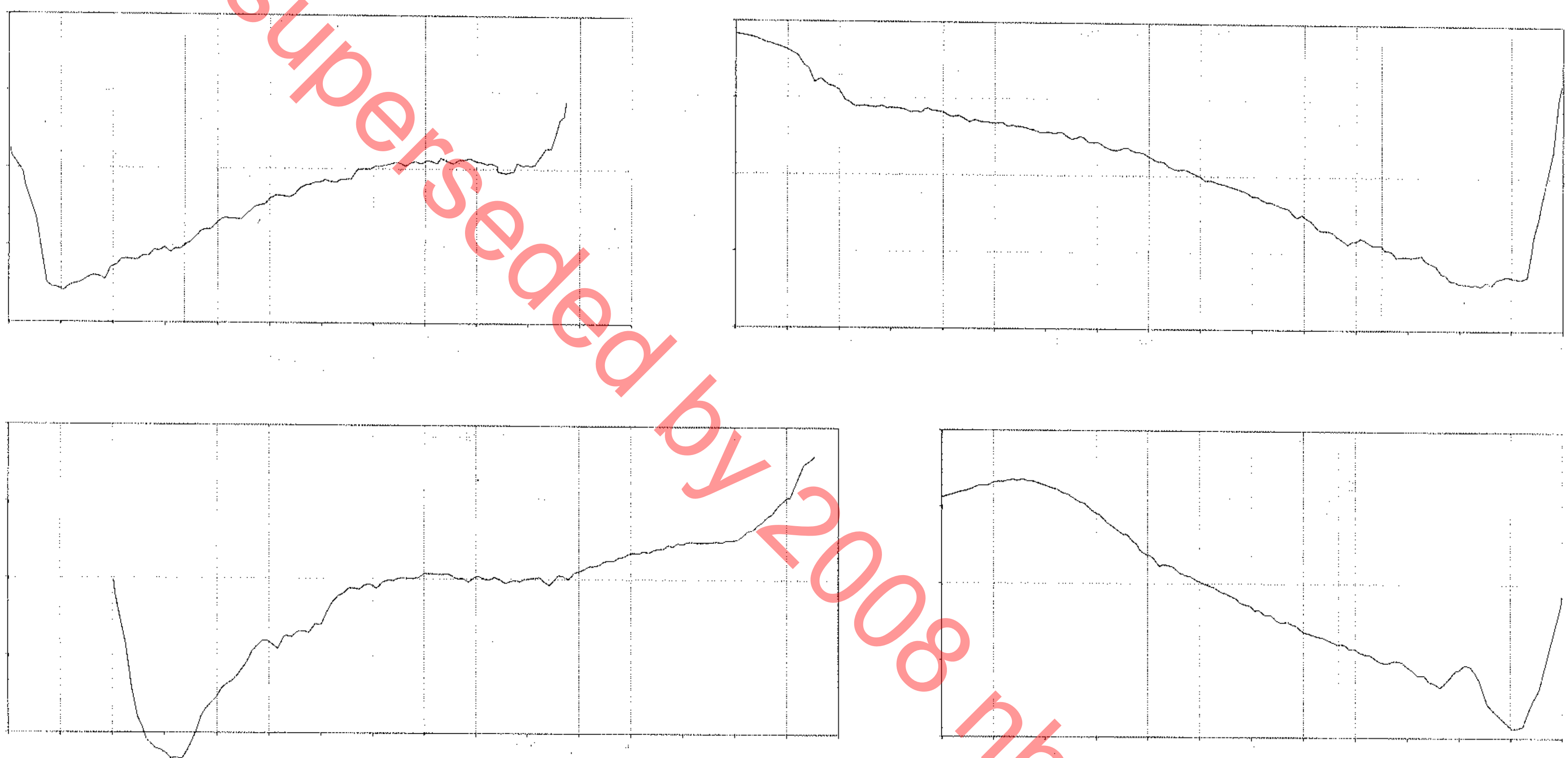
FIGURE F-3



NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND

Historic Cross Section (1946 or 1951)
2005 Annual Survey

Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-623 | REV. NO. 0 | 19 Jun 06

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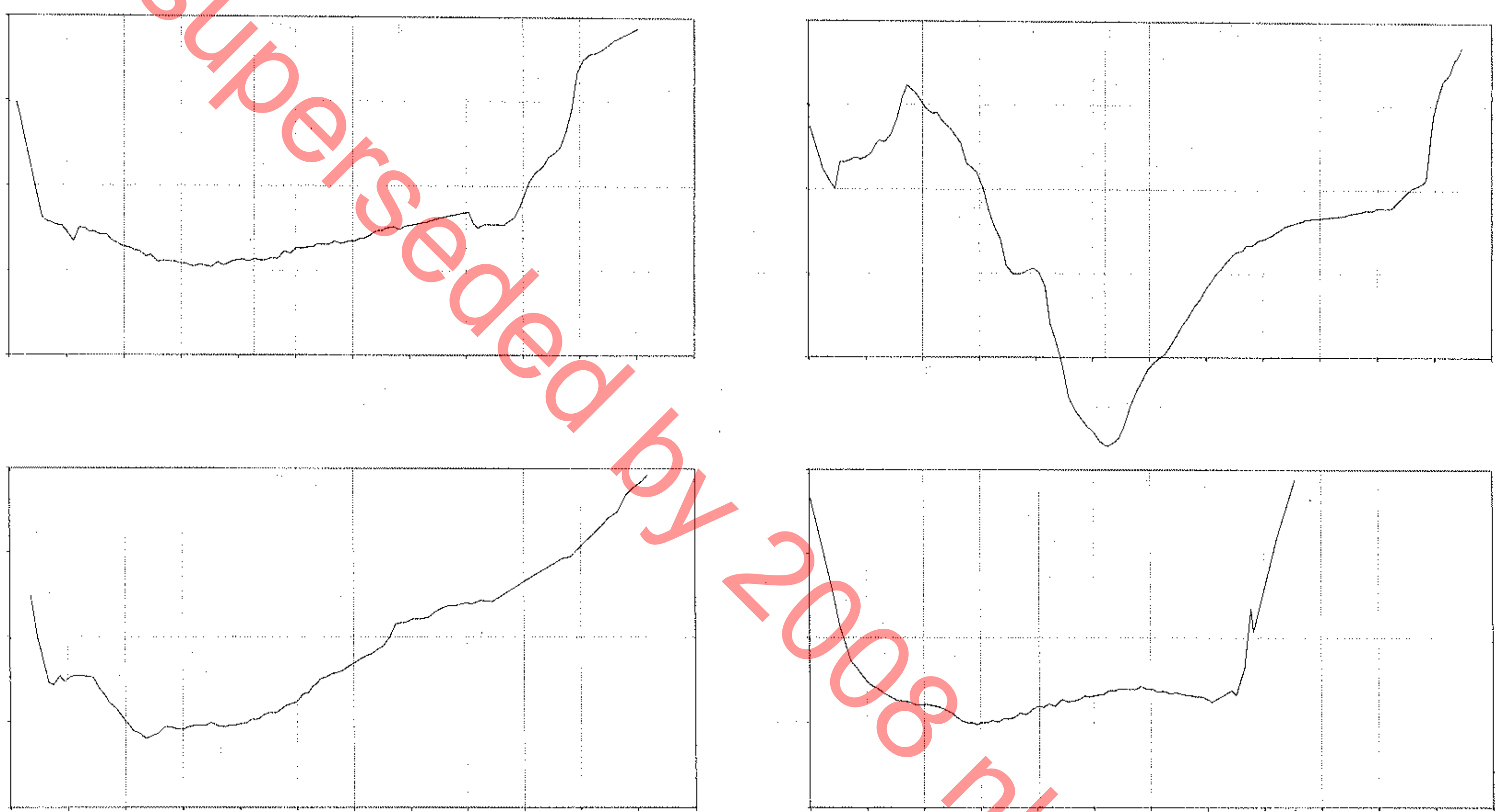
FIGURE F-4



NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND

Historic Cross Section (1946 or 1951)
2005 Annual Survey

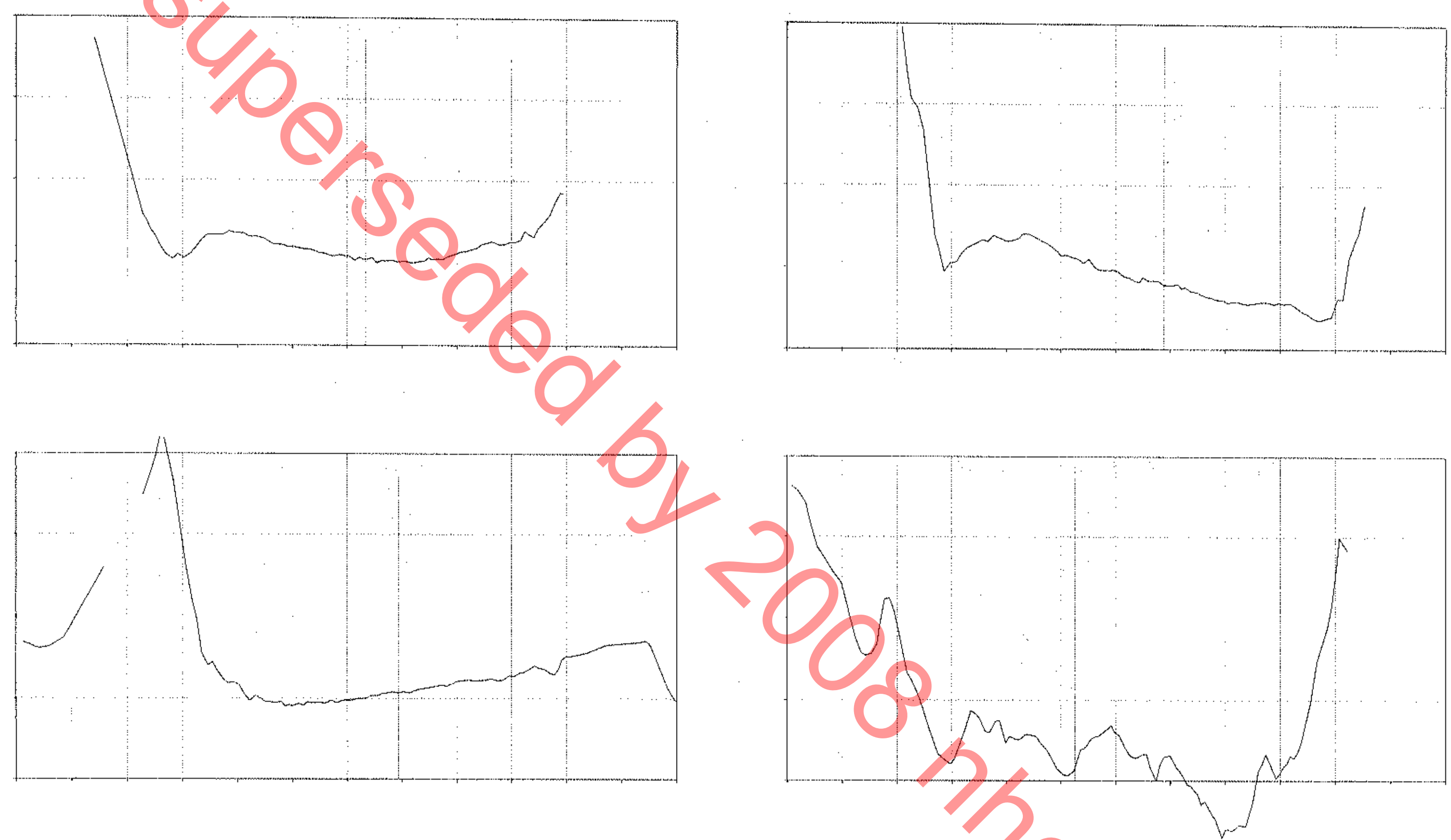
Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-624	REV. NO.	0	19 Jun 06	

northwest hydraulic consultants

FIGURE F-5



NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND
Historic Cross Section (1946 or 1951)
2005 Annual Survey

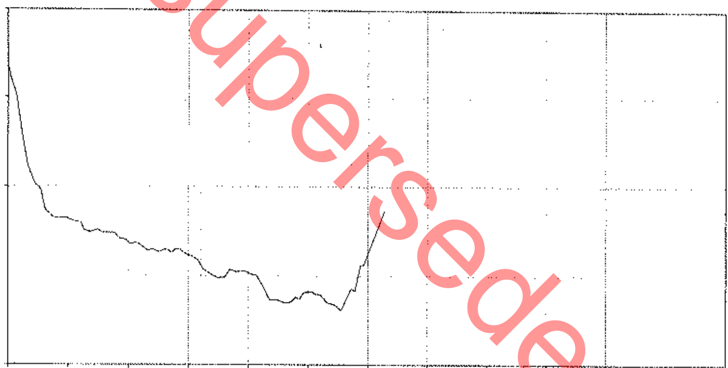
Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

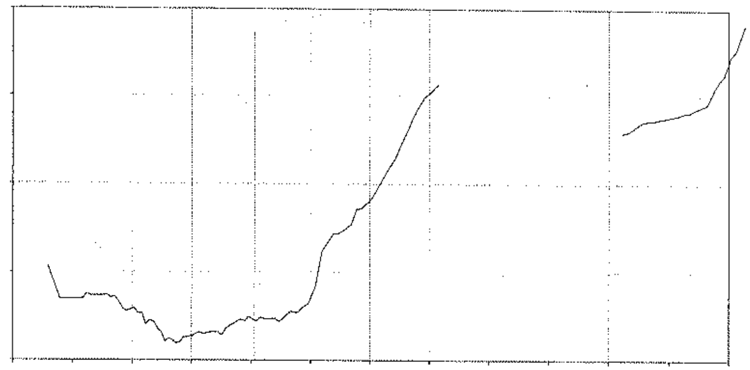
Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

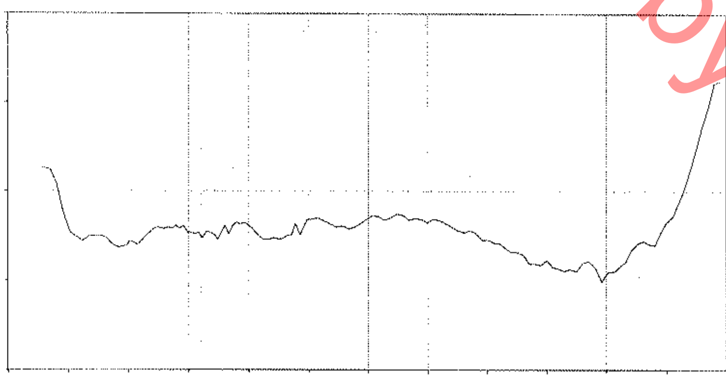
4325-625	REV NO 0	19 Jun 06

northwest hydraulic consultants

FIGURE F-6







NOTES
1) CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2) CROSS SECTIONS ONLY REPRESENT SURVEYED PORTION OF THE CHANNEL.

LEGEND

Historic Cross Section (1946 or 1951)
2005 Annual Survey

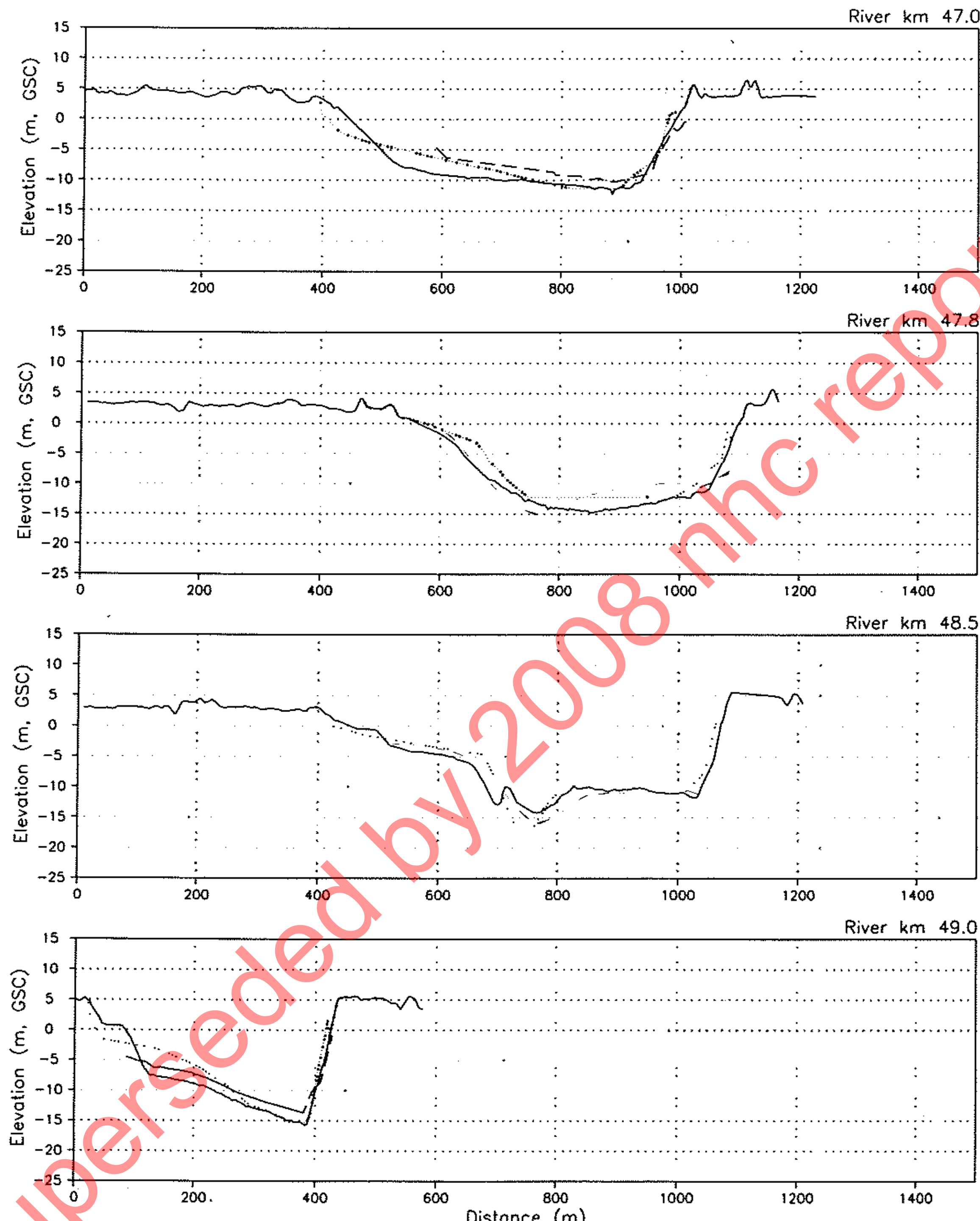
Fraser Basin Council
LOWER FRASER RIVER HYDRAULIC MODEL

Historic River Changes
Cross Section Comparisons
1946/1951 versus 2005

4325-626	REV. NO	0	19 Jun 06	

northwest hydraulic consultants

FIGURE F-7



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND

....	1950/1952 – Dominion Public Works
— — —	1991 – Hydrographic Survey of Canada
———	2005 – Public Works and Government Service Canada

Fraser Basin Council

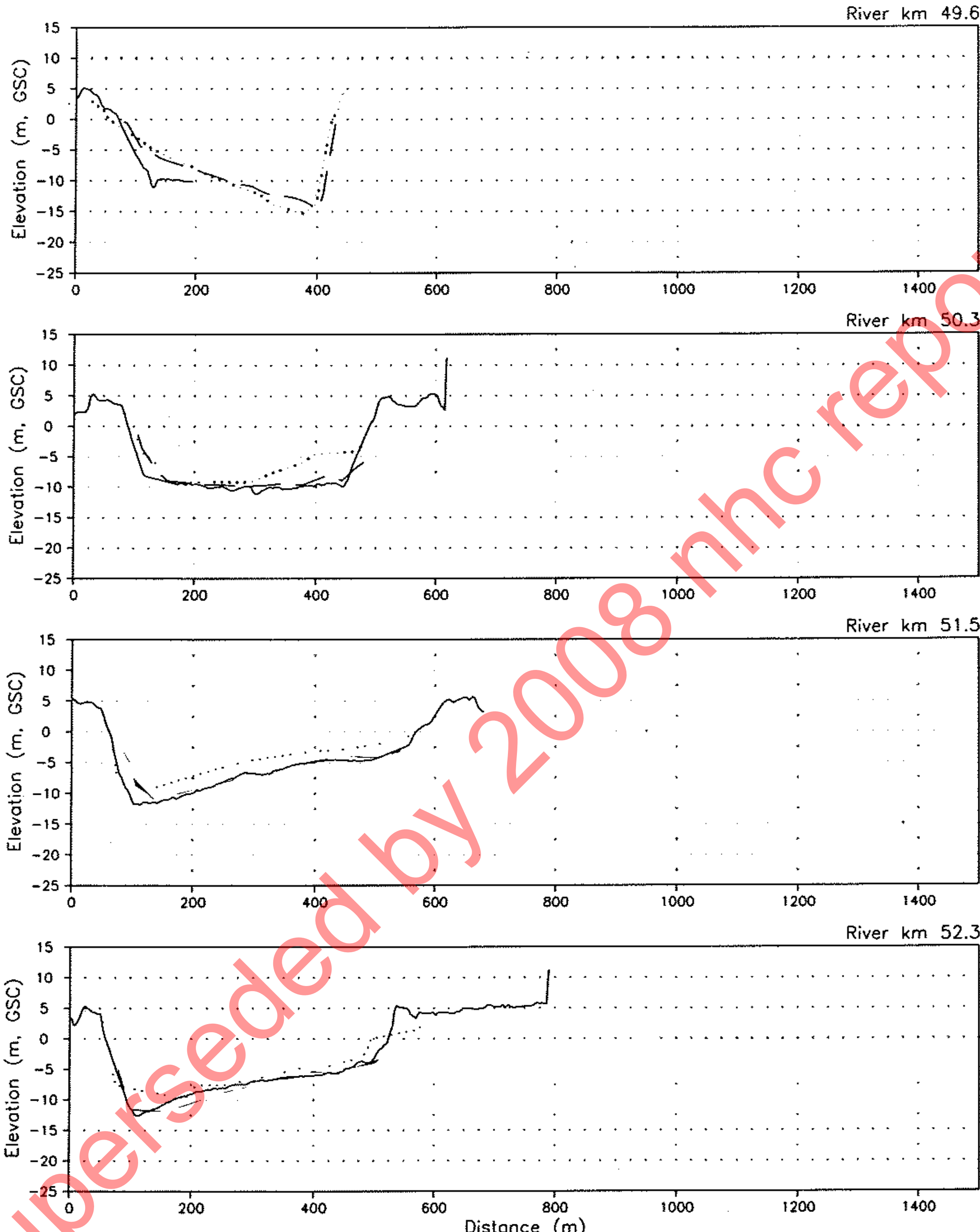
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-802	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-8



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

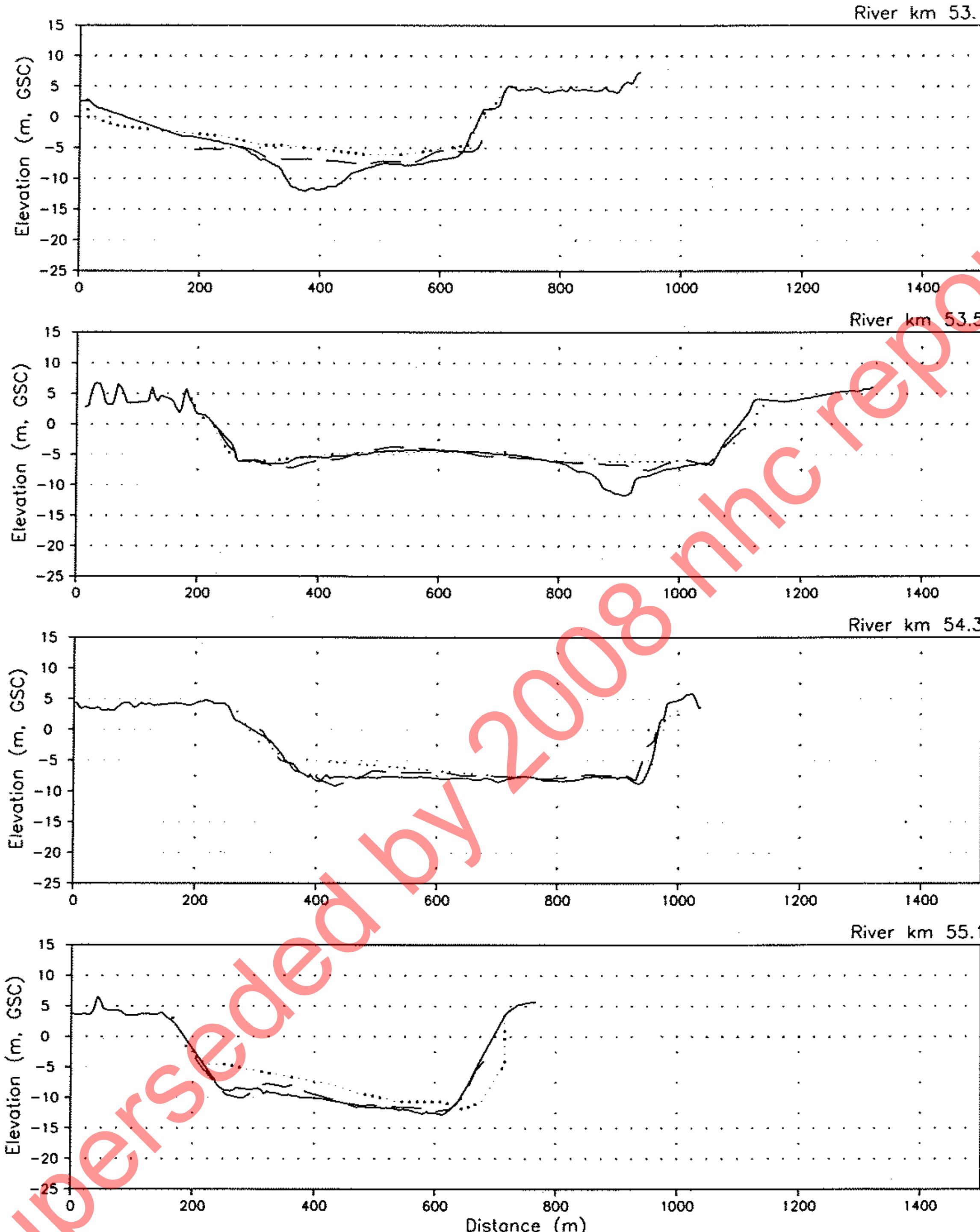
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-804	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-9



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
· · · · 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
—— 2005 – Public Works and Government Service Canada

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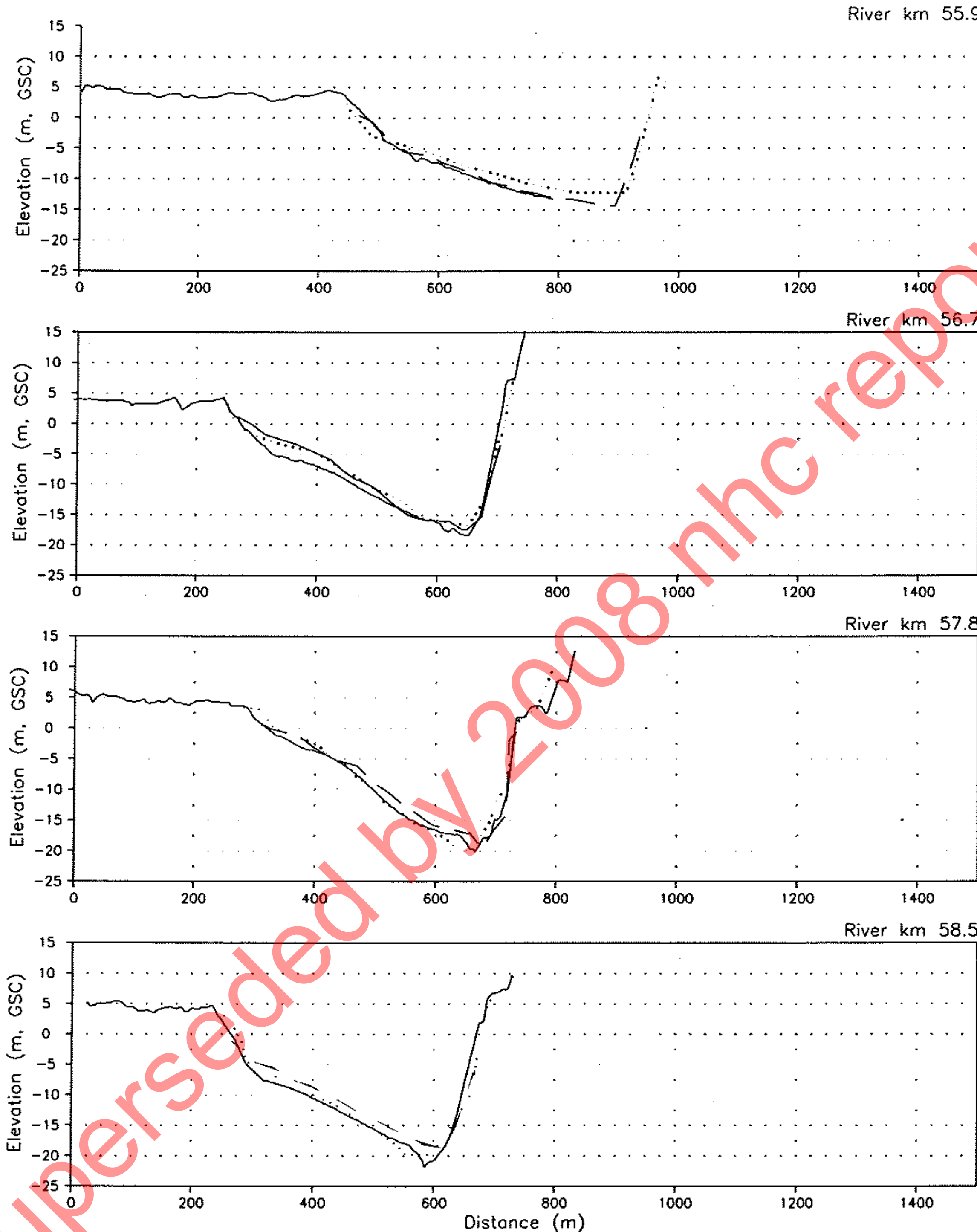
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-805	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-10

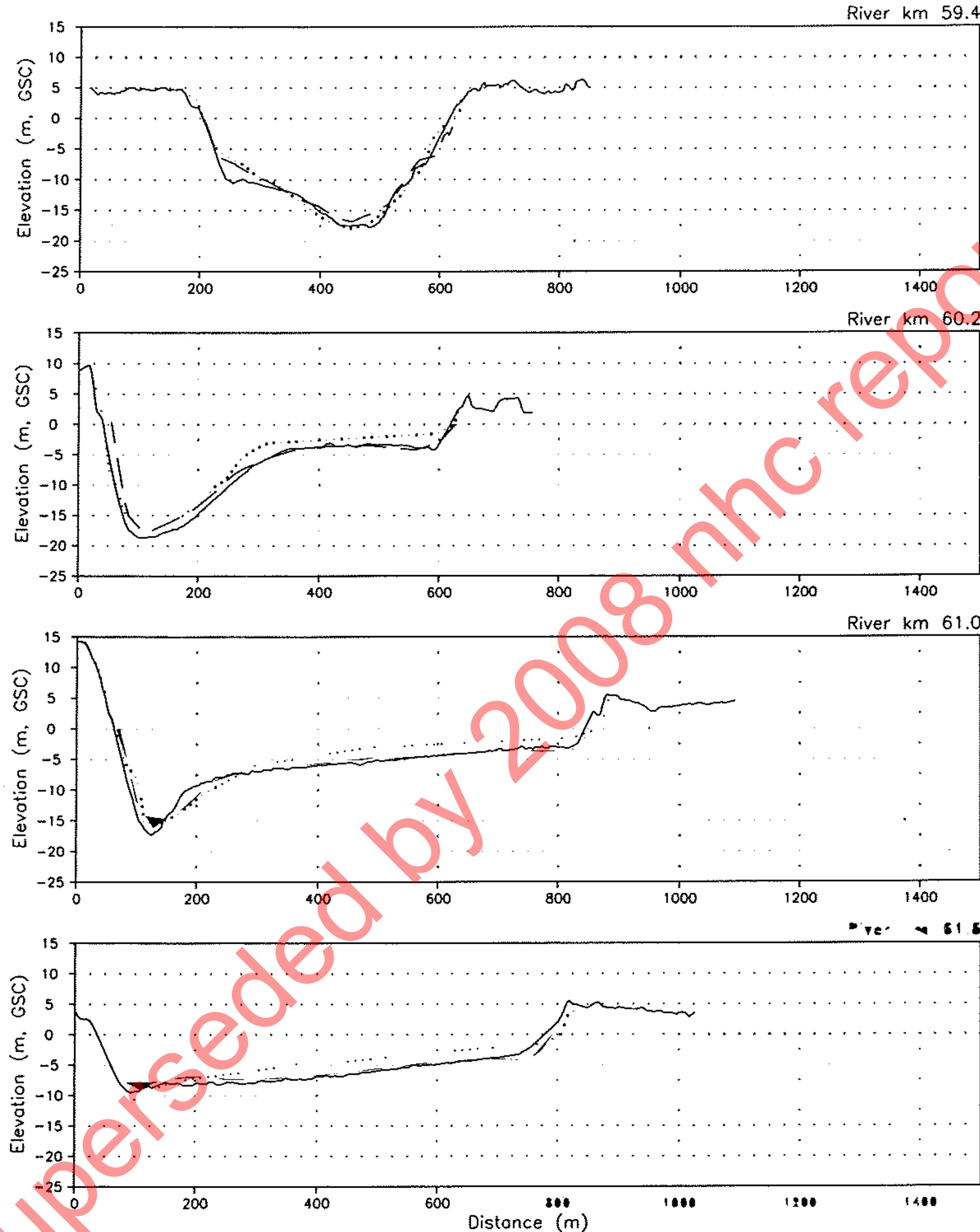


NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council		
LOWER FRASER RIVER HYDRAULIC MODEL		
1950/52, 1991 and 2005 Cross Section Comparisons River Kilometer 47 to 93		
4325-806	REV. NO. 0	31-Oct-06
northwest hydraulic consultants		

FIGURE F-11



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
· · · · 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

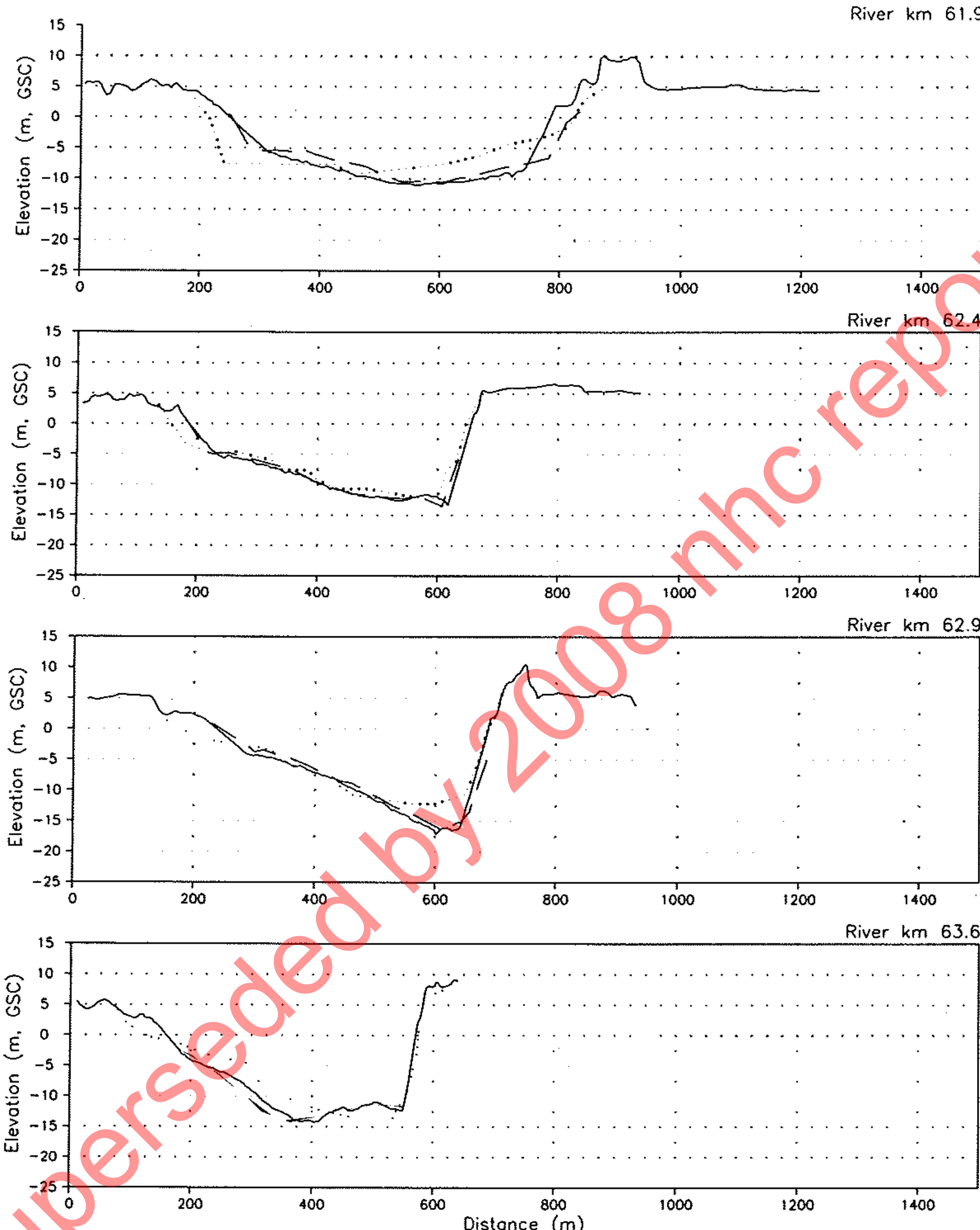
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-807	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-12



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
· · · · 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
—— 2005 – Public Works and Government Service Canada

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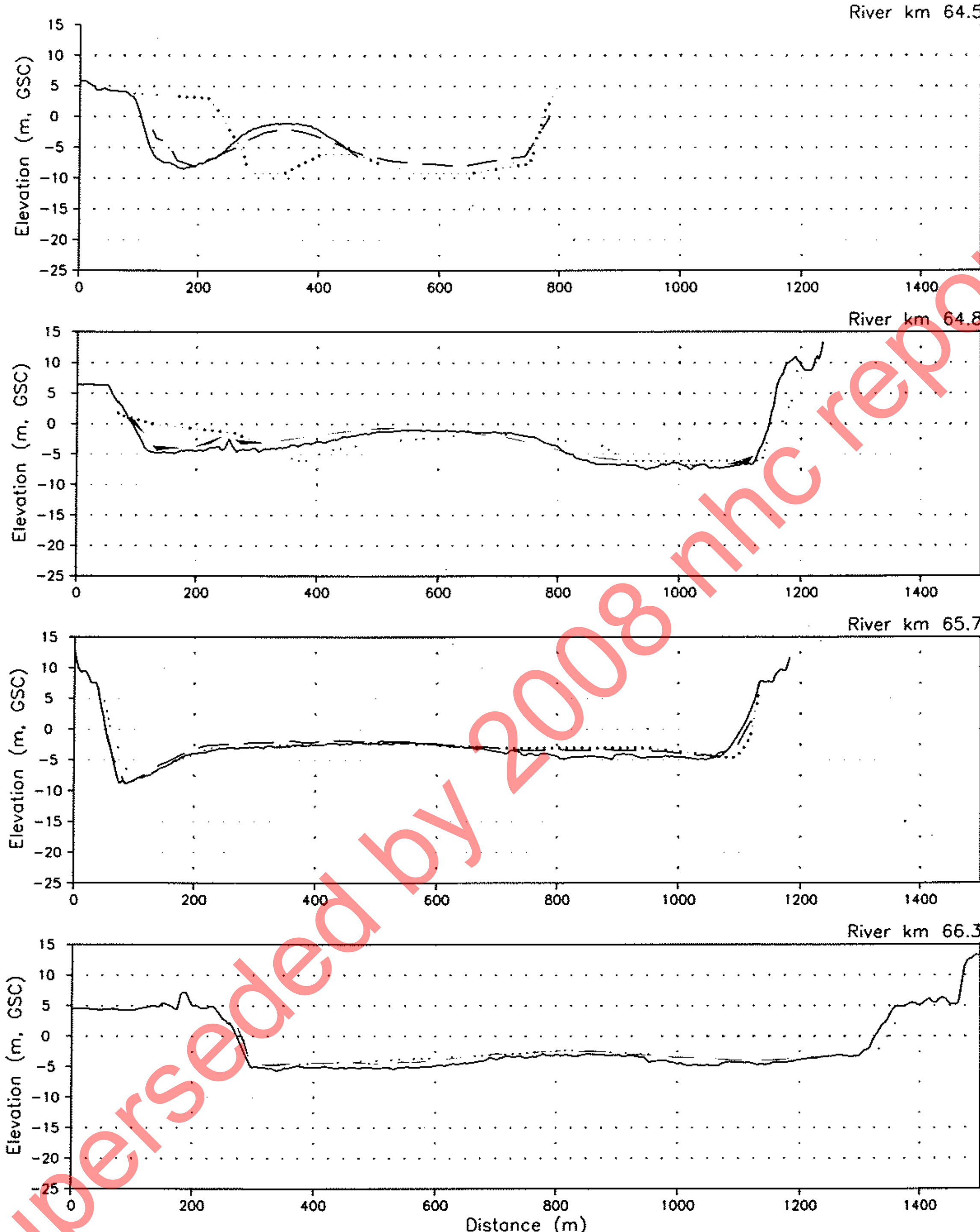
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-808	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-13



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
· · · · 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
—— 2005 – Public Works and Government Service Canada

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LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-809	REV. NO. 0	31-Oct-06

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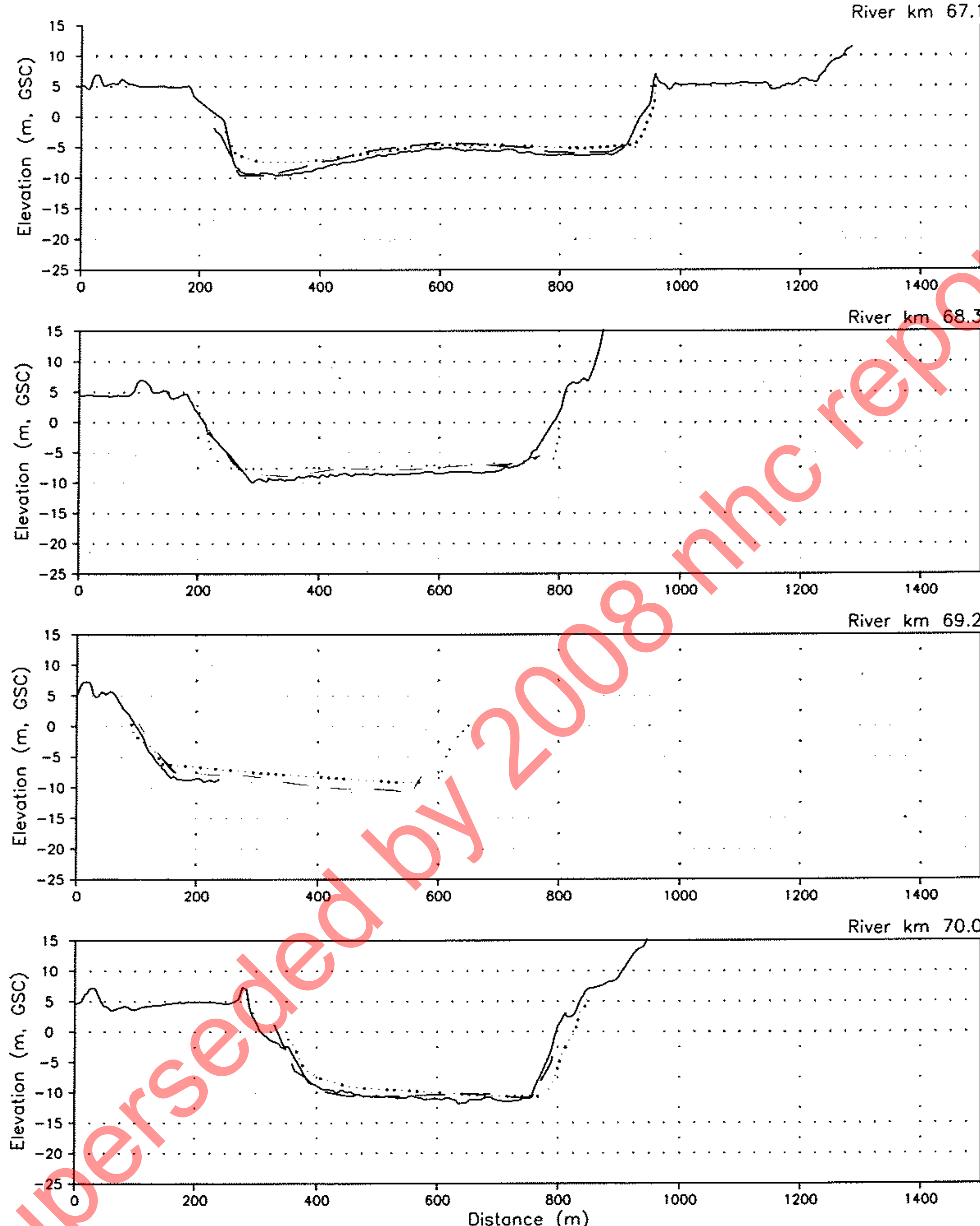
FIGURE F-14



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

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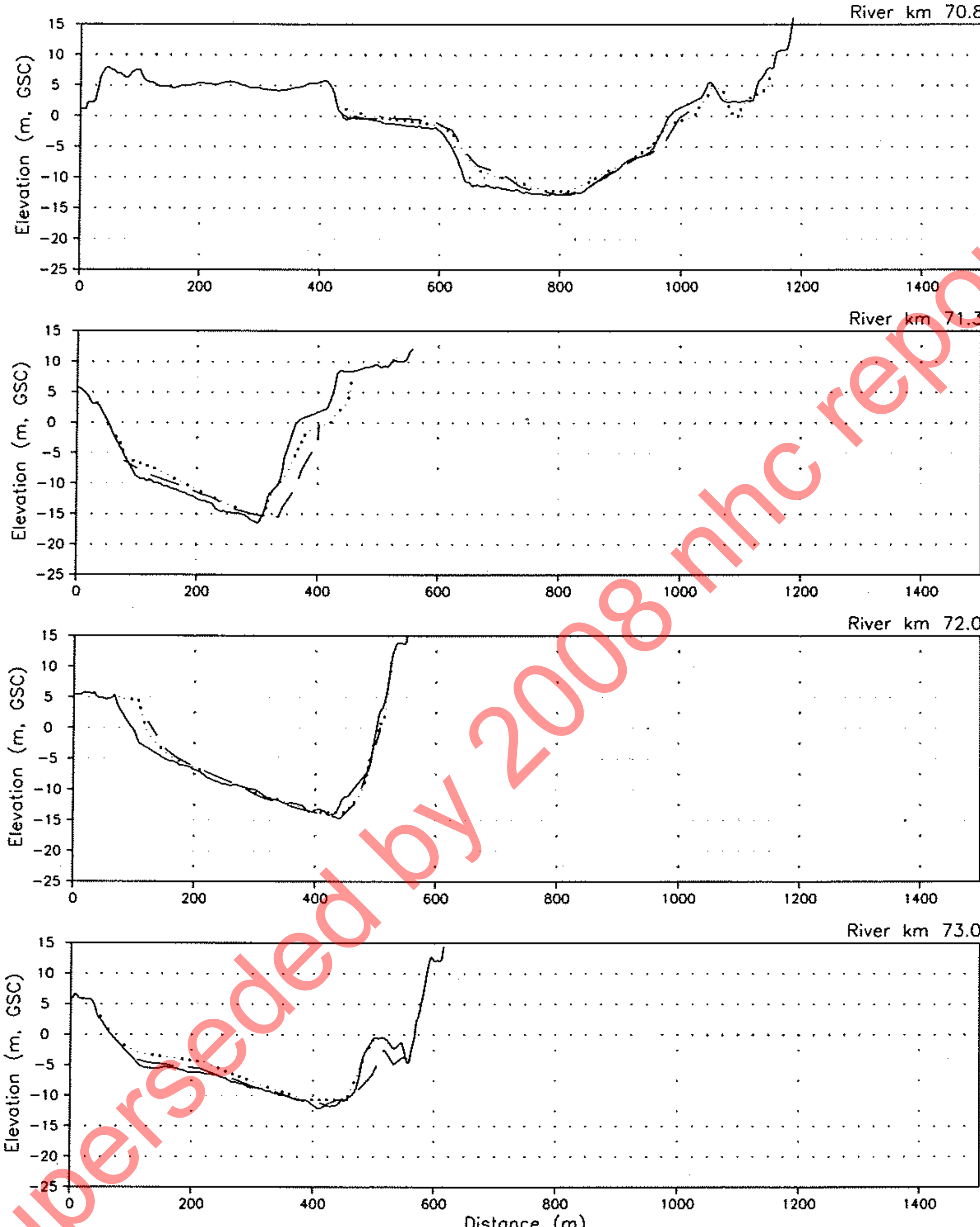
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005 Cross Section Comparisons River Kilometer 47 to 93

4325-810	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-15



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

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LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-811	REV. NO. 0	31-Oct-06

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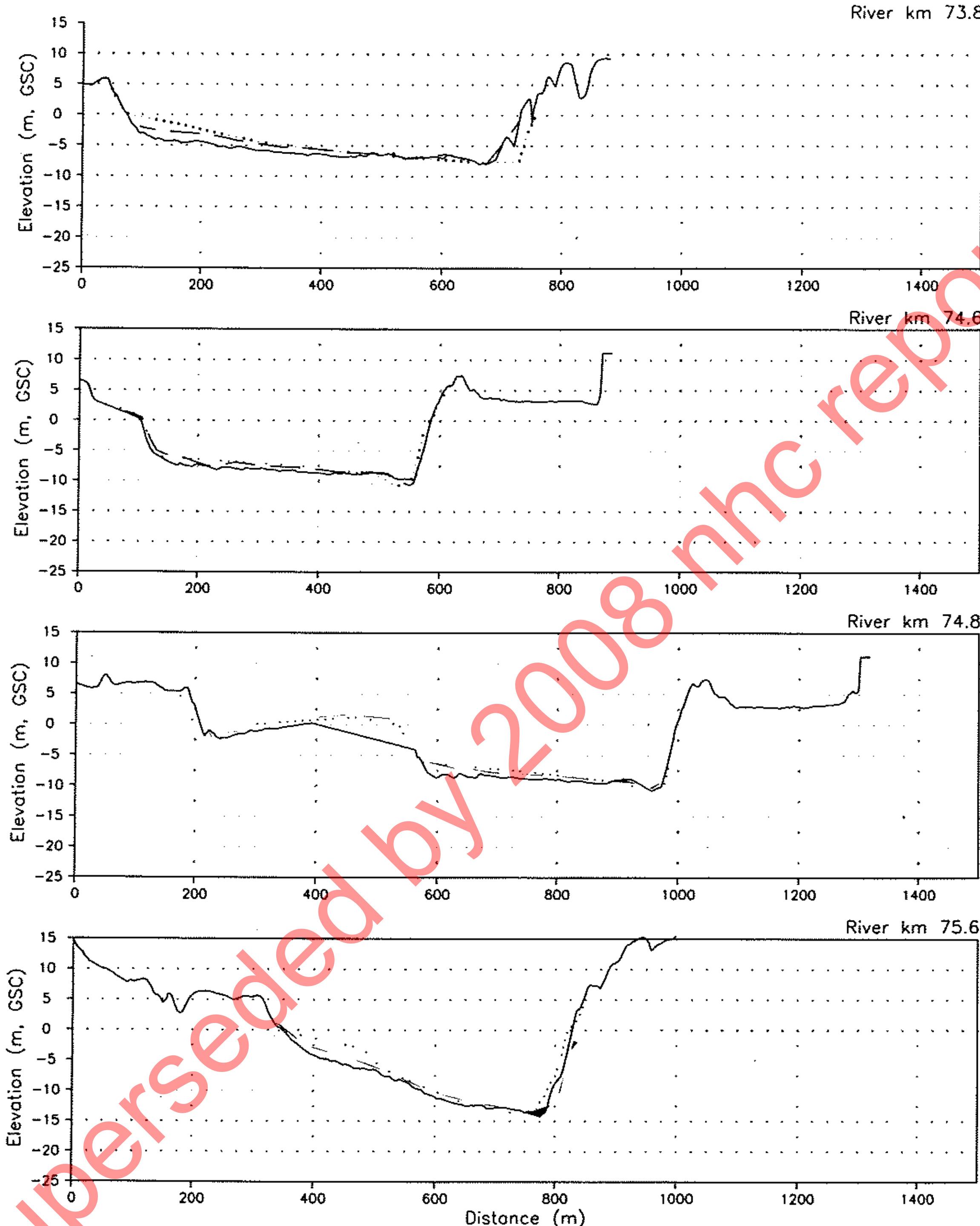
FIGURE F-16



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
· · · · 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

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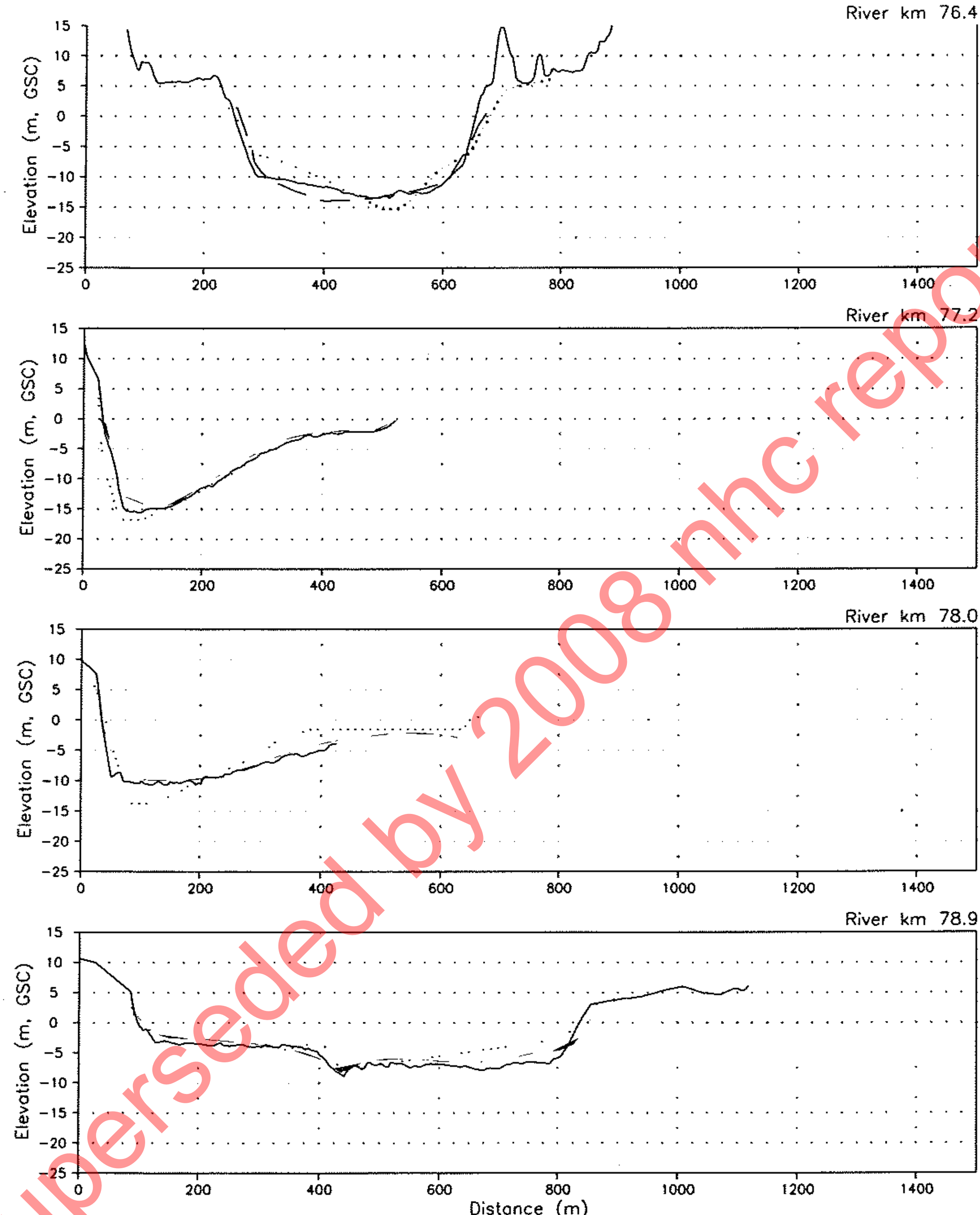
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325–812	REV. NO. 0	31–Oct–06

northwest hydraulic consultants

FIGURE F-17



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-813	REV. NO. 0	31-Oct-06

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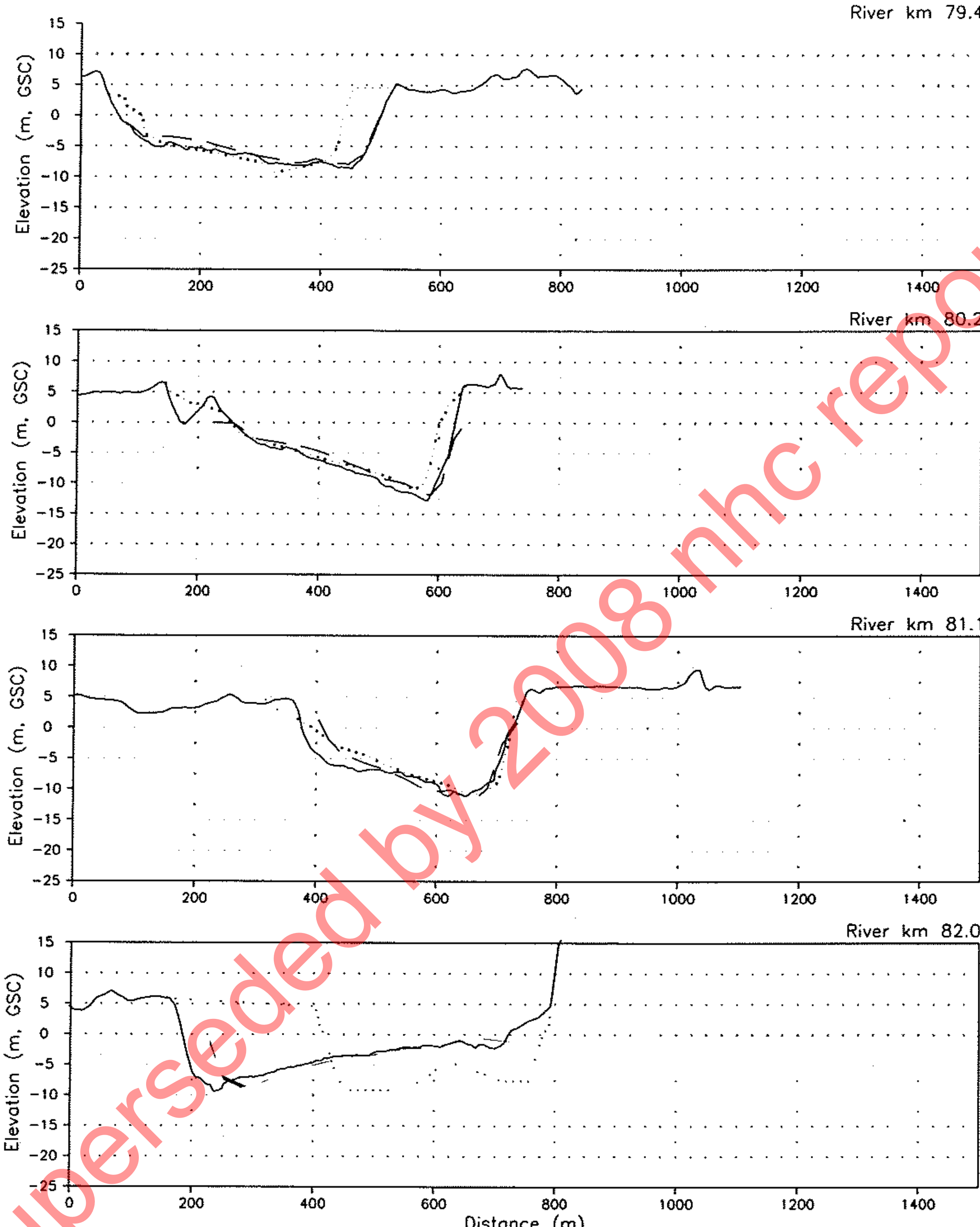
FIGURE F-18



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND

· · · ·	1950/1952 – Dominion Public Works
— —	1991 – Hydrographic Survey of Canada
——	2005 – Public Works and Government Service Canada

Fraser Basin Council

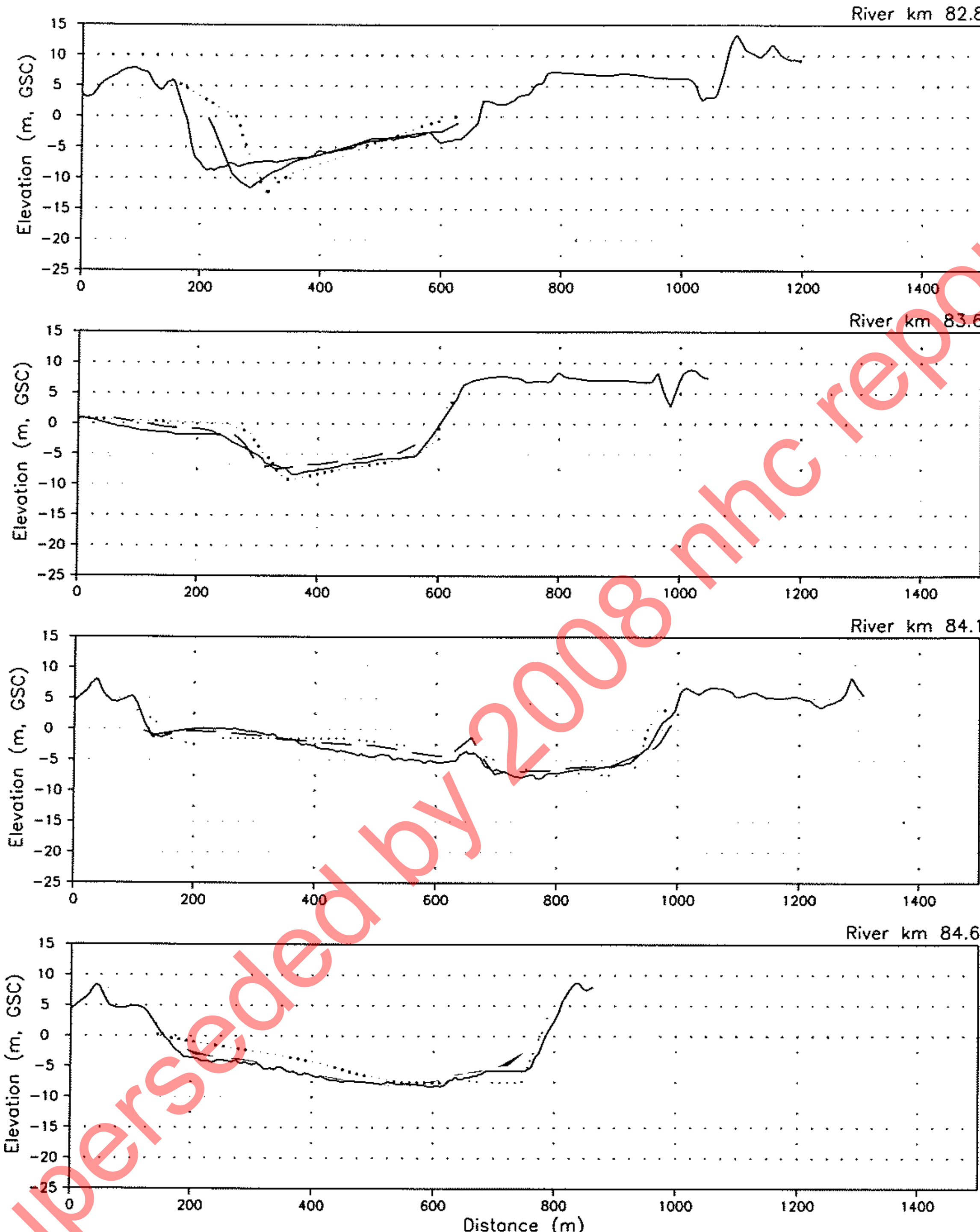
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-814	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-19



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
- · · · · 1950/1952 – Dominion Public Works
- — — 1991 – Hydrographic Survey of Canada
- ——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-815	REV. NO. 0	31-Oct-06

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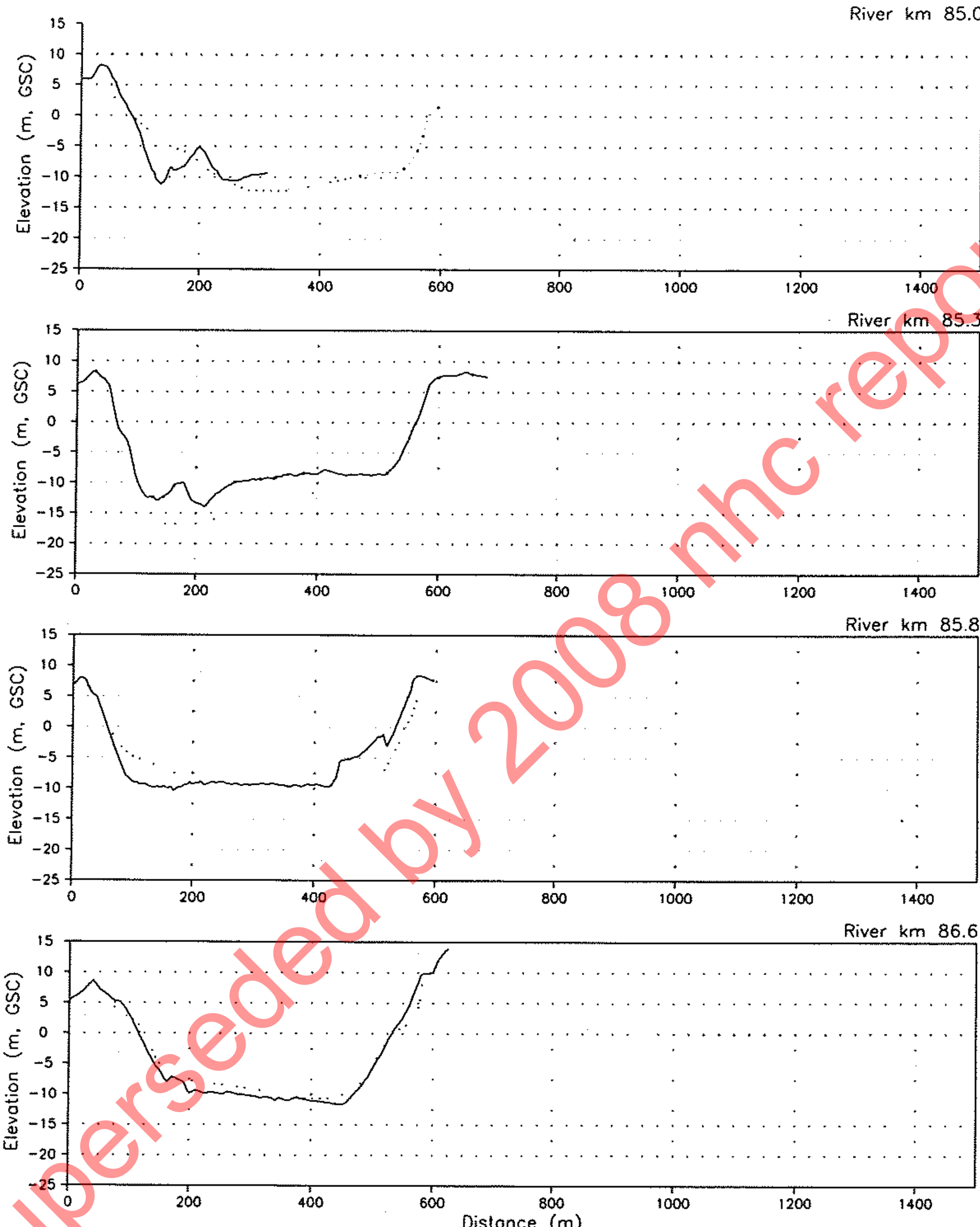
FIGURE F-20



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

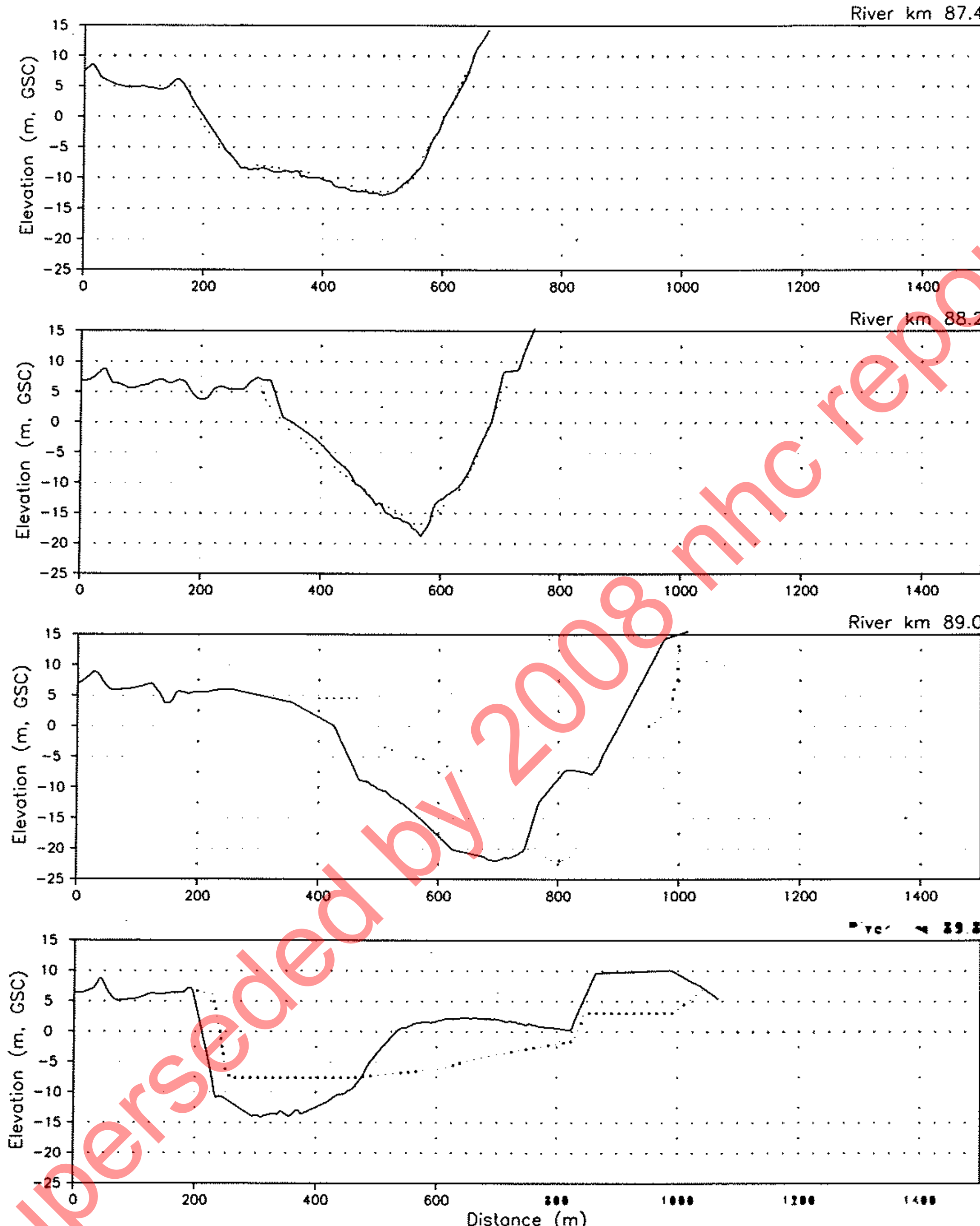
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-816	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-21



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
—— 2005 – Public Works and Government Service Canada

Fraser Basin Council

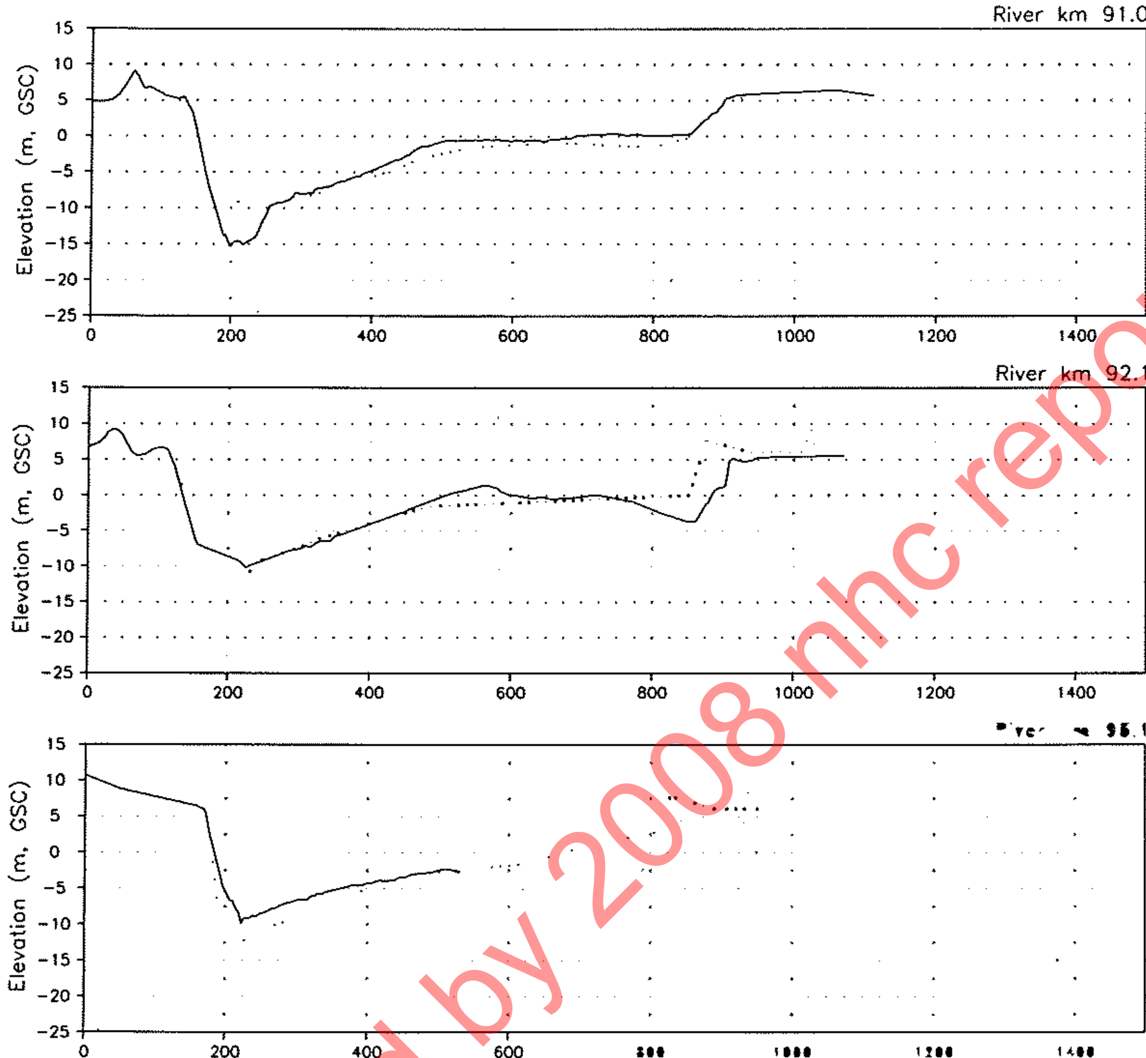
LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-817	REV. NO. 0	31-Oct-06

northwest hydraulic consultants

FIGURE F-22



NOTES:
1. CROSS SECTIONS VIEWED LOOKING DOWNSTREAM.
2. 1950/52 AND 1991 SECTIONS ARE WITHIN BANKS ONLY.
3. VERTICAL DISTORTION OF 1H:10V

LEGEND
. . . . 1950/1952 – Dominion Public Works
— — 1991 – Hydrographic Survey of Canada
——— 2005 – Public Works and Government Service Canada

Fraser Basin Council

LOWER FRASER RIVER HYDRAULIC MODEL

1950/52, 1991 and 2005
Cross Section Comparisons
River Kilometer 47 to 93

4325-818	REV. NO. 0	31-Oct-06

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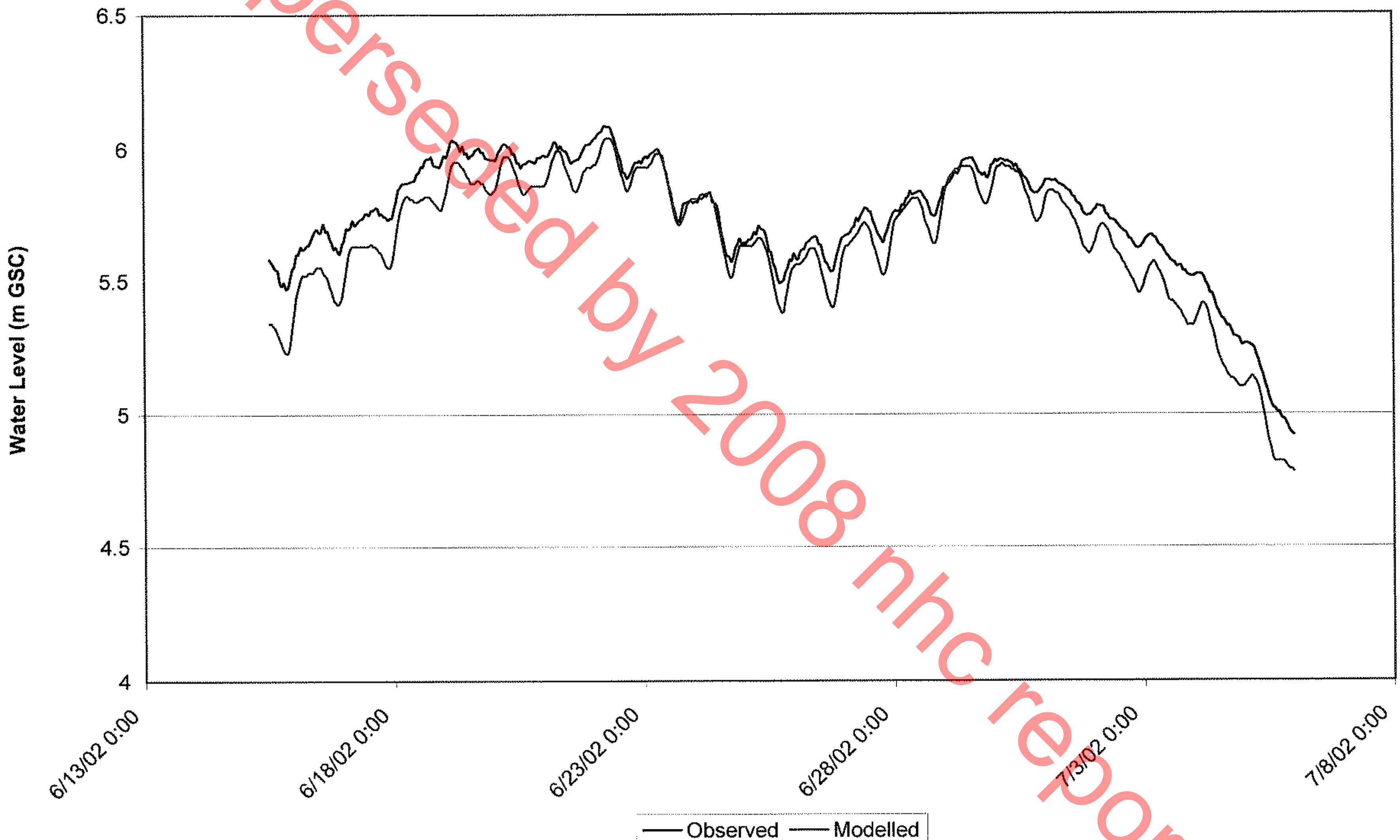
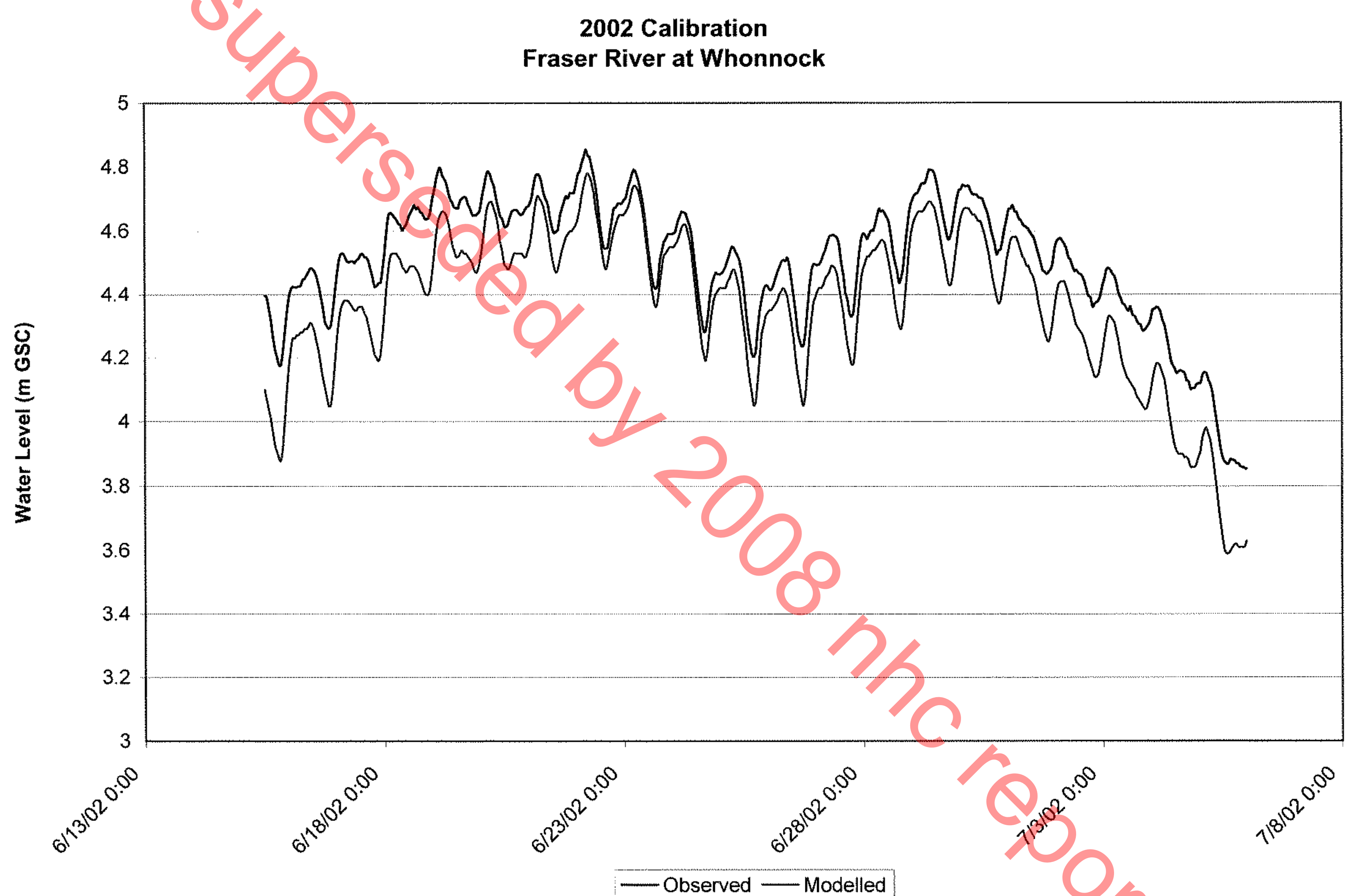
FIGURE F-23

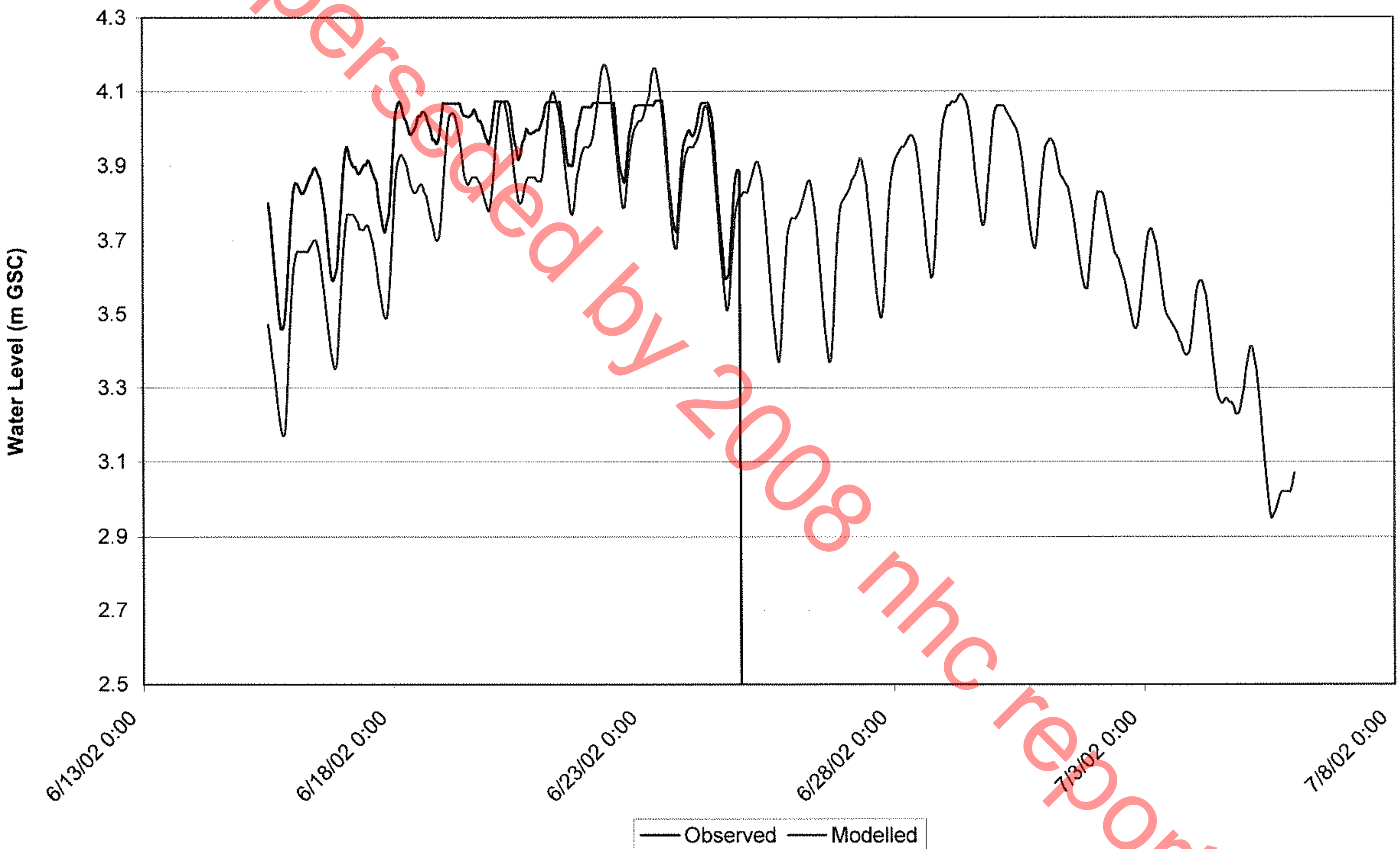
APPENDIX G
CALIBRATION/VERIFICATION PLOTS

2002 Calibration
Fraser River at Mission

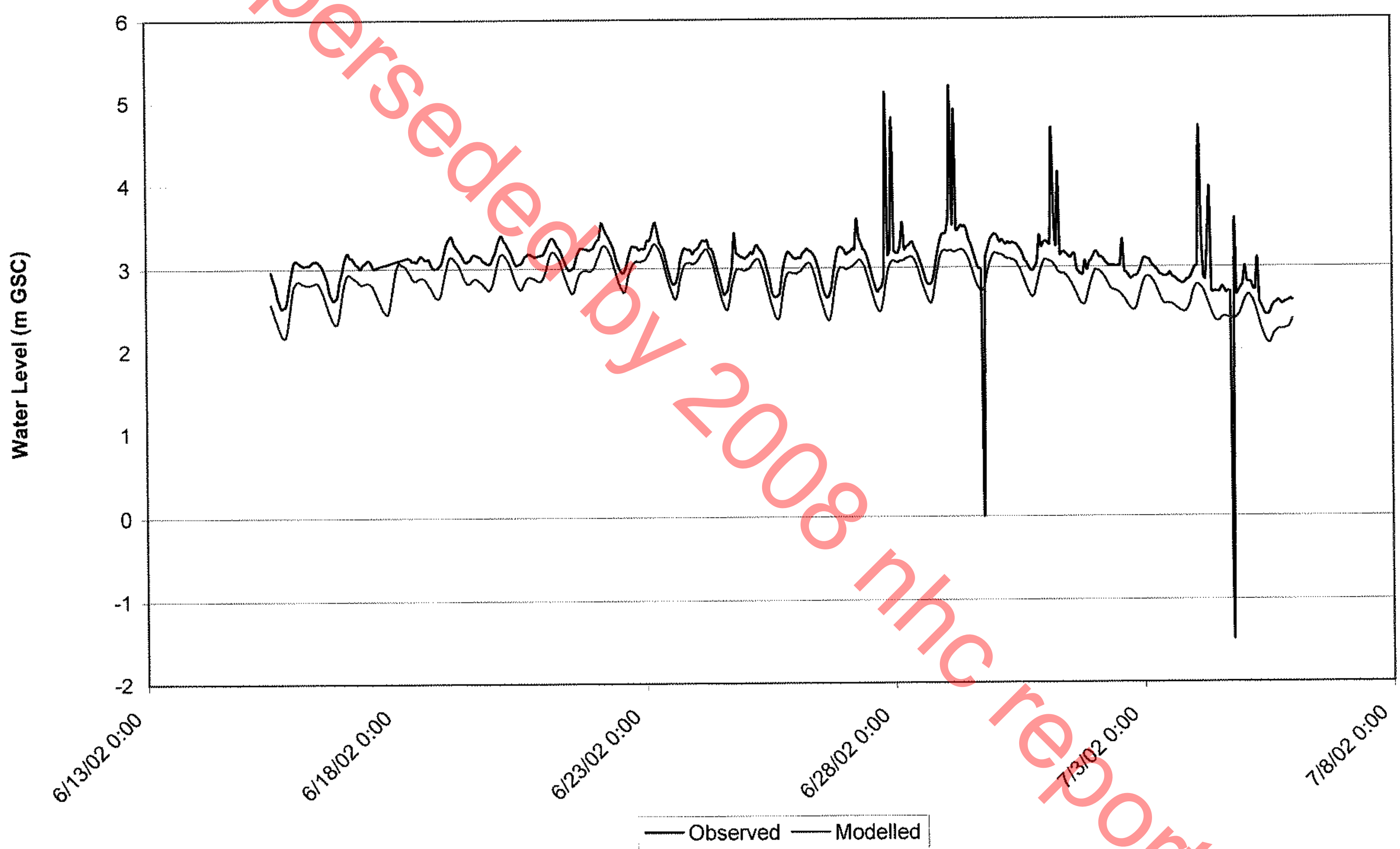


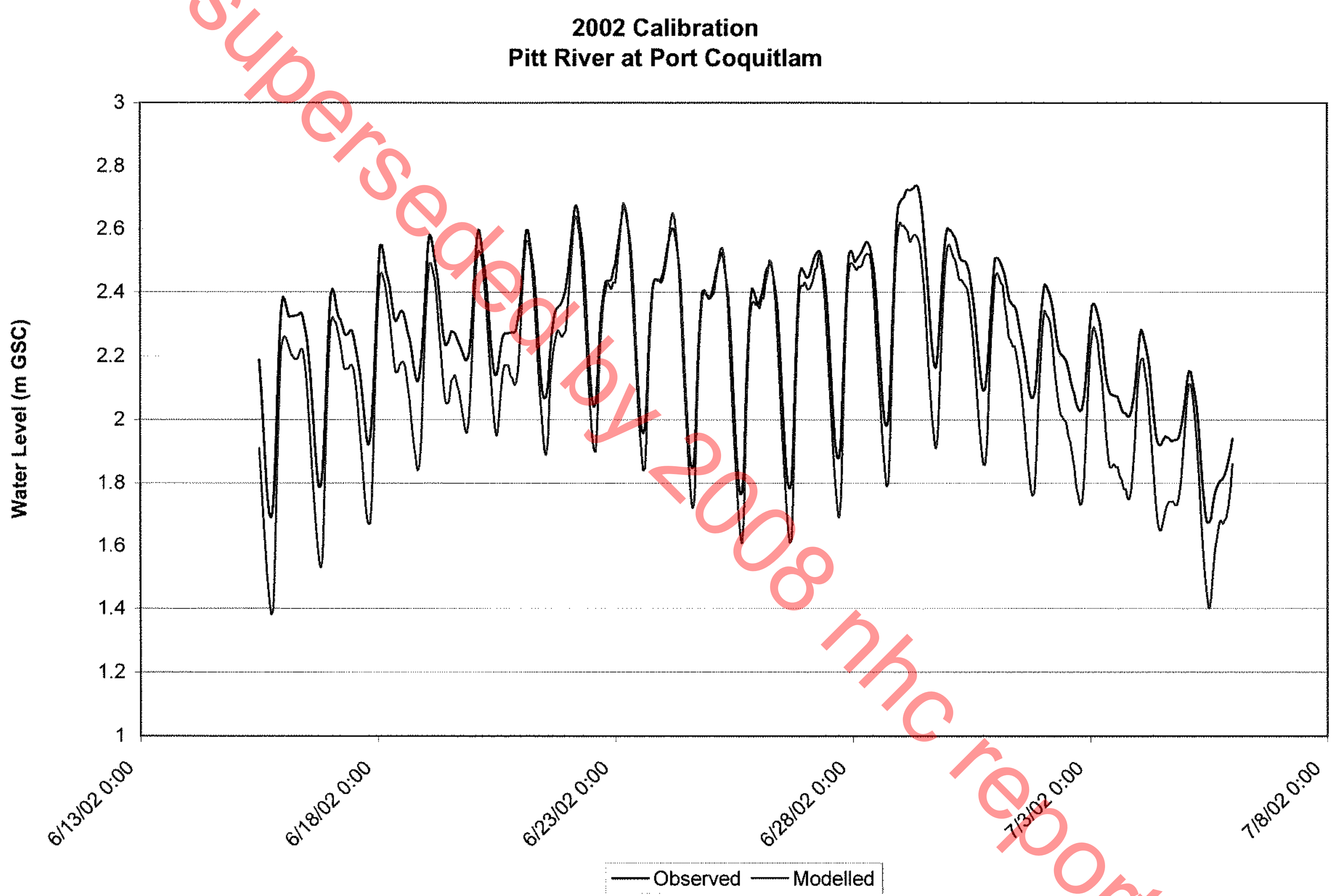

2002 Calibration
Fraser River at Whonnock
Water Level (m GSC)
5
4.8
4.6
4.4
4.2
4
3.8
3.6
3.4
3.2
3
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

2002 Calibration
Fraser River at McMillan Channel

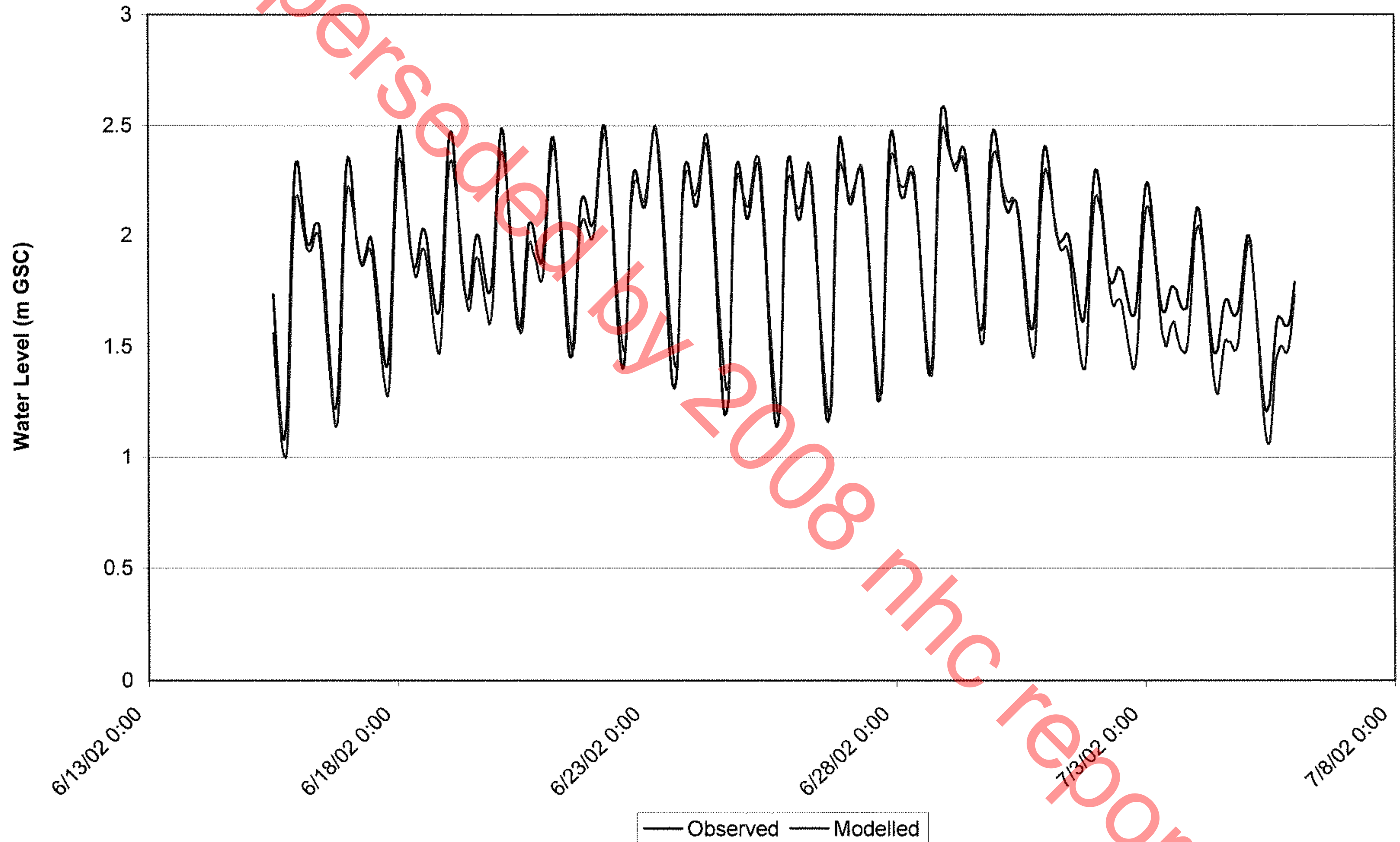


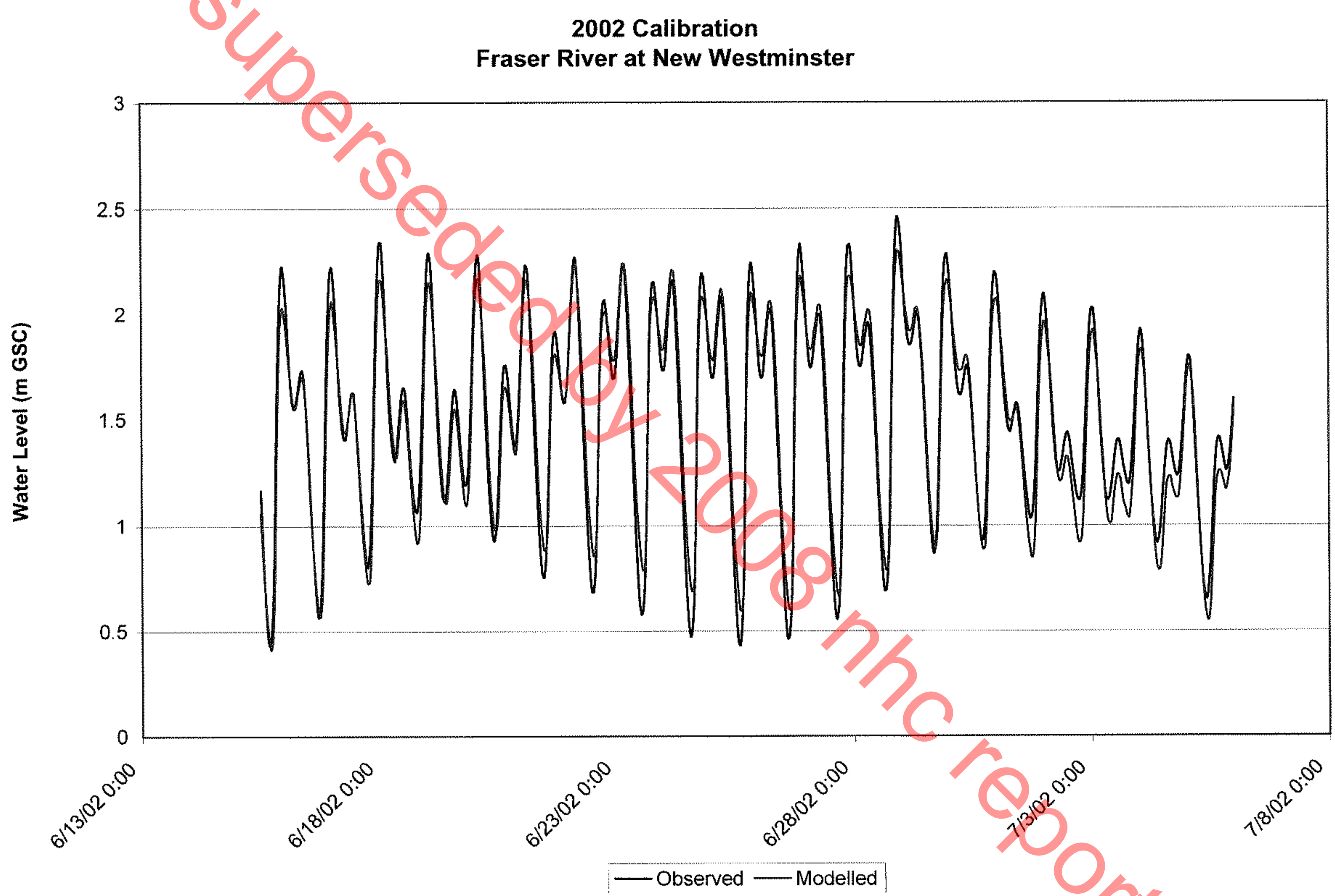
2002 Calibration
Fraser River at Barnston Channel

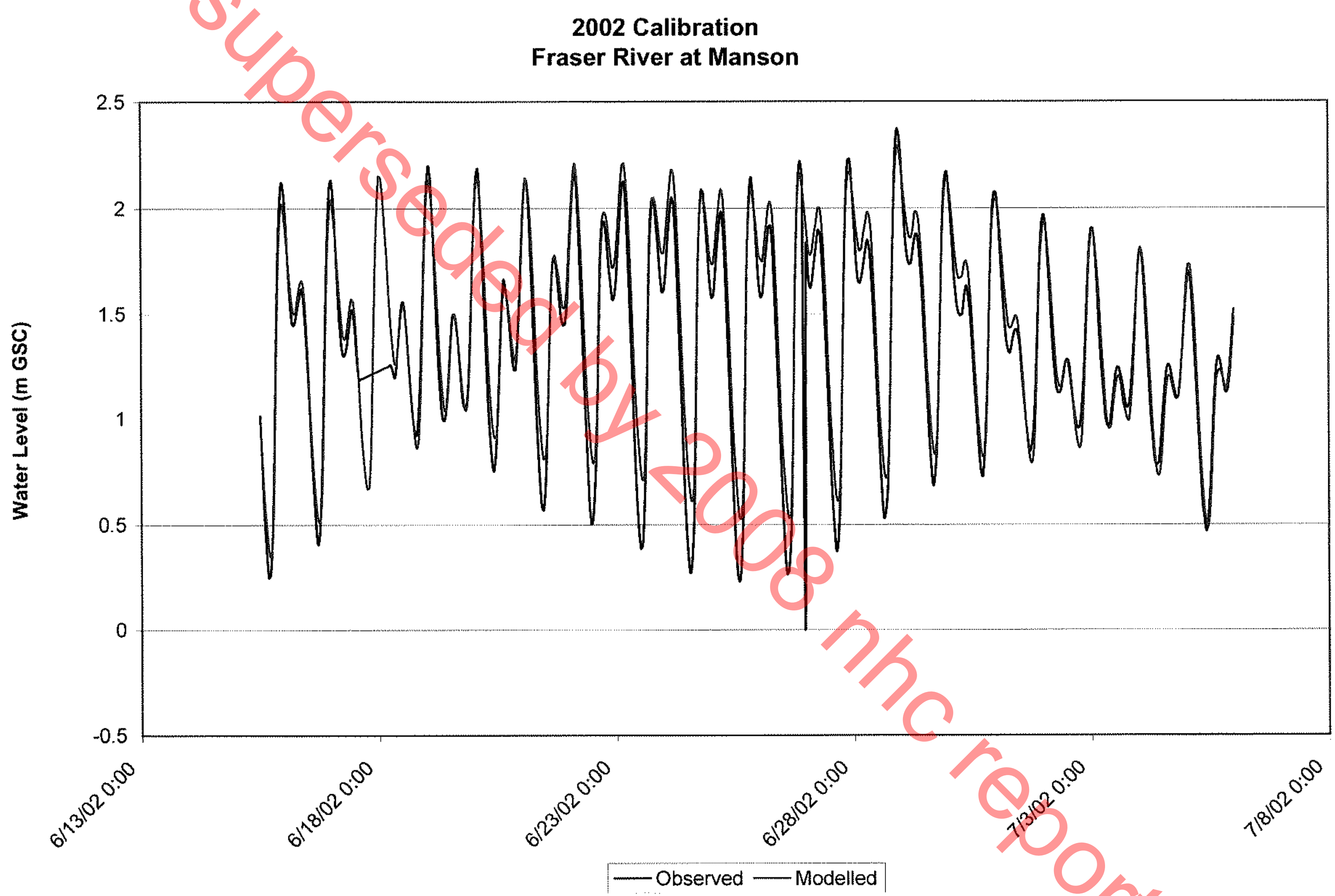



2002 Calibration
Pitt River at Port Coquitlam
Water Level (m GSC)
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

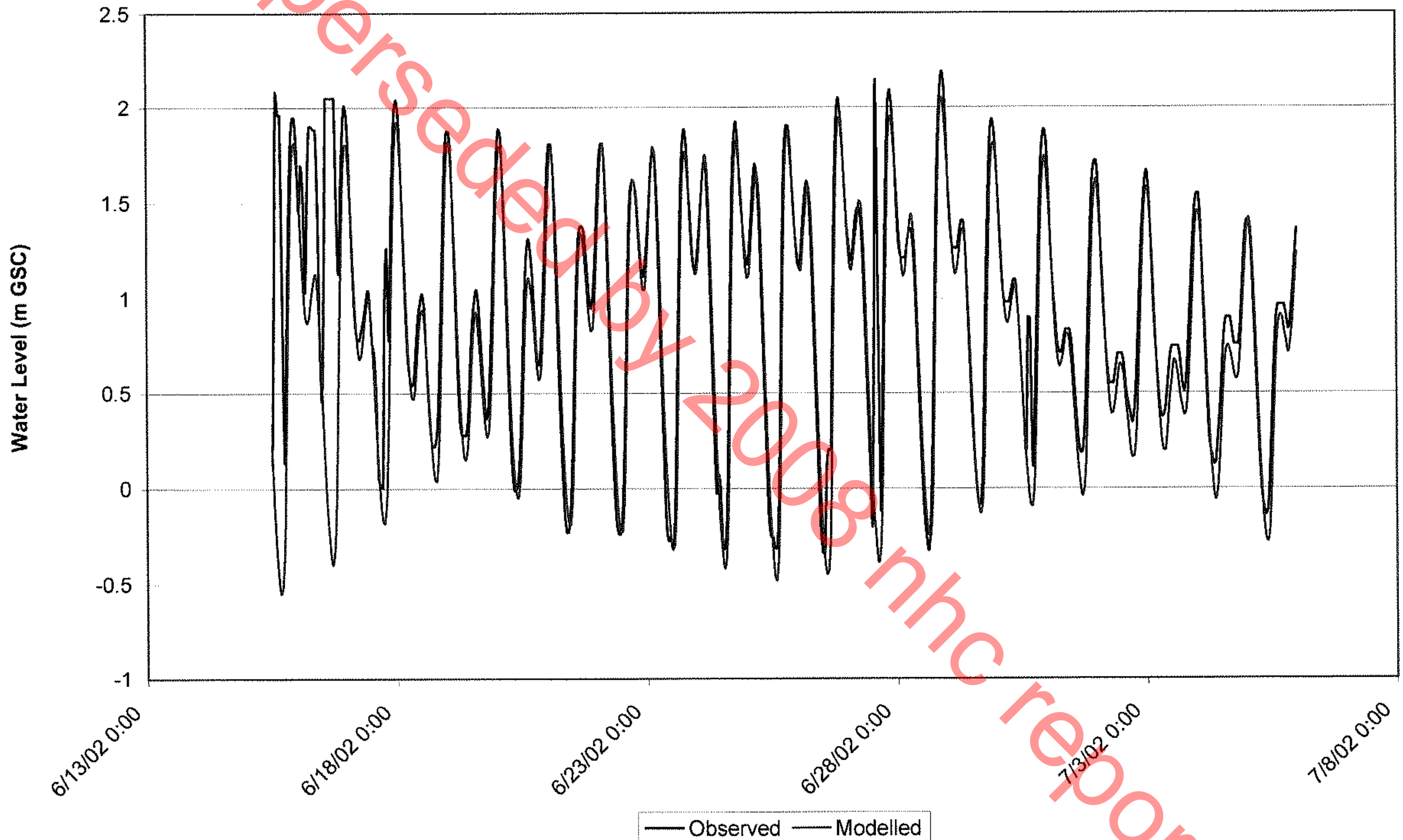
2002 Calibration
Fraser River at Port Mann

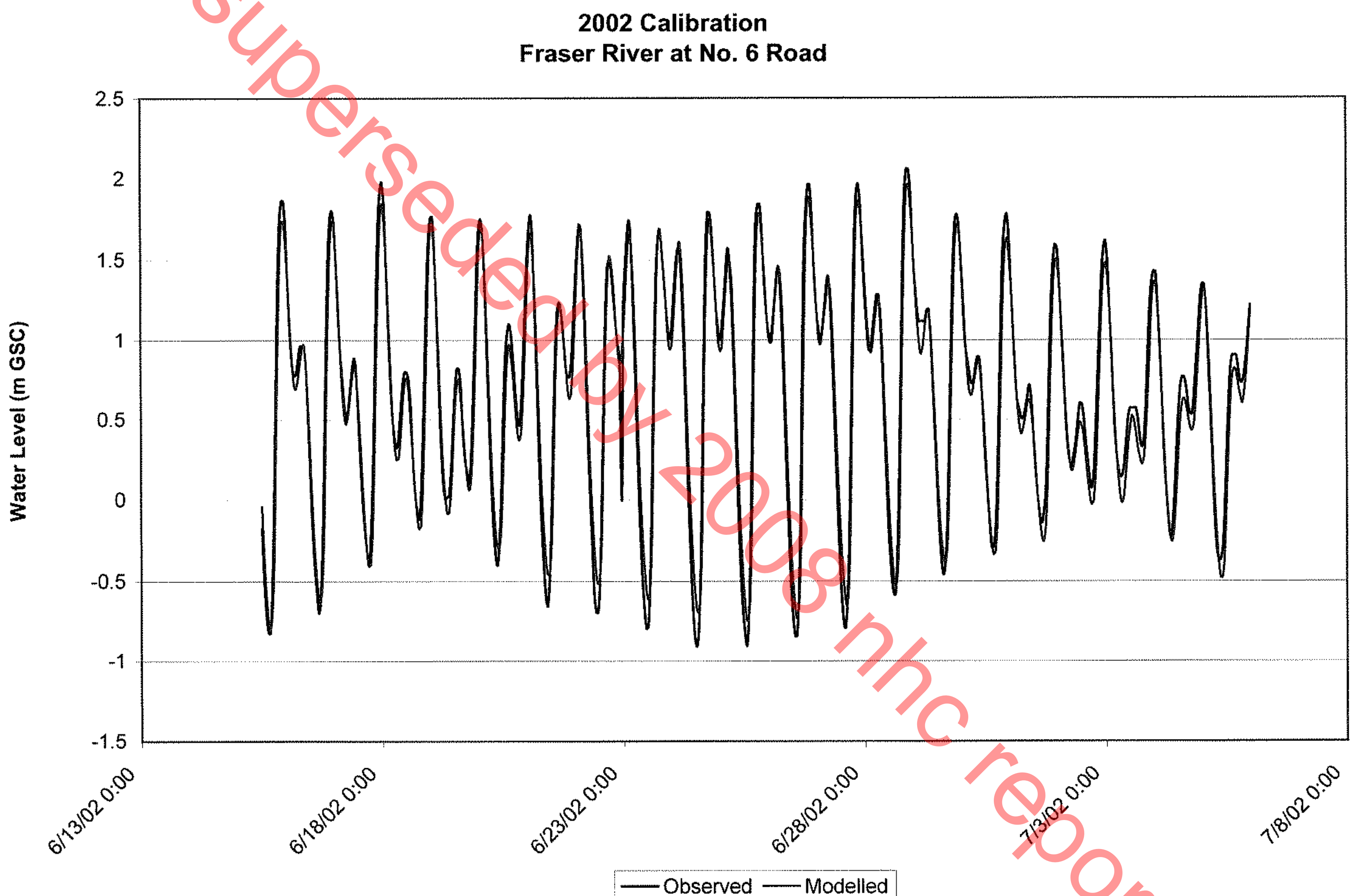



2002 Calibration
Fraser River at New Westminster
Water Level (m GSC)
3
2.5
2
1.5
1
0.5
0
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

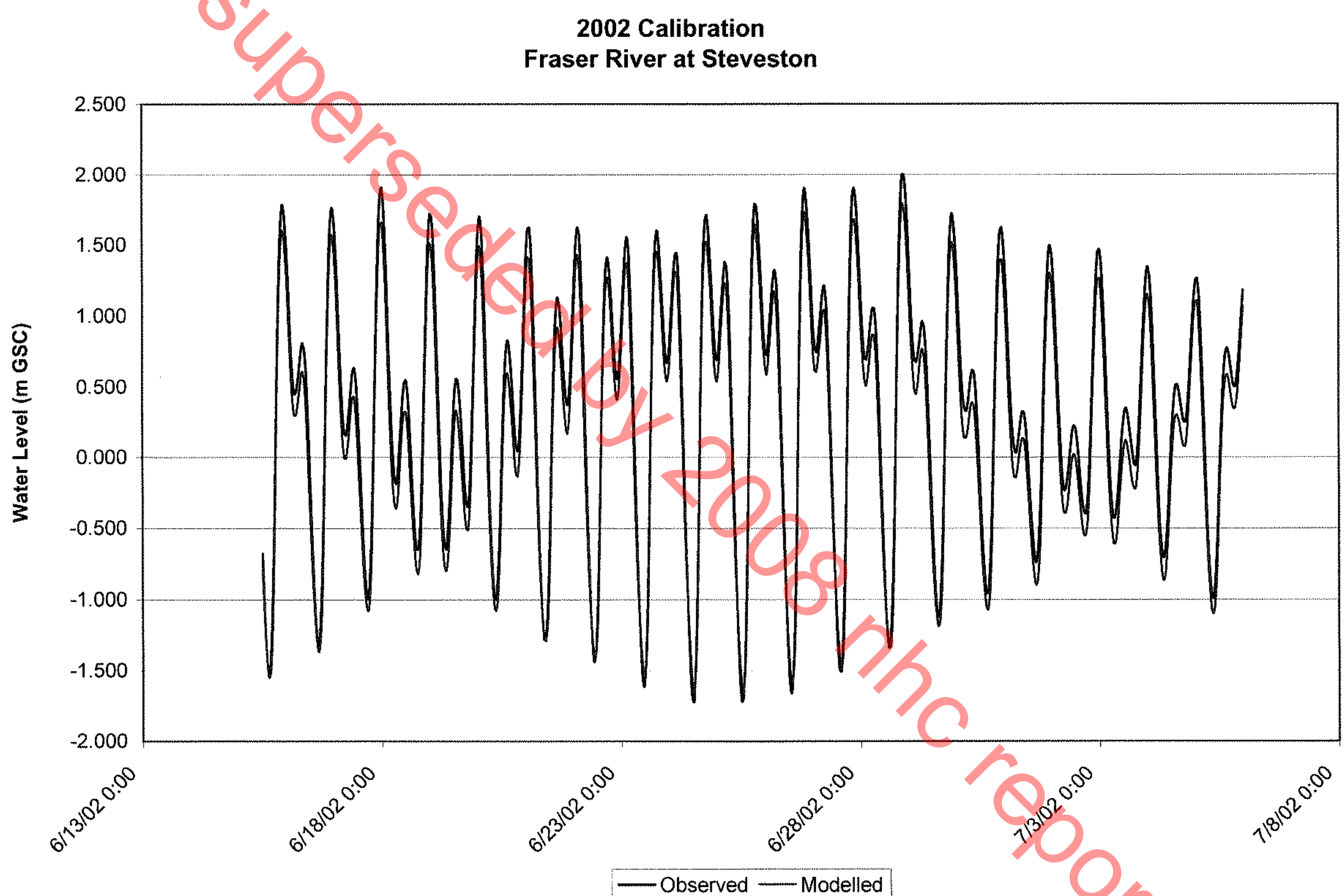

2002 Calibration
Fraser River at Manson
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

2002 Calibration
Fraser River at Nelson Road

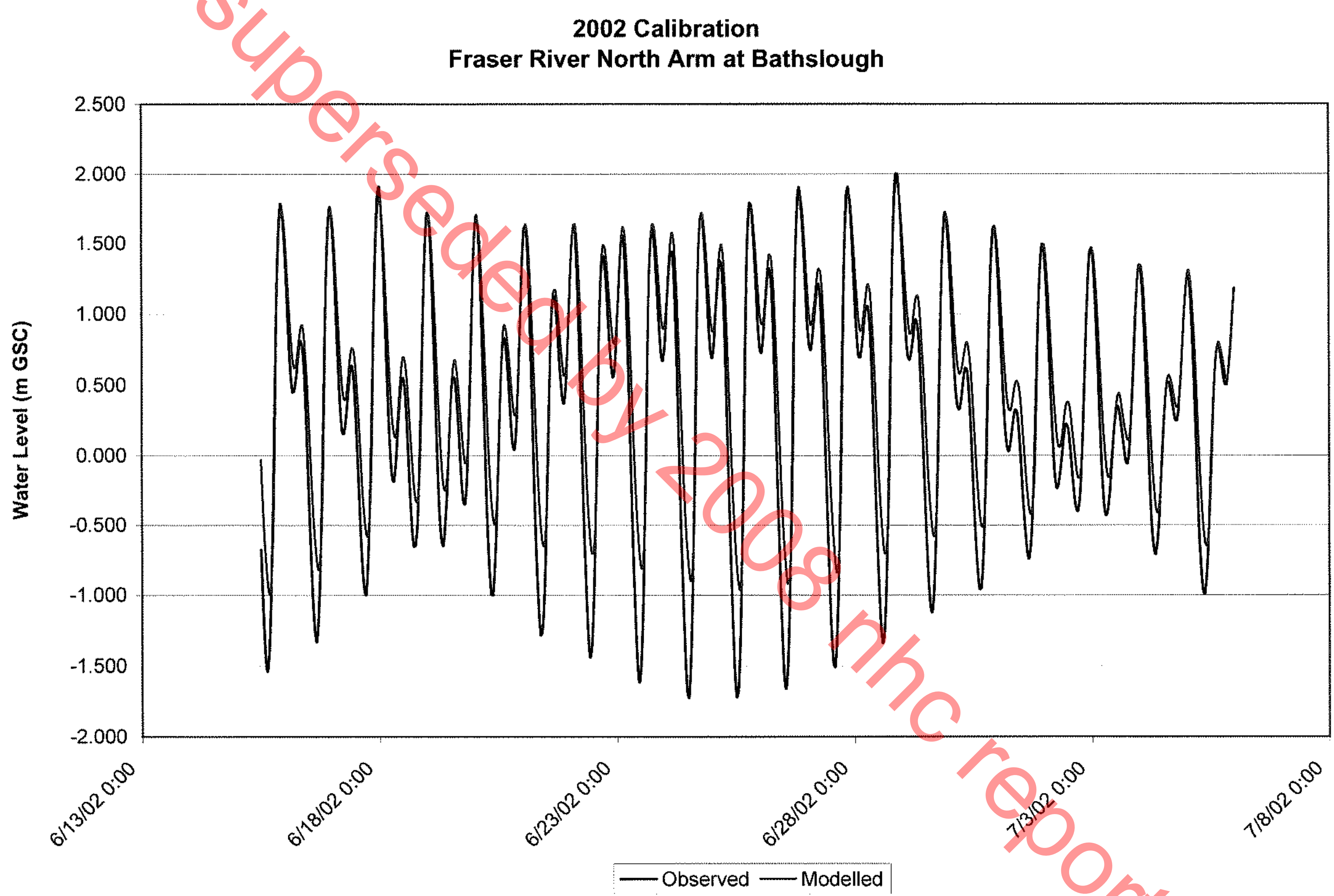




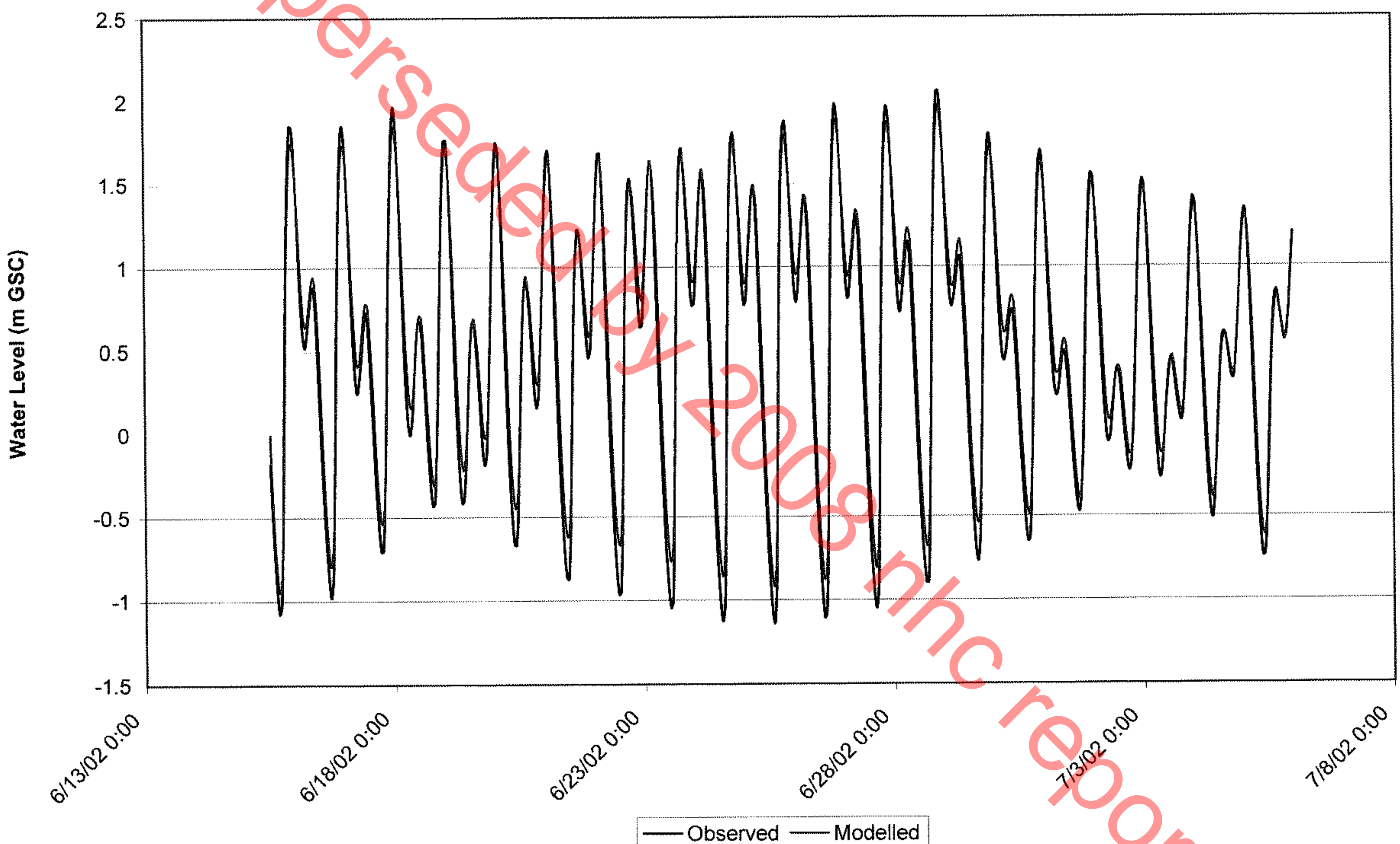
2002 Calibration
Fraser River at No. 6 Road
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

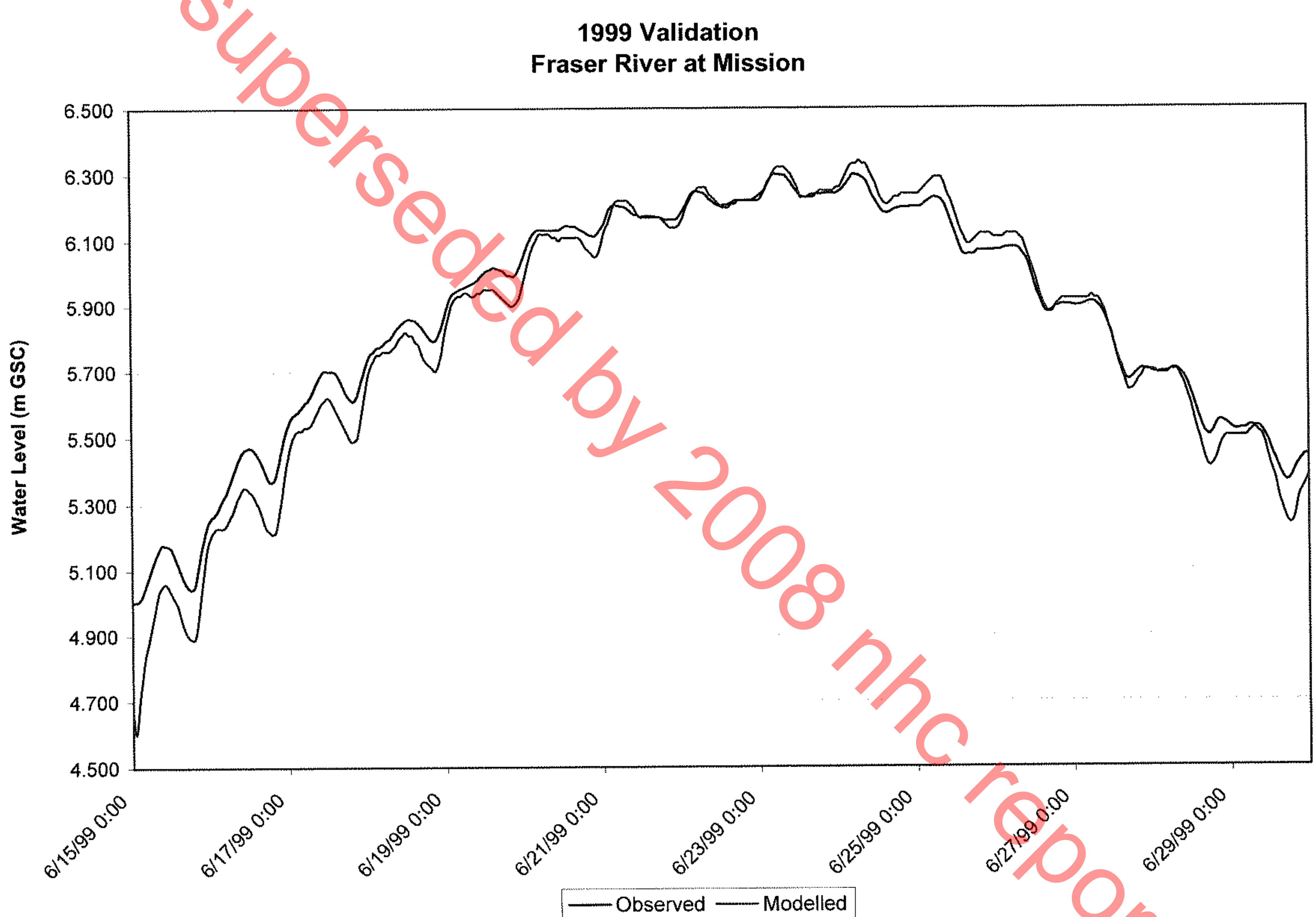


2002 Calibration
Fraser River at Steveston
Water Level (m GSC)
2.500
2.000
1.500
1.000
0.500
0.000
-0.500
-1.000
-1.500
-2.000
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

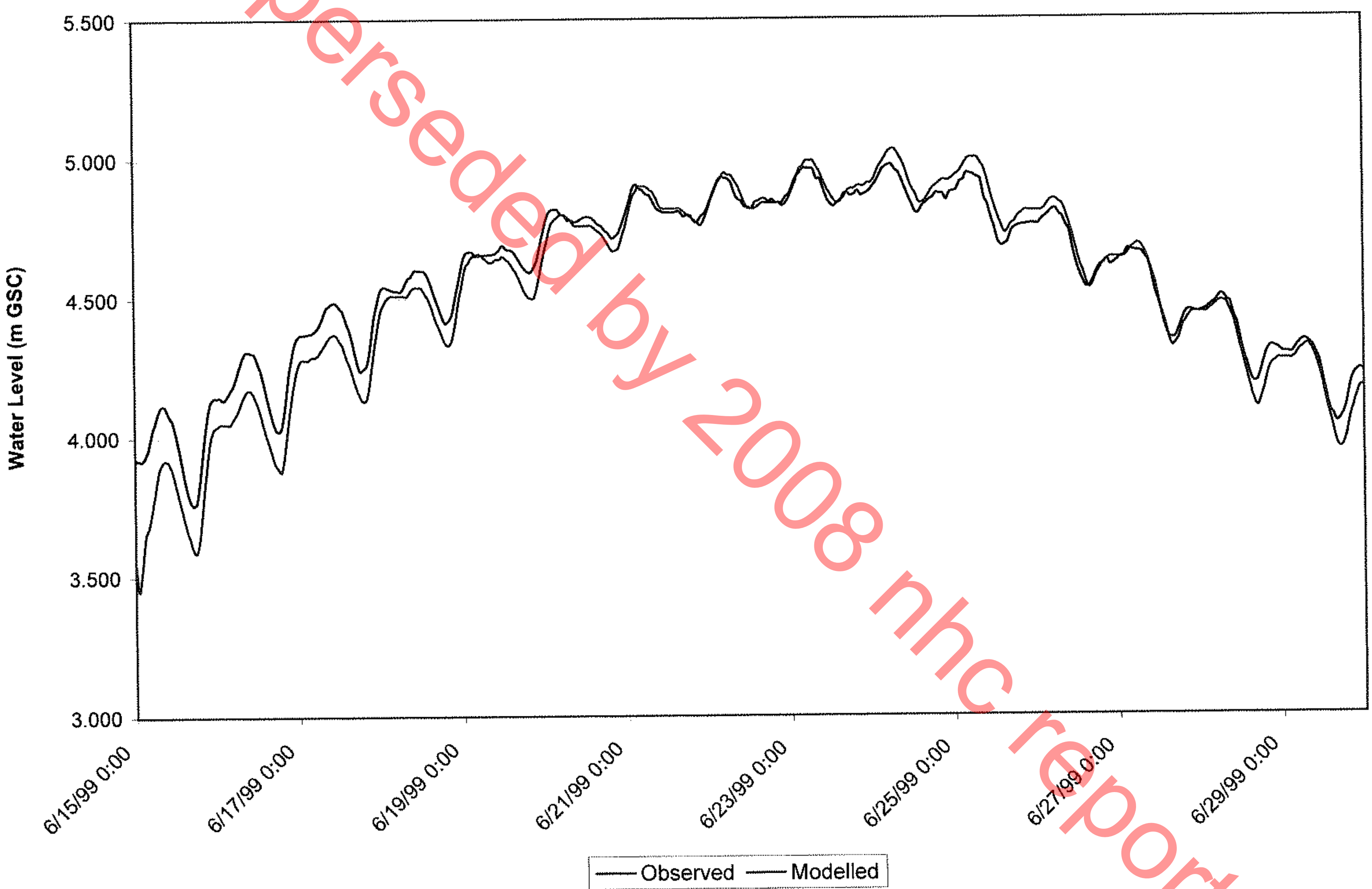

2002 Calibration
Fraser River North Arm at Bathslough
2.500
2.000
1.500
1.000
0.500
0.000
-0.500
-1.000
-1.500
-2.000
Water Level (m GSC)
6/13/02 0:00
6/18/02 0:00
6/23/02 0:00
6/28/02 0:00
7/3/02 0:00
7/8/02 0:00
Observed
Modelled

2002 Calibration
Fraser River North Arm at Vancouver

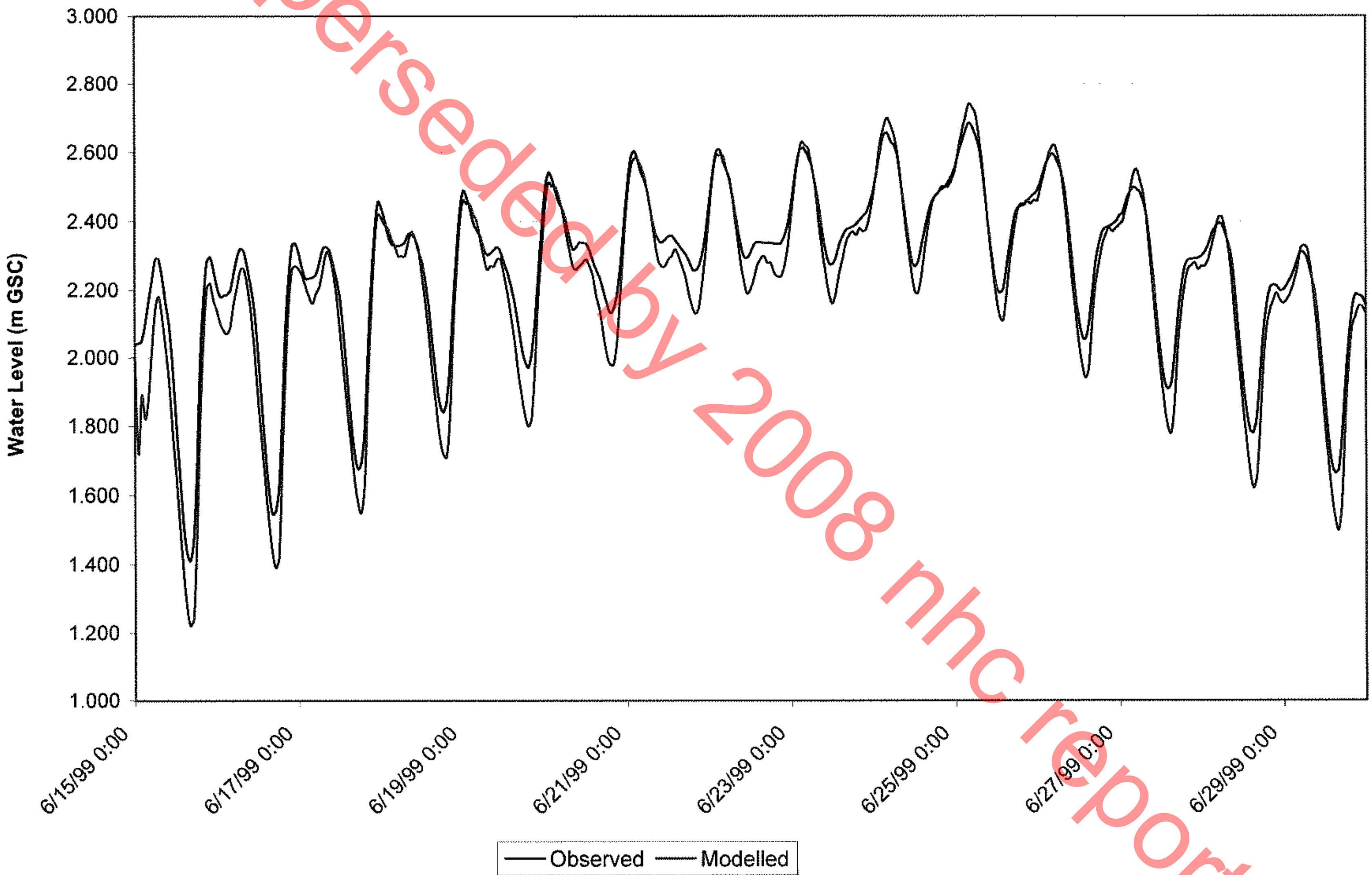



1999 Validation
Fraser River at Mission
Water Level (m GSC)
6.500
6.300
6.100
5.900
5.700
5.500
5.300
5.100
4.900
4.700
4.500
6/15/99 0:00
6/17/99 0:00
6/19/99 0:00
6/21/99 0:00
6/23/99 0:00
6/25/99 0:00
6/27/99 0:00
6/29/99 0:00
Observed
Modelled

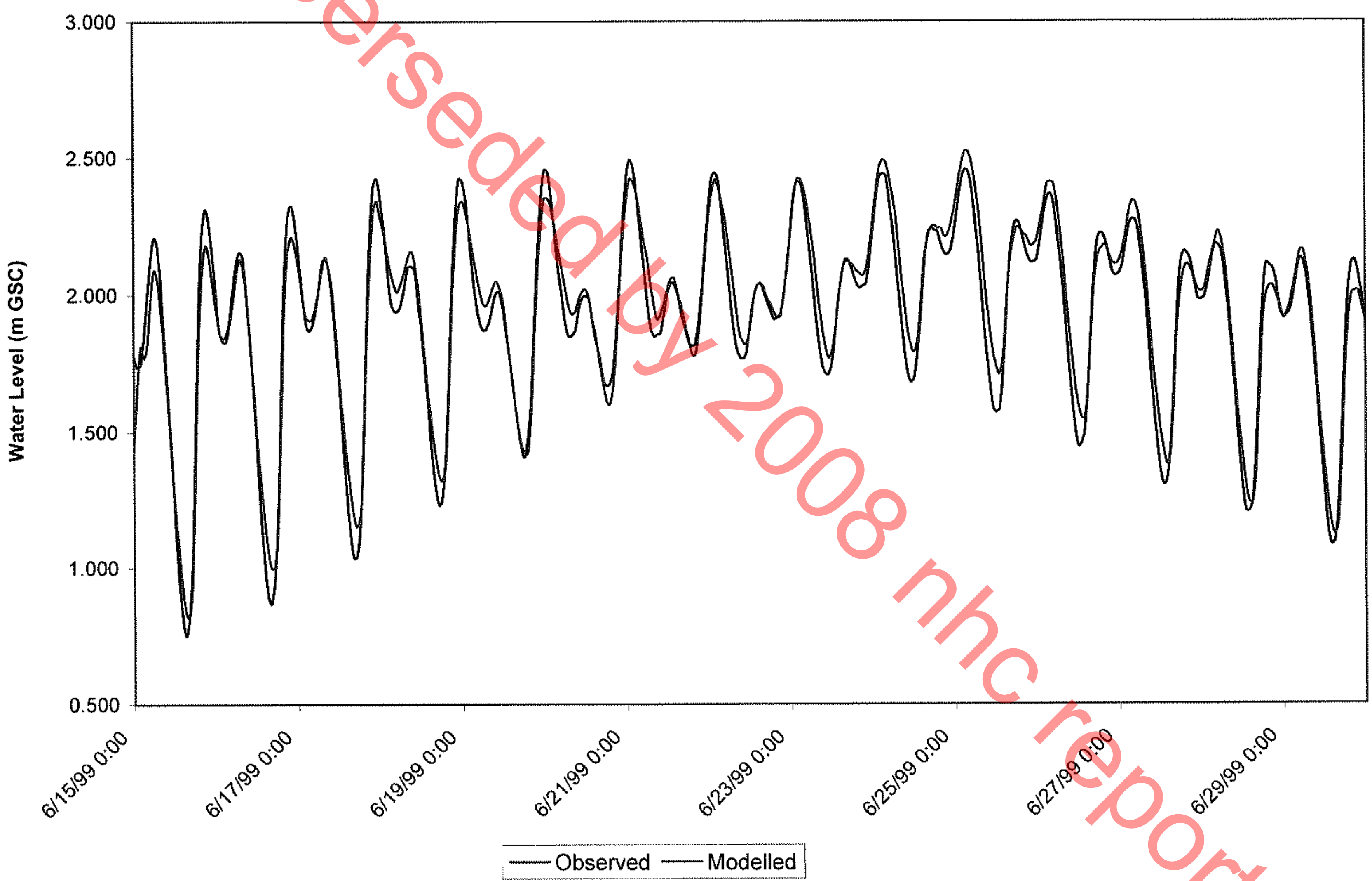
1999 Validation
Fraser River at Whonnock

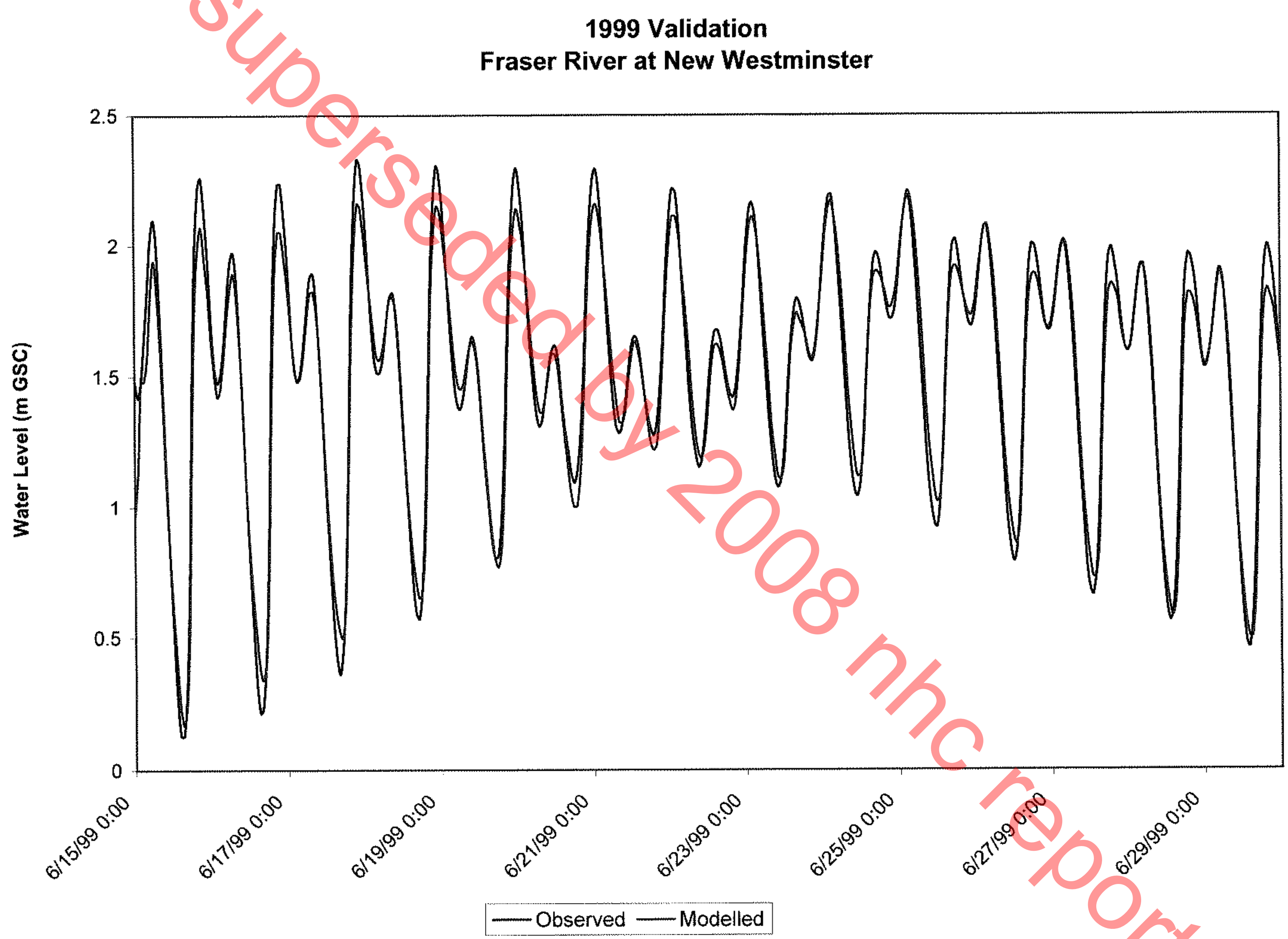


1999 Validation
Pitt River at Port Coquitlam

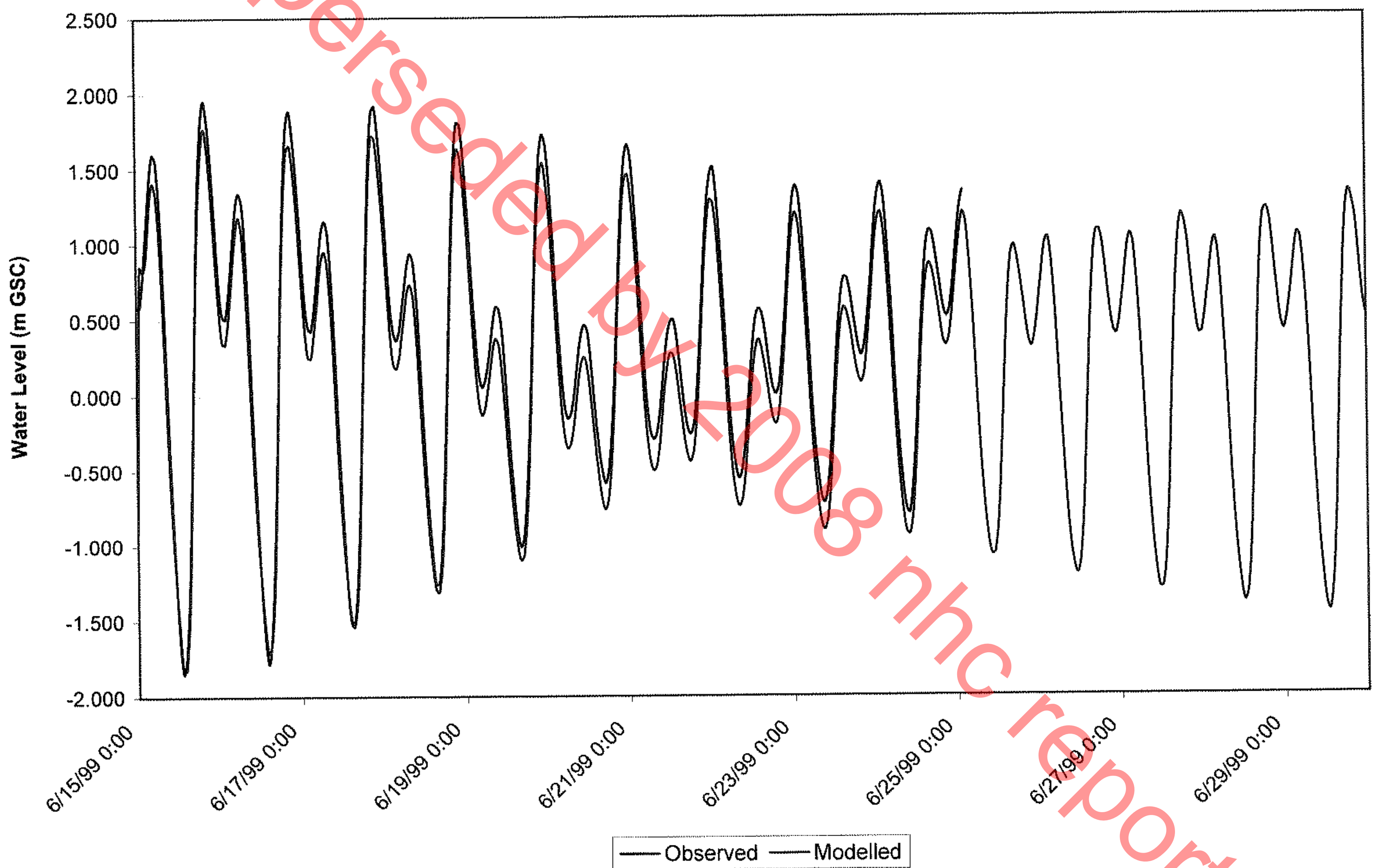


1999 Validation
Fraser River at Port Mann

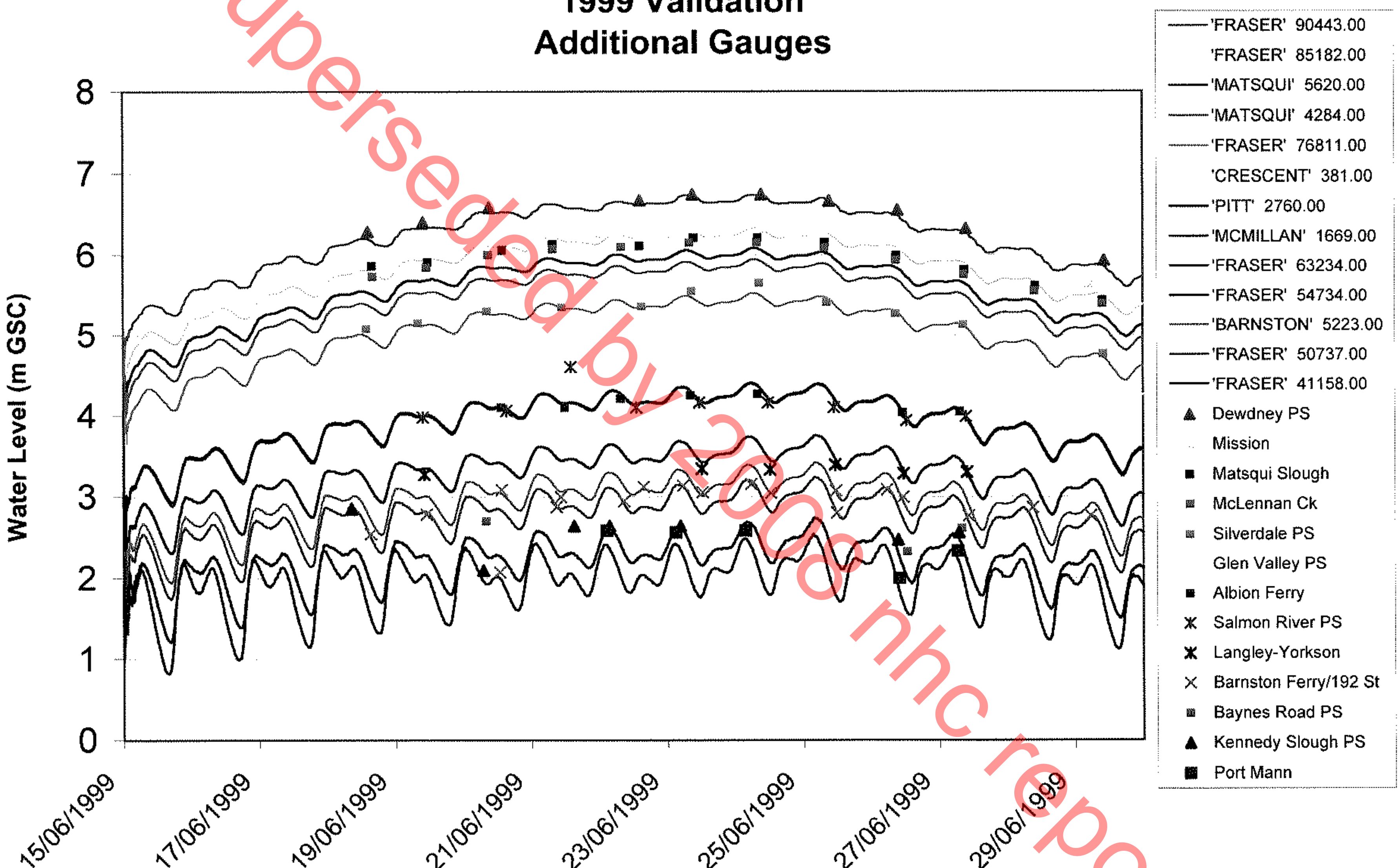



1999 Validation
Fraser River at New Westminster
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
6/15/99 0:00
6/17/99 0:00
6/19/99 0:00
6/21/99 0:00
6/23/99 0:00
6/25/99 0:00
6/27/99 0:00
6/29/99 0:00
Observed
Modelled

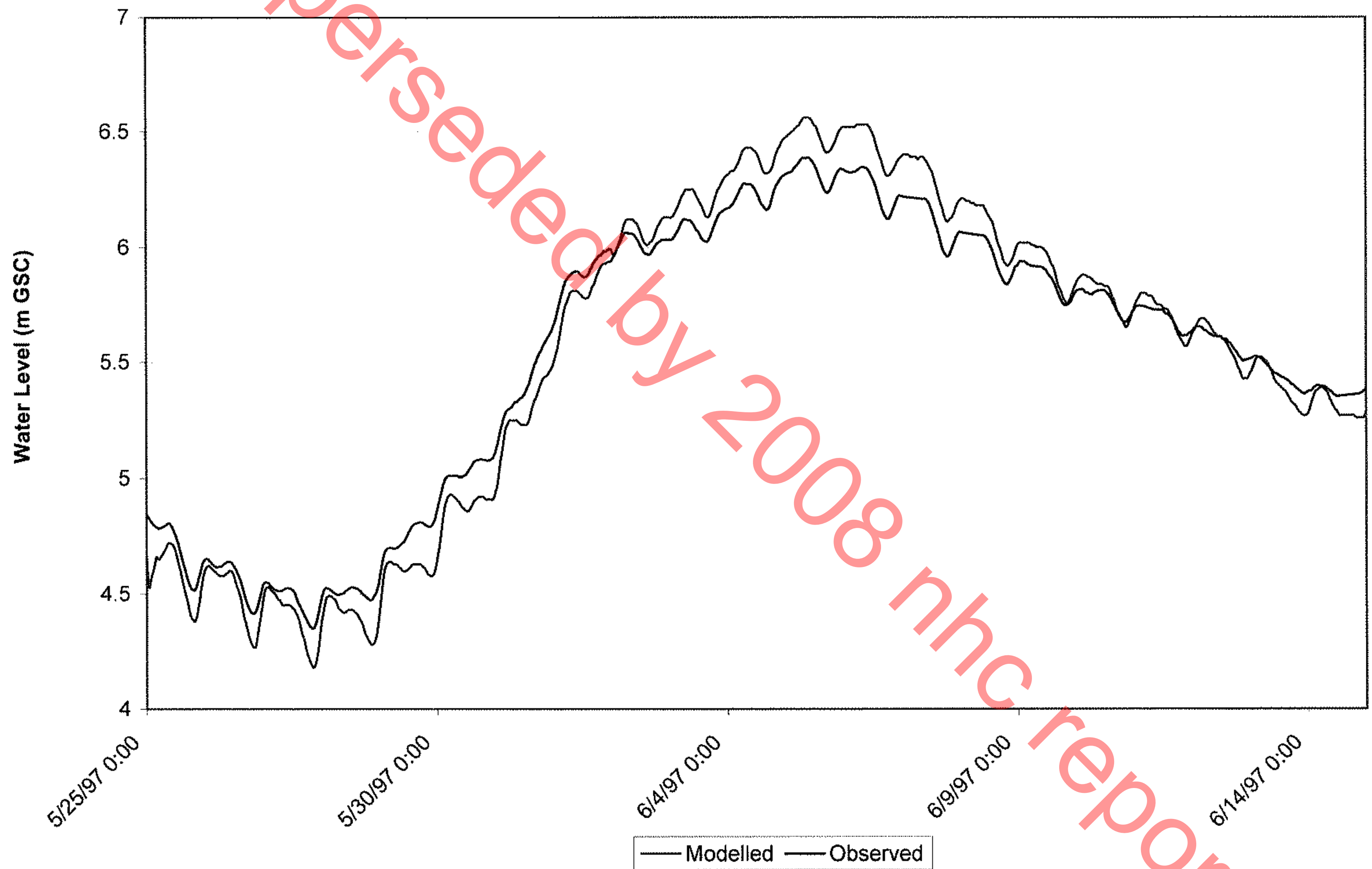
1999 Validation
Fraser River at Steveston

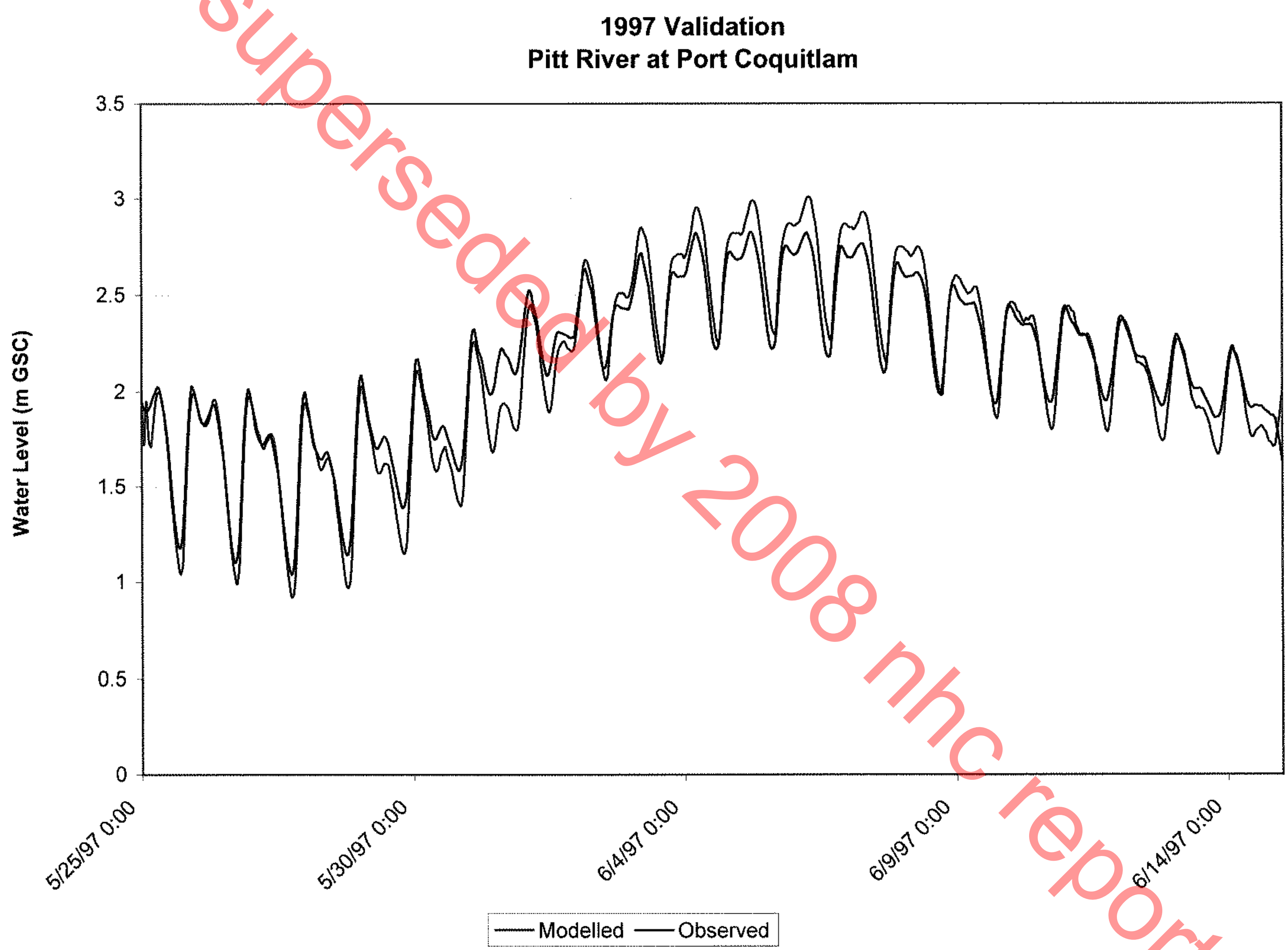


1999 Validation
Additional Gauges



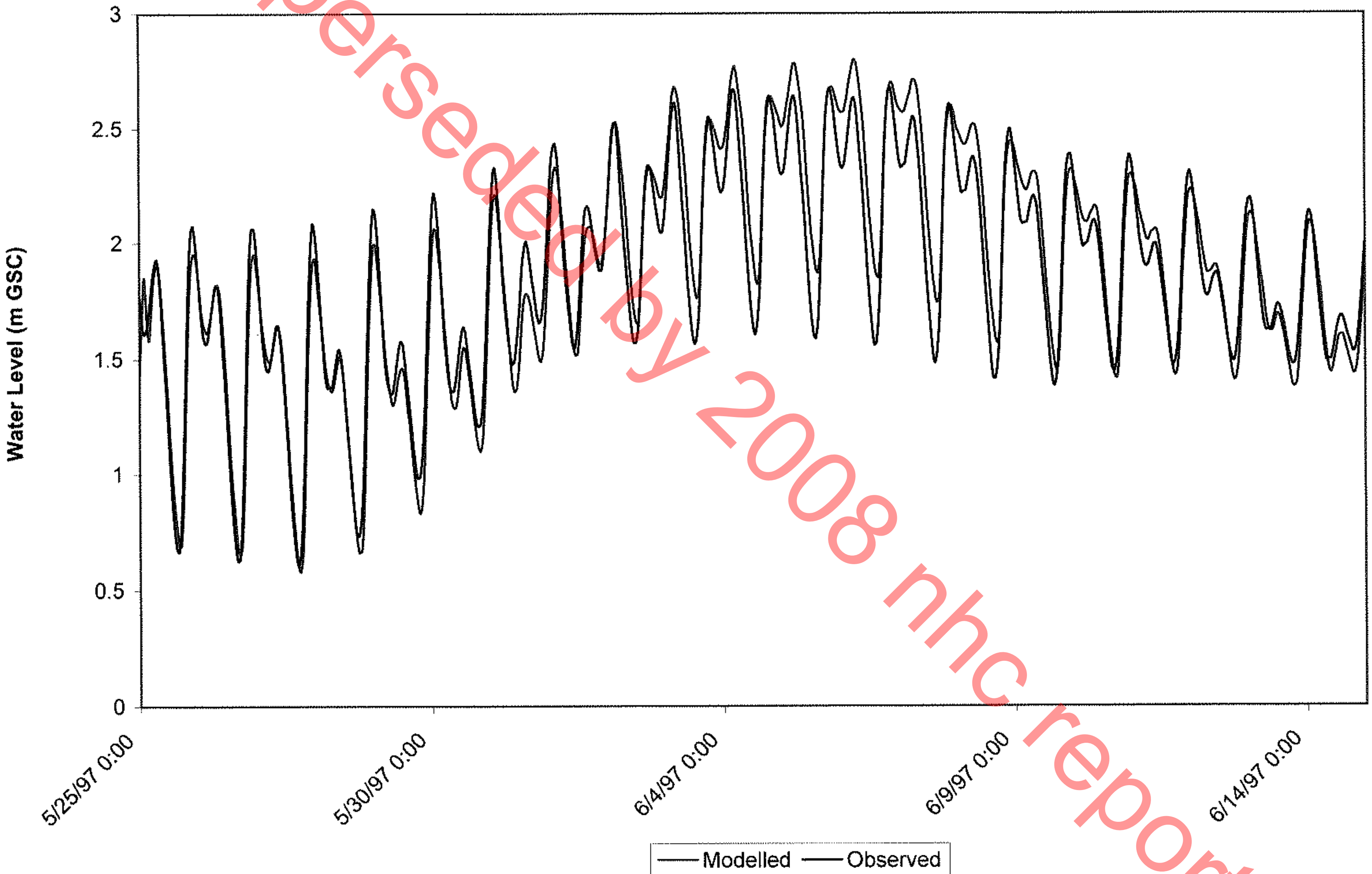
1997 Validation
Fraser River at Mission

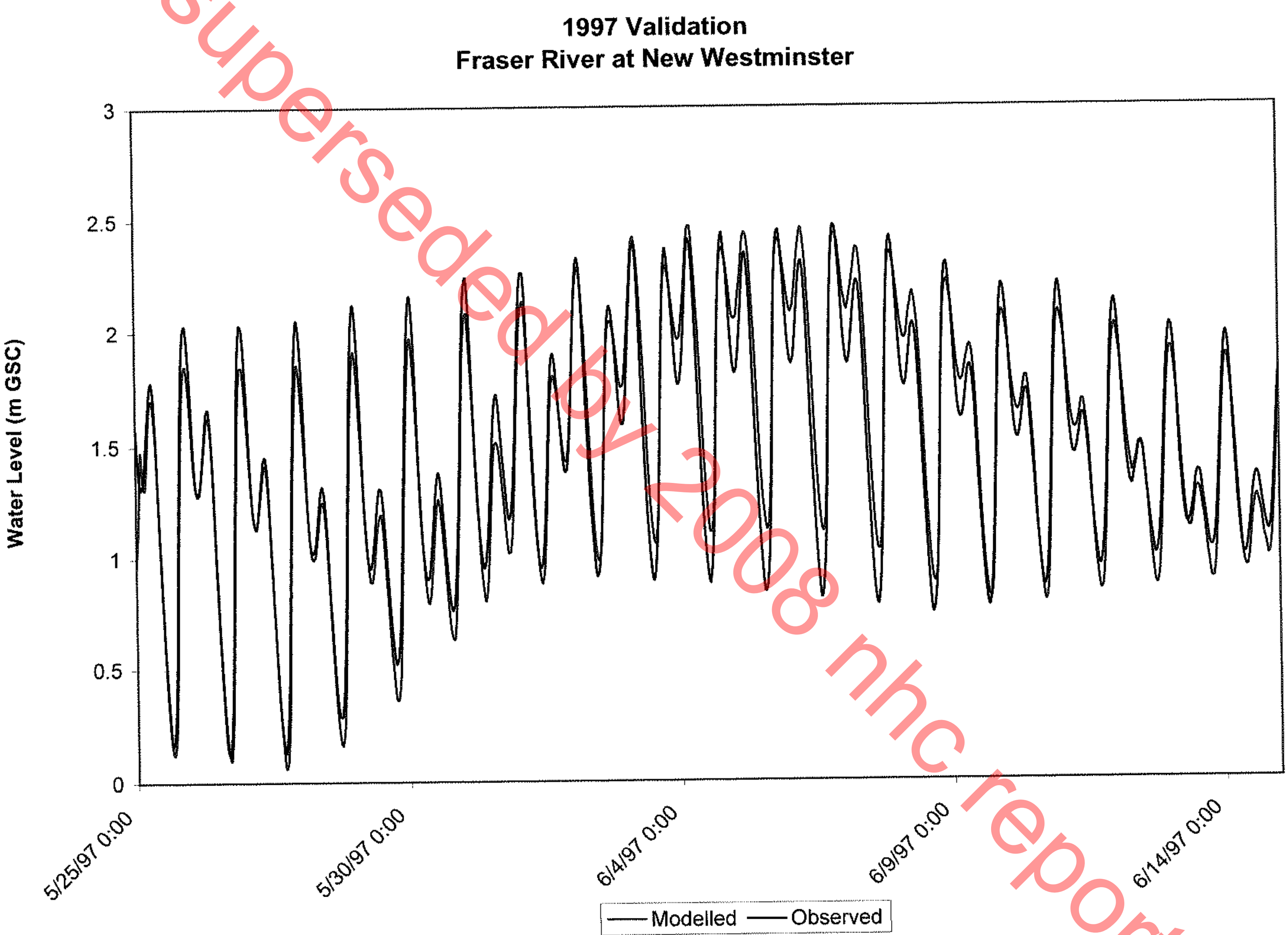




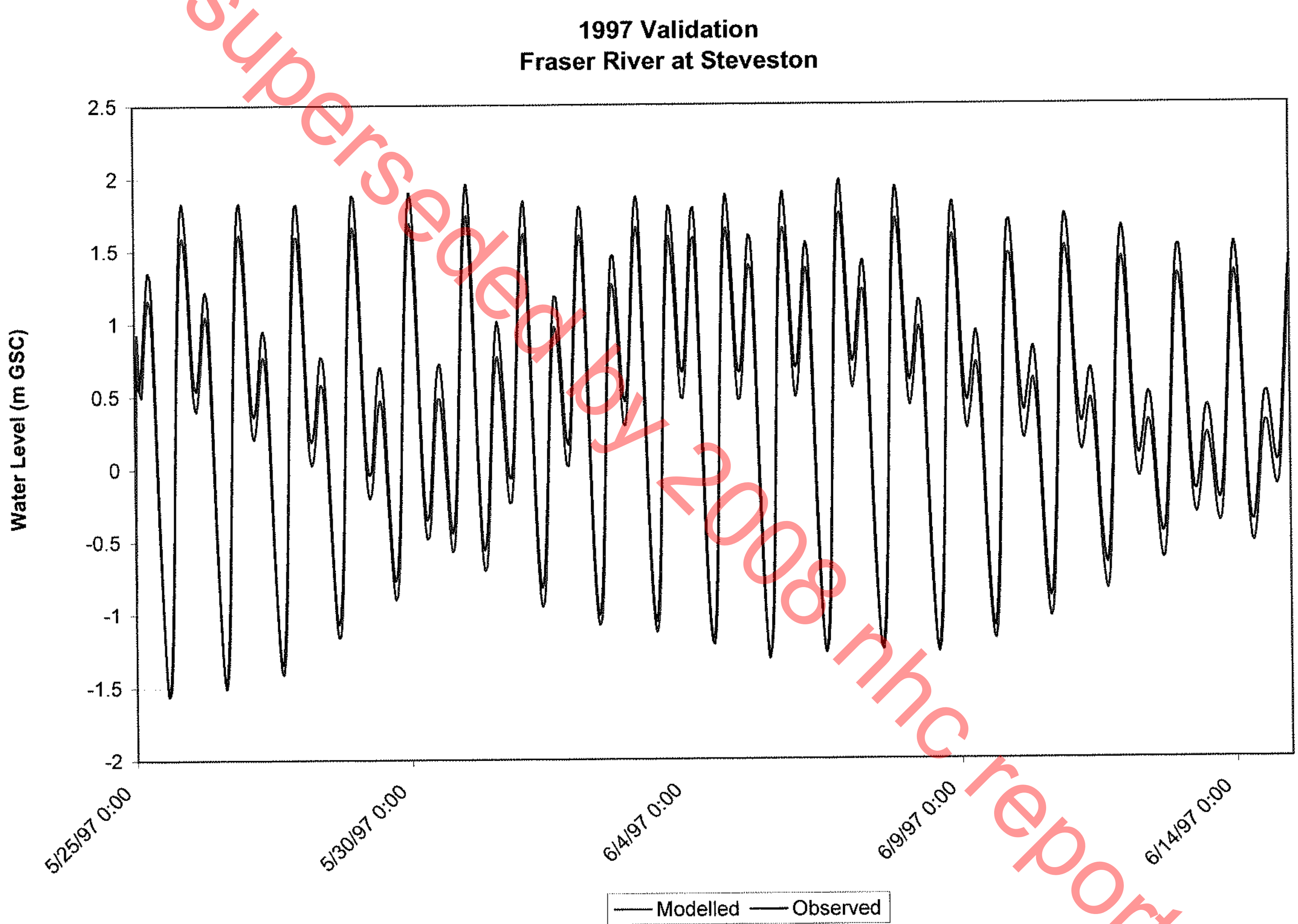
1997 Validation
Pitt River at Port Coquitlam
Water Level (m GSC)
3.5
3
2.5
2
1.5
1
0.5
0
5/25/97 0:00
5/30/97 0:00
6/4/97 0:00
6/9/97 0:00
6/14/97 0:00
Modelled
Observed

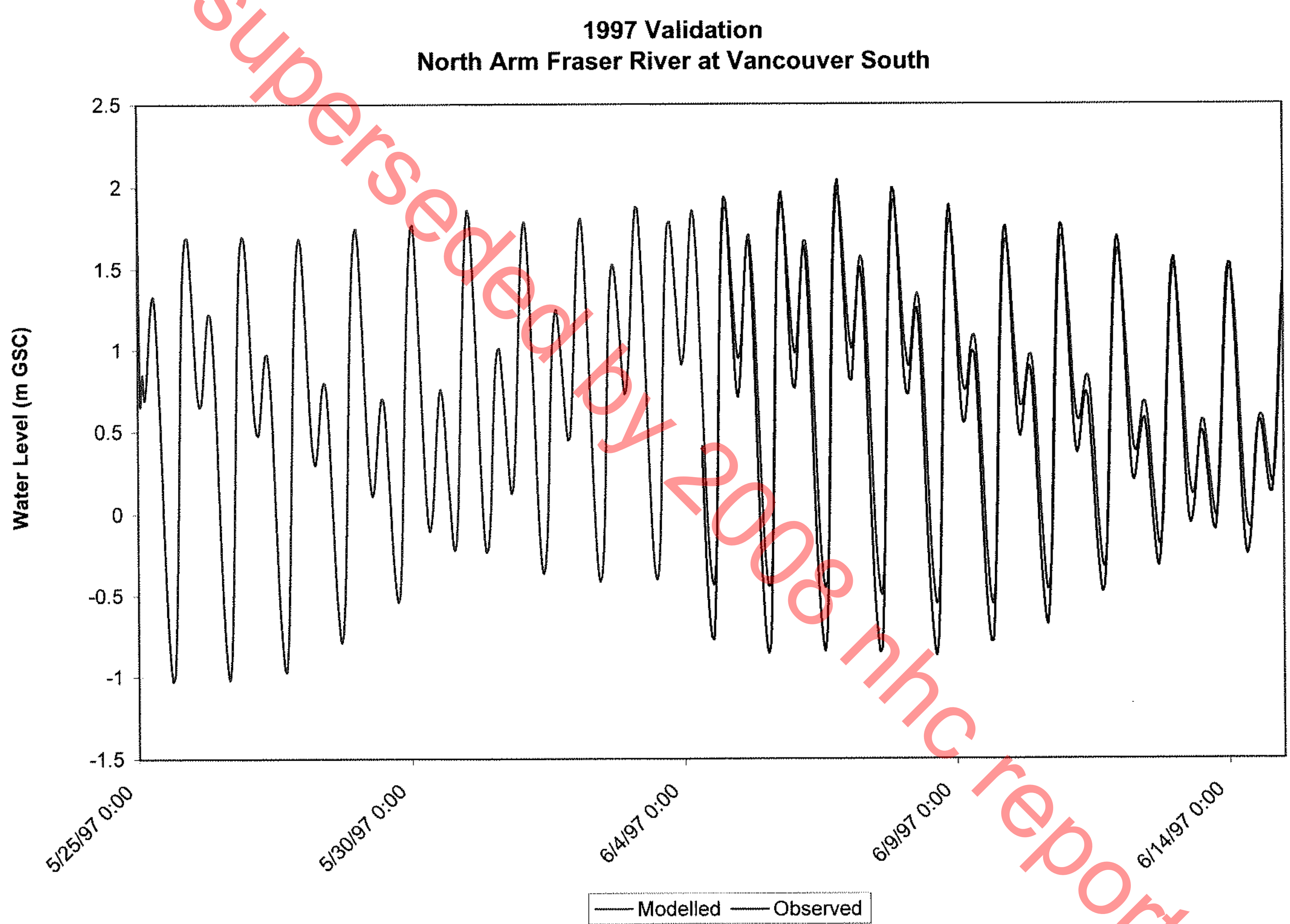
1997 Validation
Fraser River at Port Mann



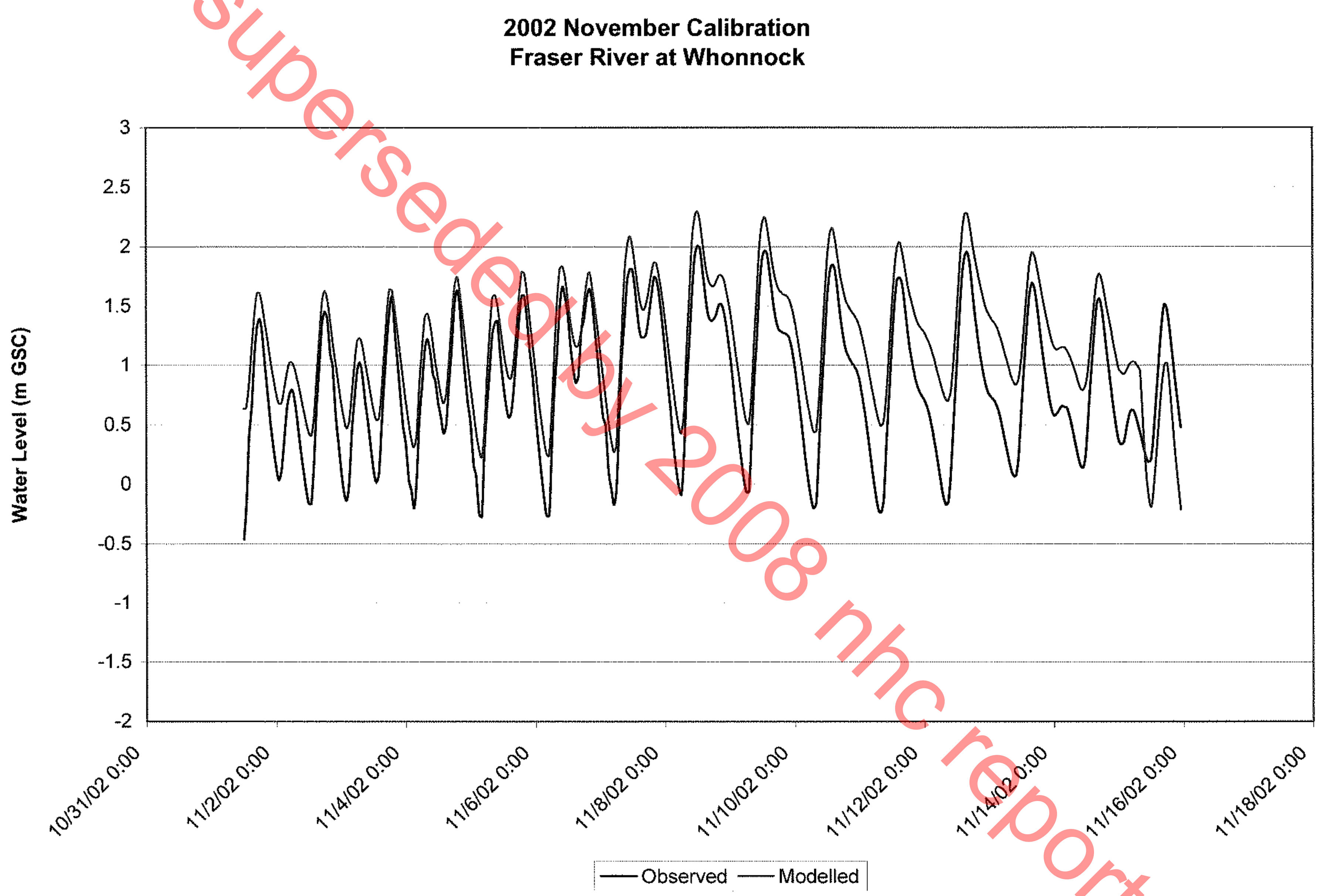


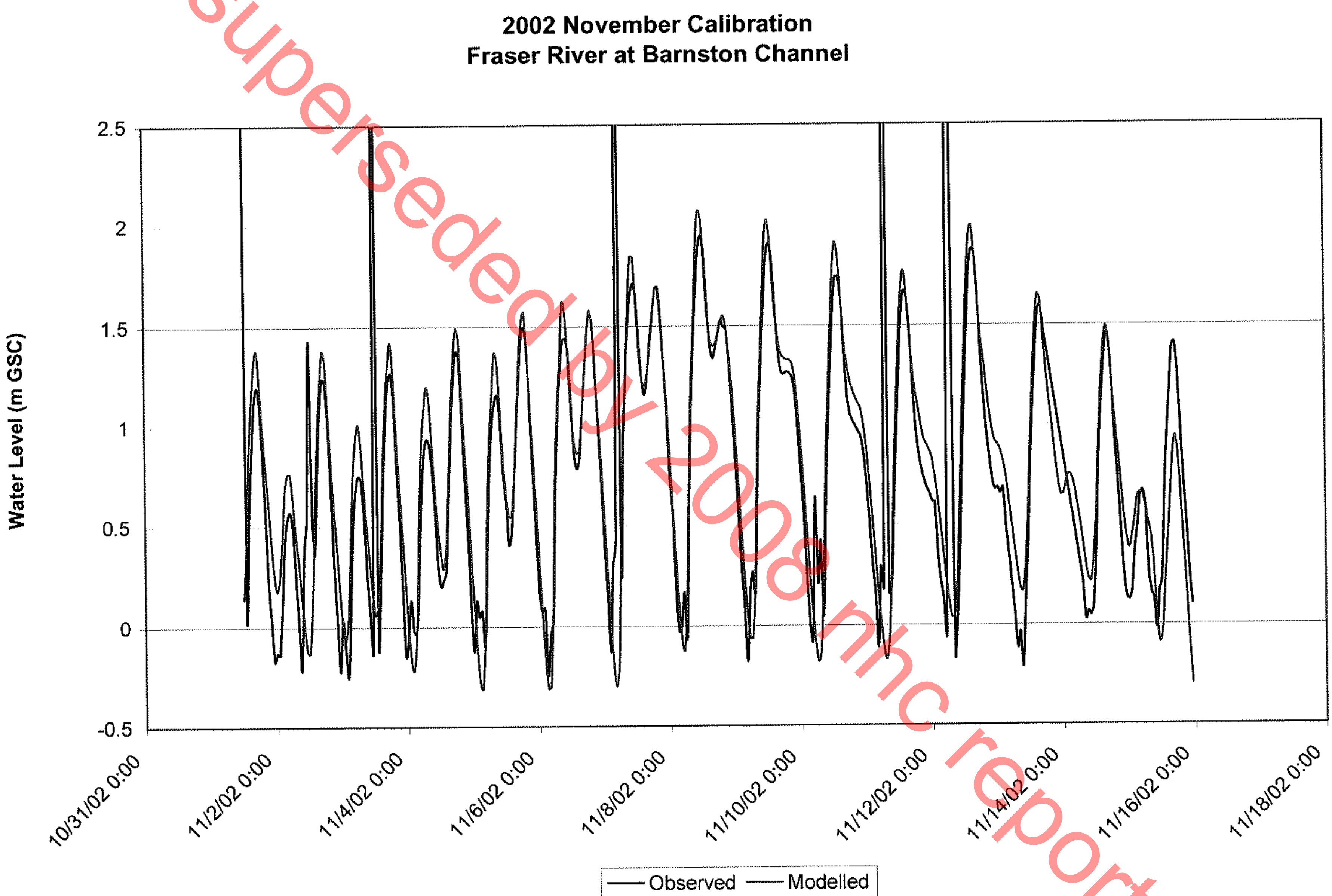
1997 Validation
Fraser River at New Westminster
Water Level (m GSC)
3
2.5
2
1.5
1
0.5
0
5/25/97 0:00
5/30/97 0:00
6/4/97 0:00
6/9/97 0:00
6/14/97 0:00
Modelled
Observed

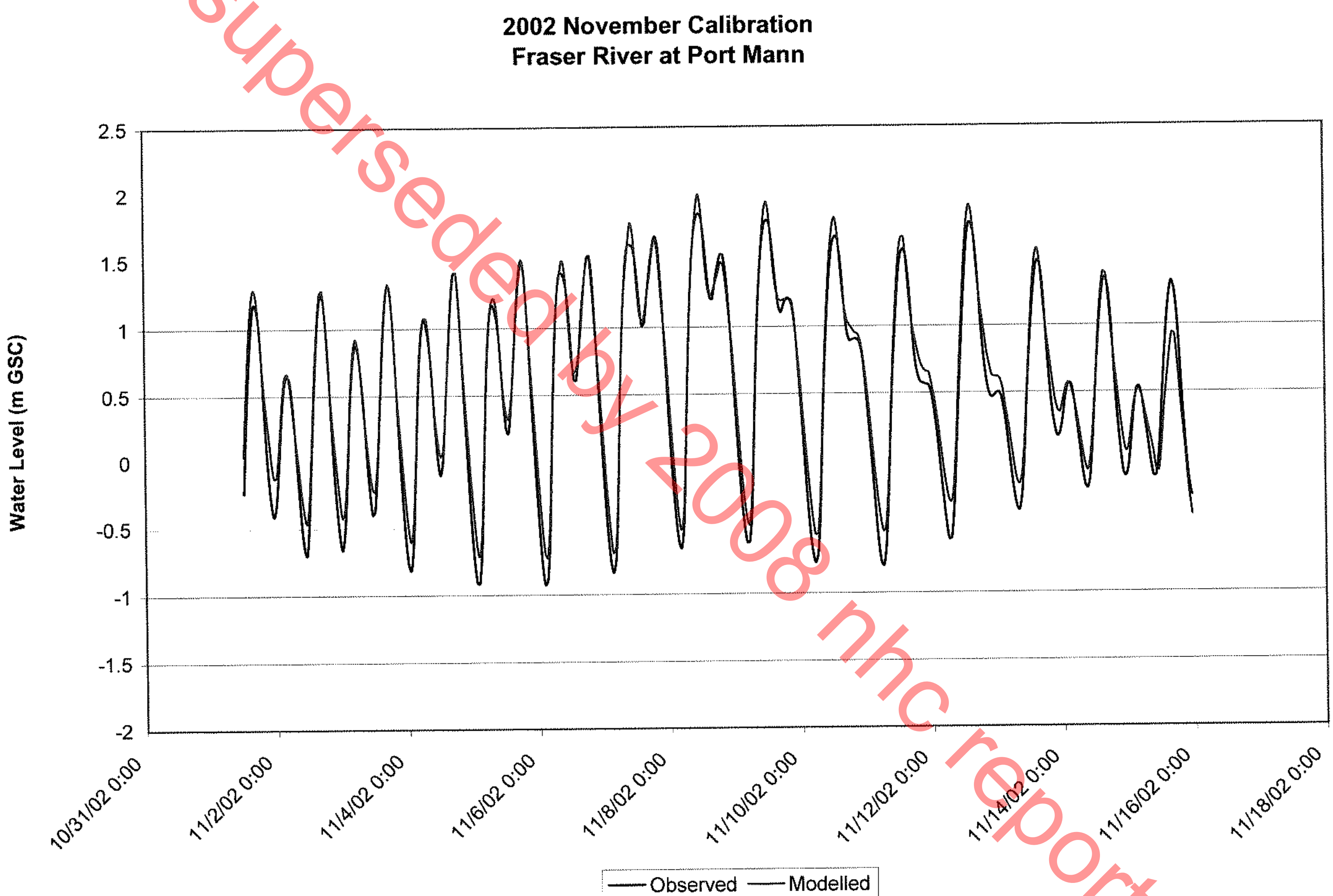

1997 Validation
Fraser River at Steveston
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
5/25/97 0:00
5/30/97 0:00
6/4/97 0:00
6/9/97 0:00
6/14/97 0:00
Modelled
Observed



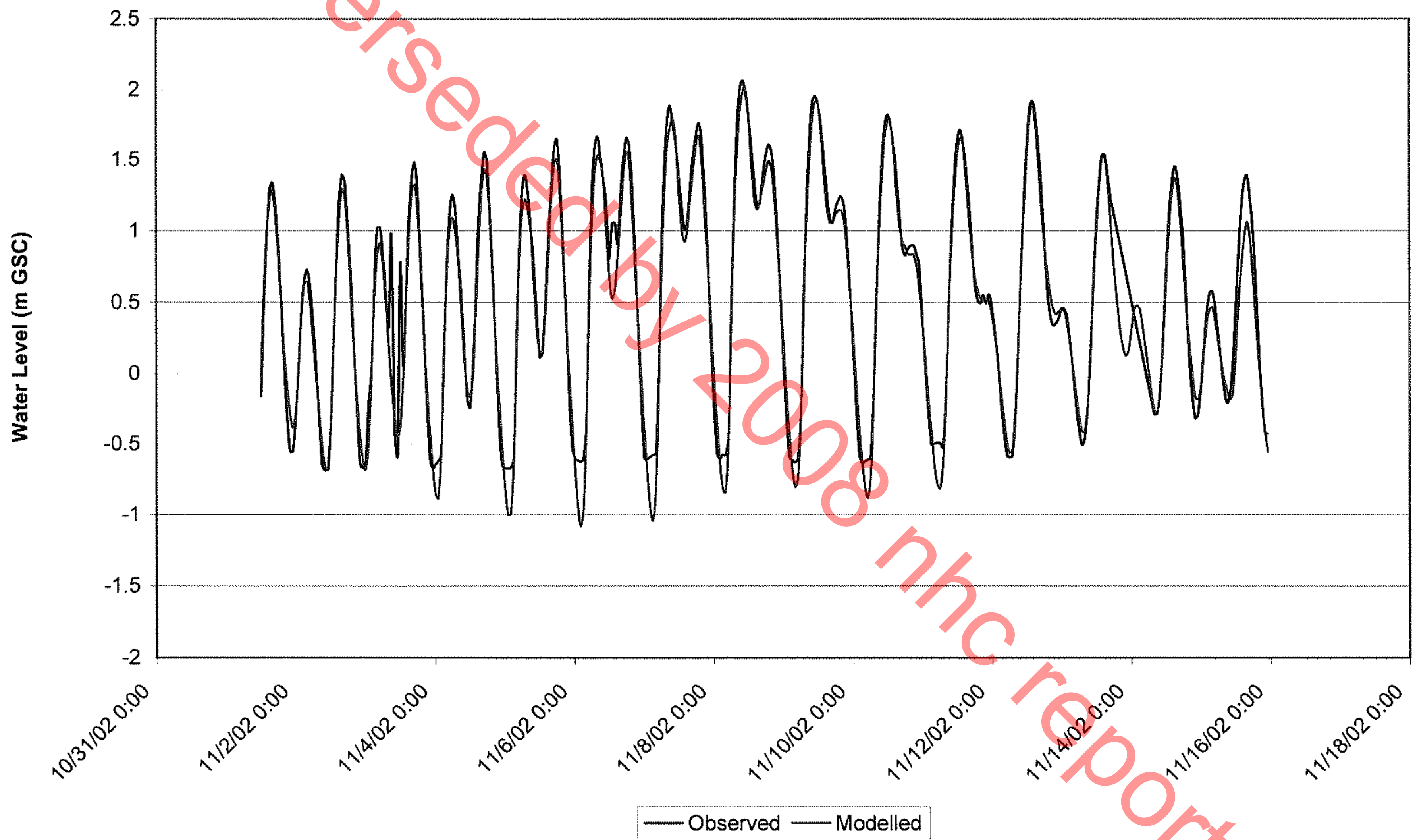
1997 Validation
North Arm Fraser River at Vancouver South
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
5/25/97 0:00
5/30/97 0:00
6/4/97 0:00
6/9/97 0:00
6/14/97 0:00
Modelled
Observed

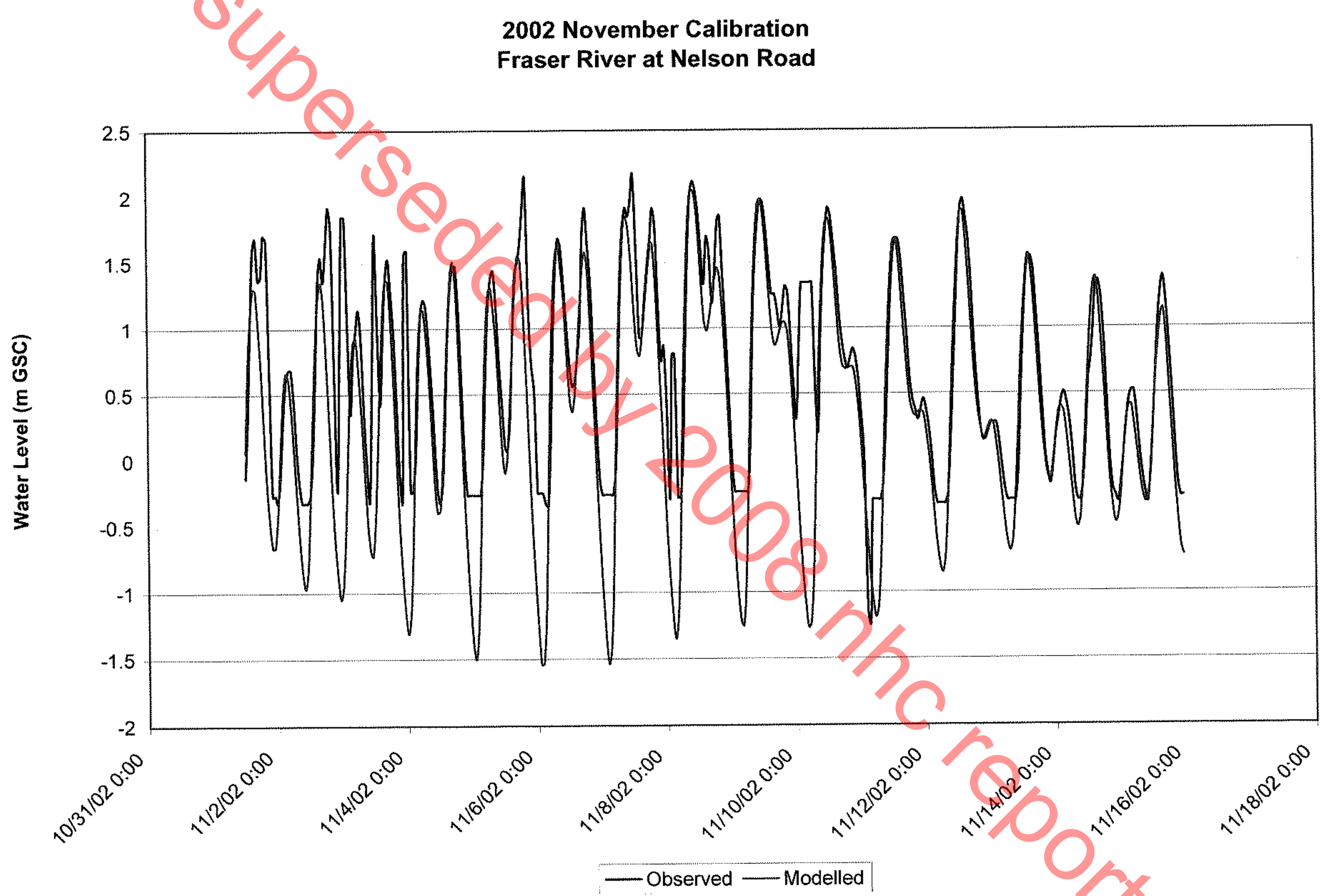

2002 November Calibration
Fraser River at Whonnock
Water Level (m GSC)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

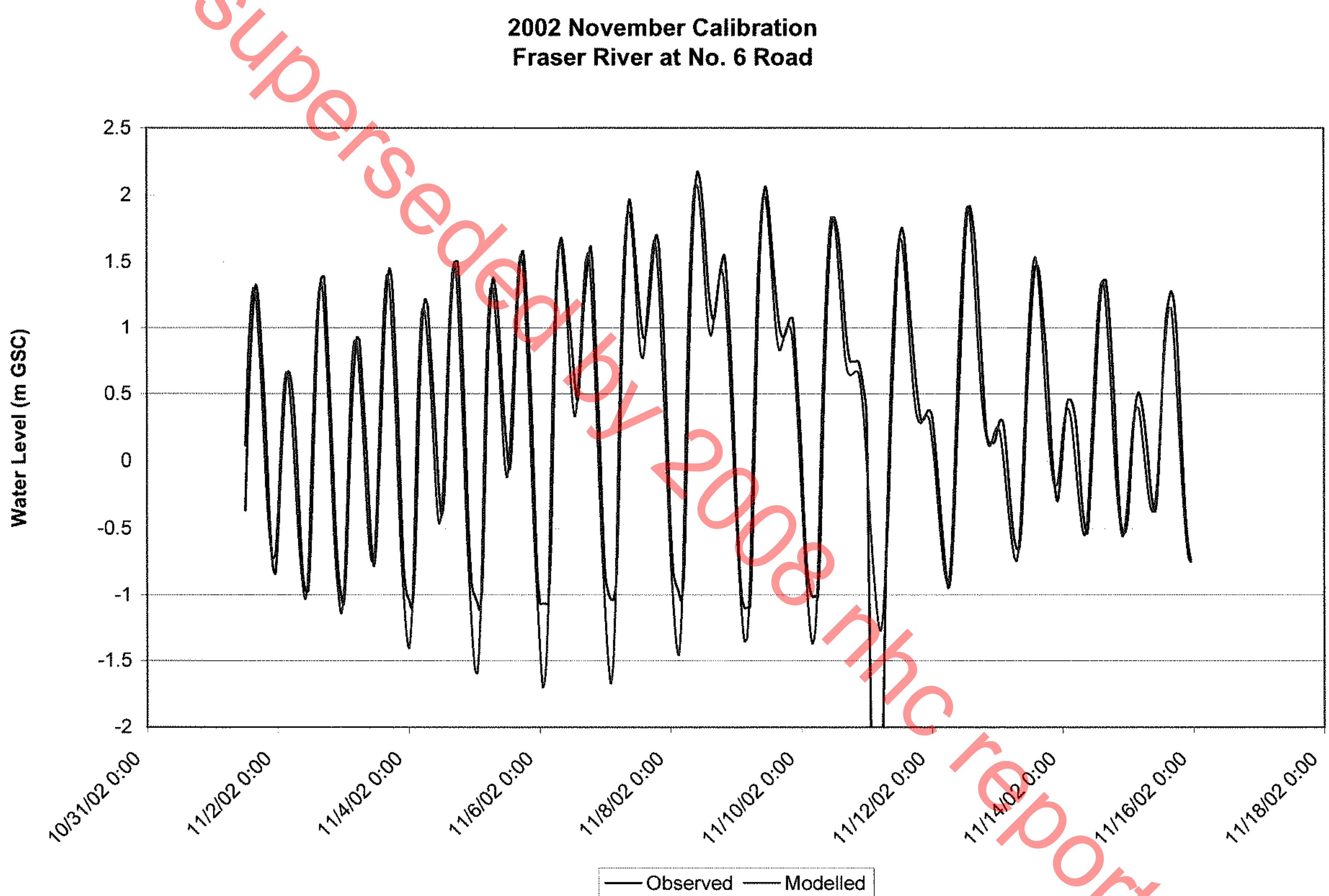

2002 November Calibration
Fraser River at Barnston Channel
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

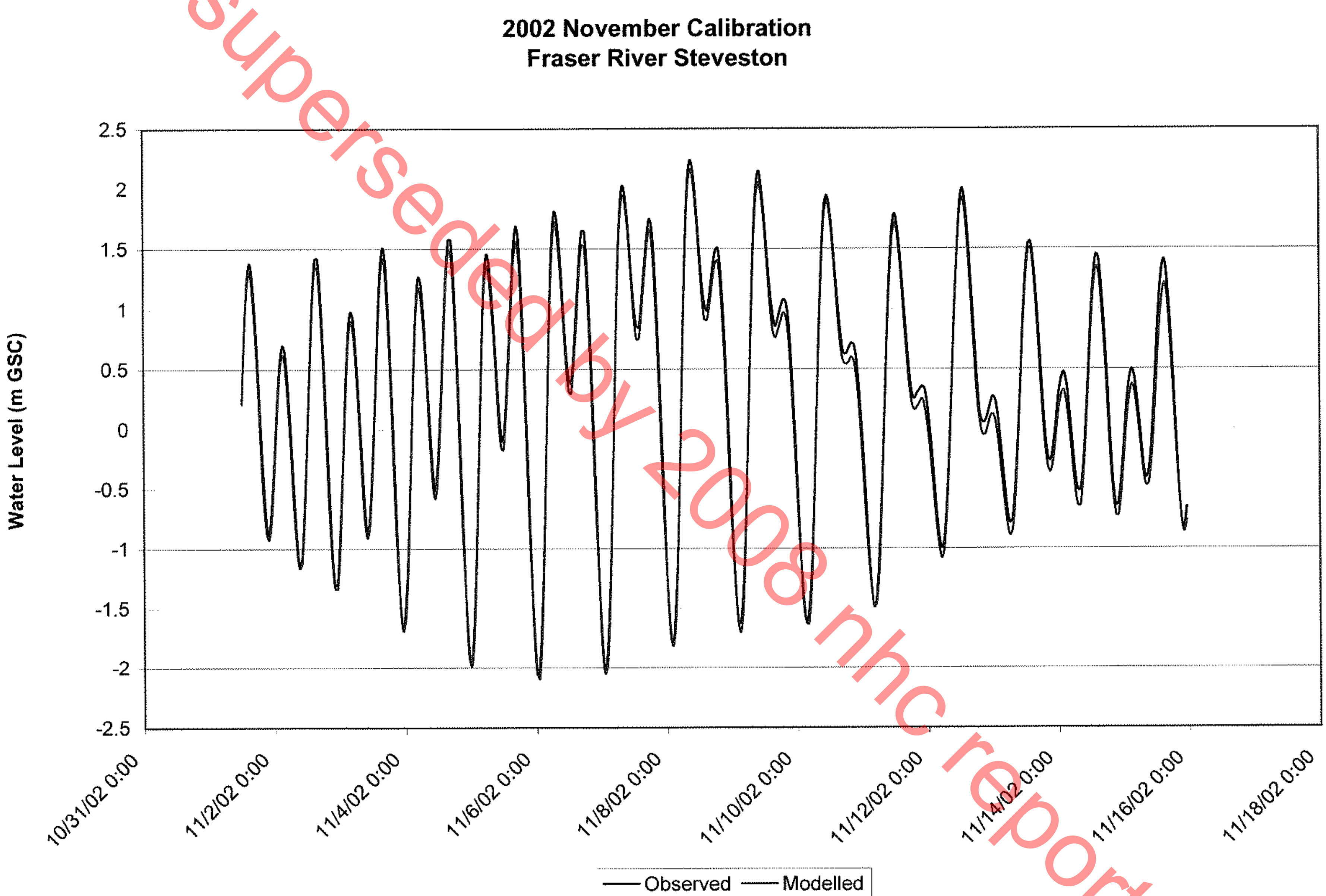

2002 November Calibration
Fraser River at Port Mann
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

2002 November Calibration
Fraser River at Manson Road

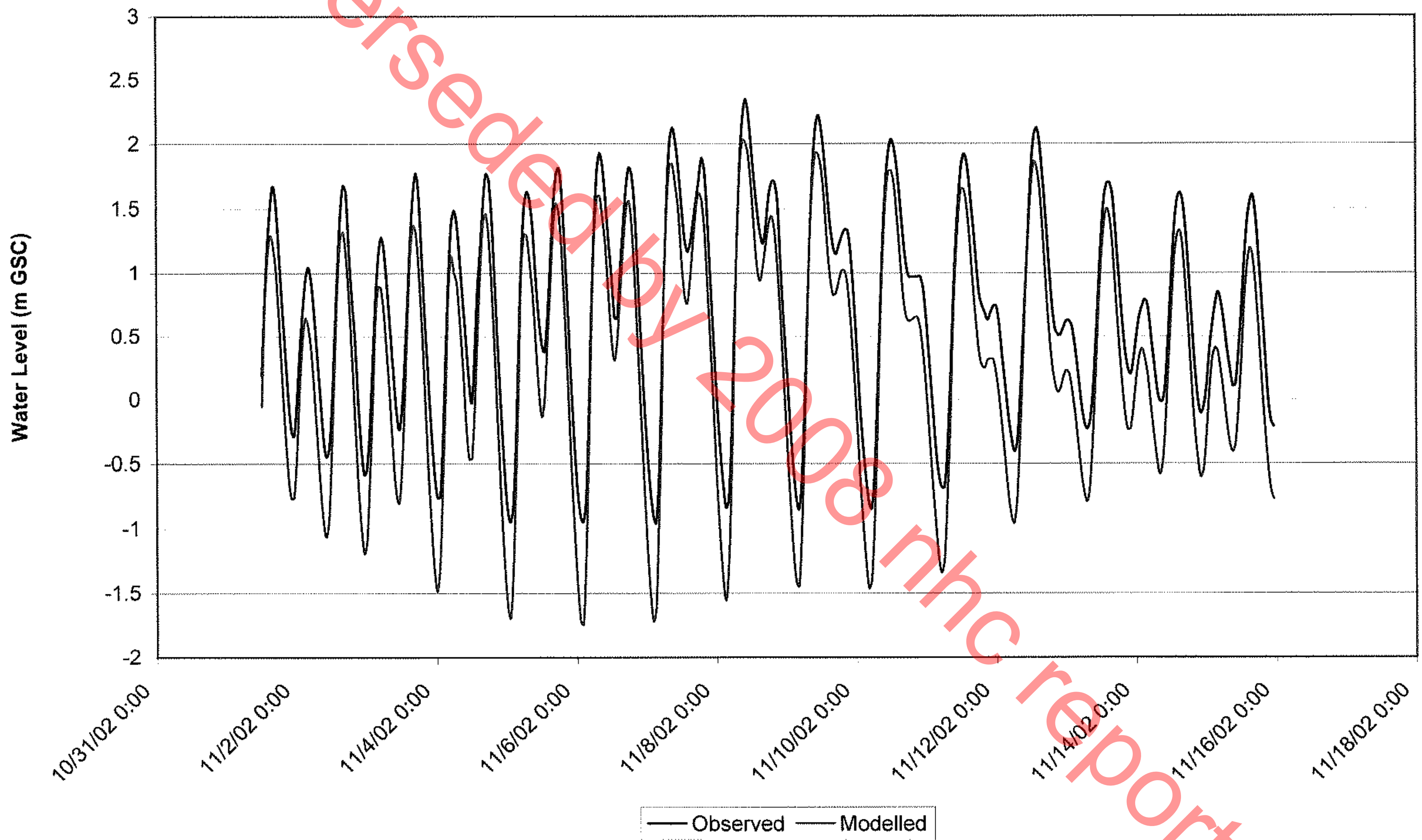


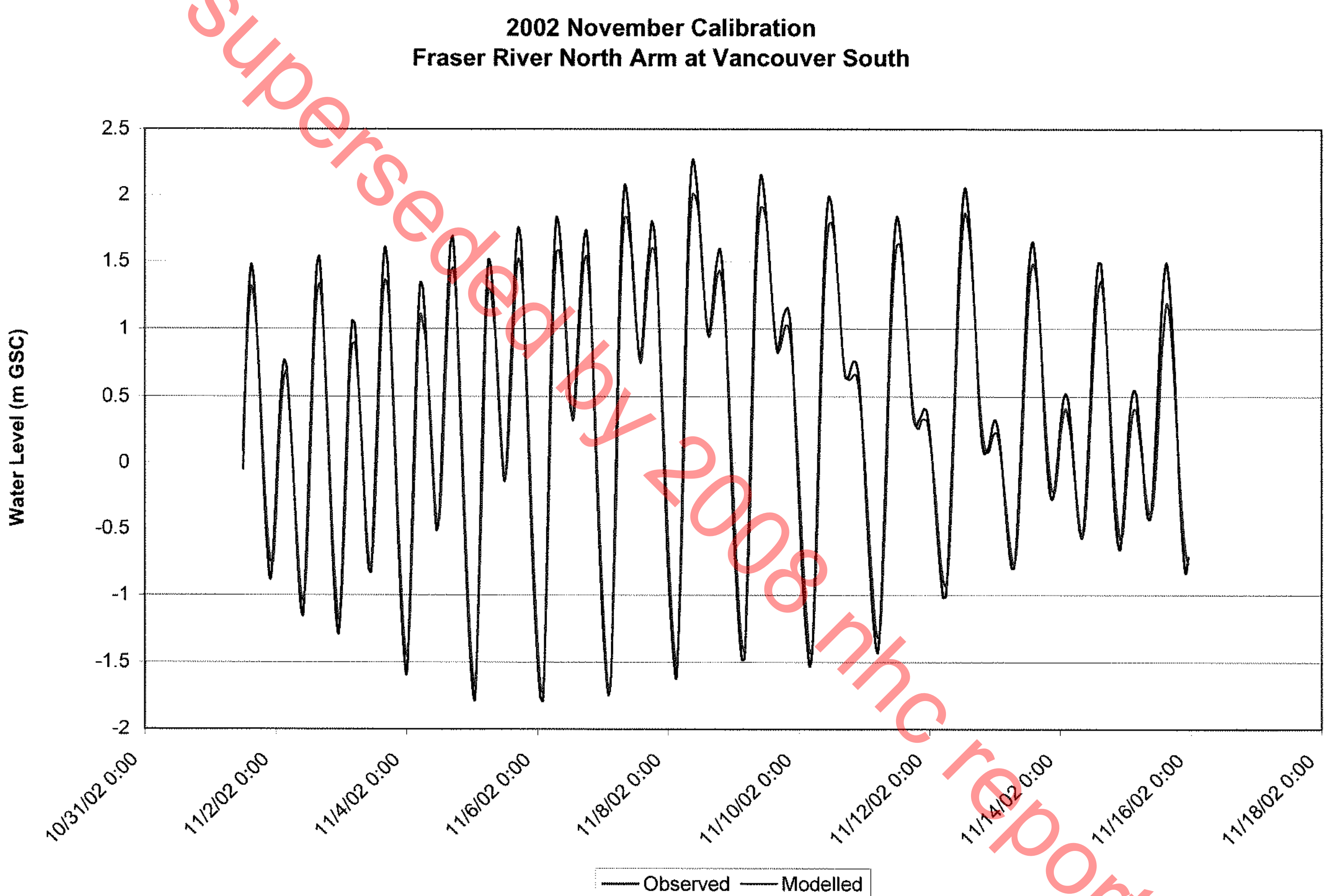

2002 November Calibration
Fraser River at Nelson Road
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled


2002 November Calibration
Fraser River at No. 6 Road
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

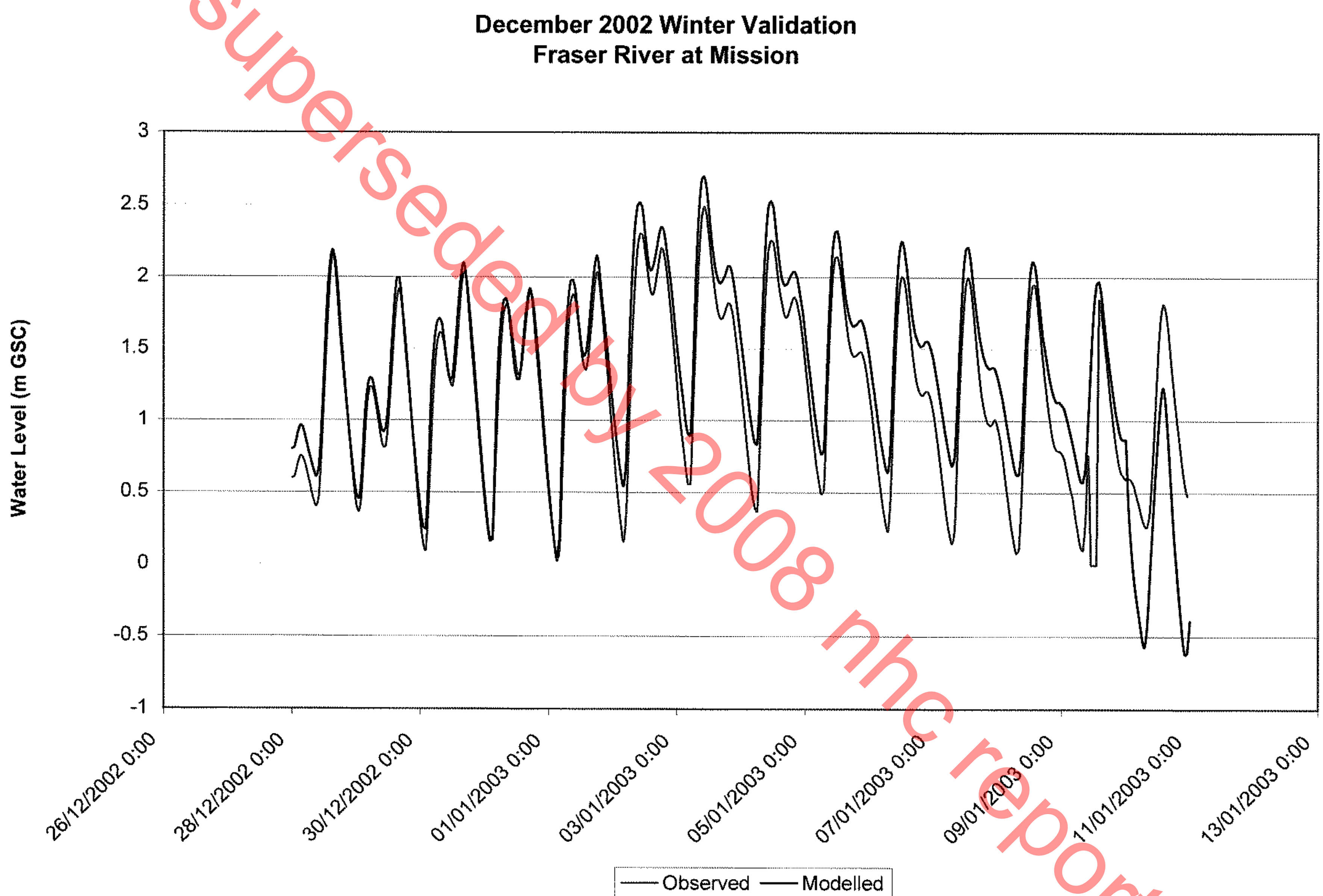

2002 November Calibration
Fraser River Steveston
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
-2.5
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

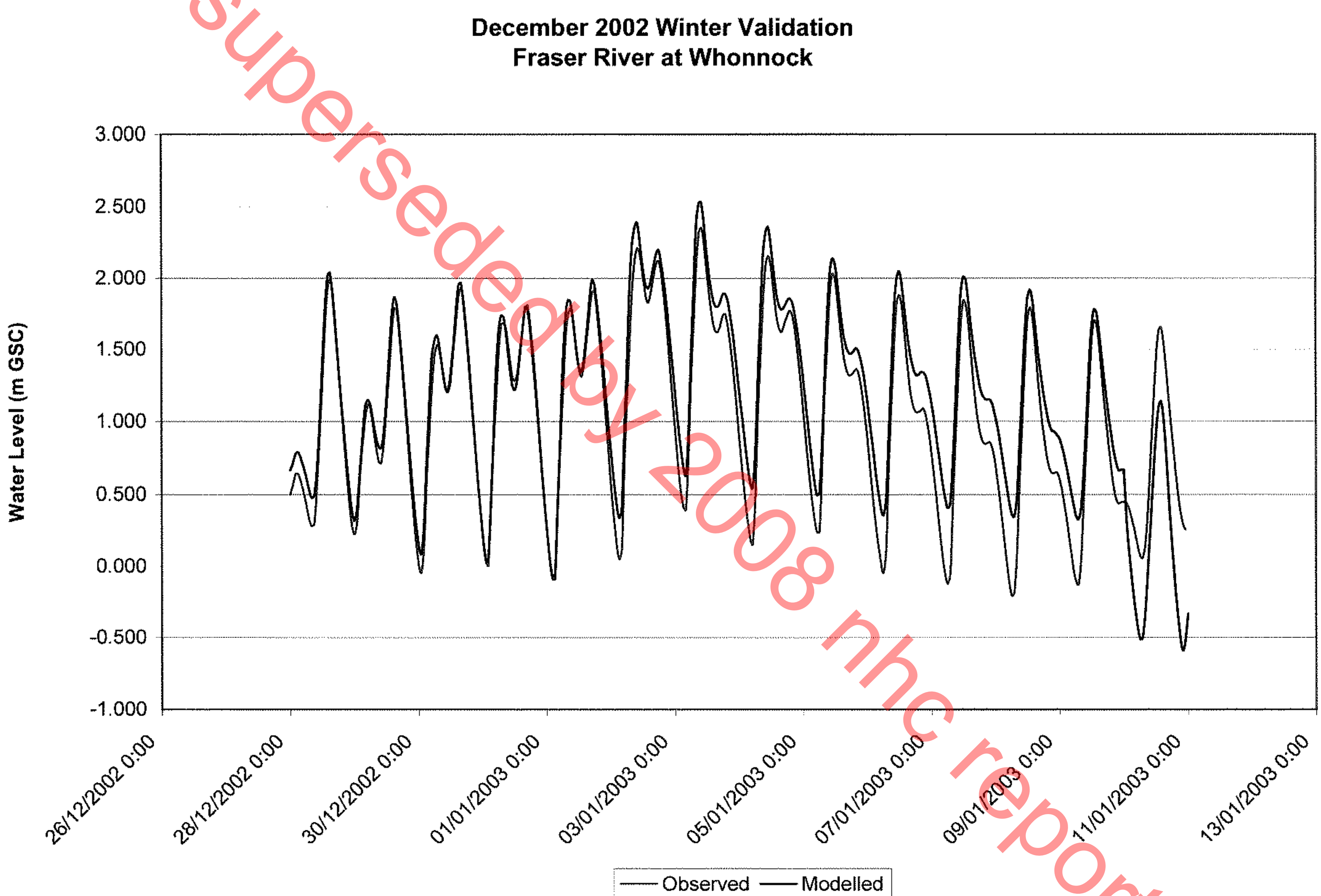
2002 November Calibration
Fraser River North Arm at Bathslough

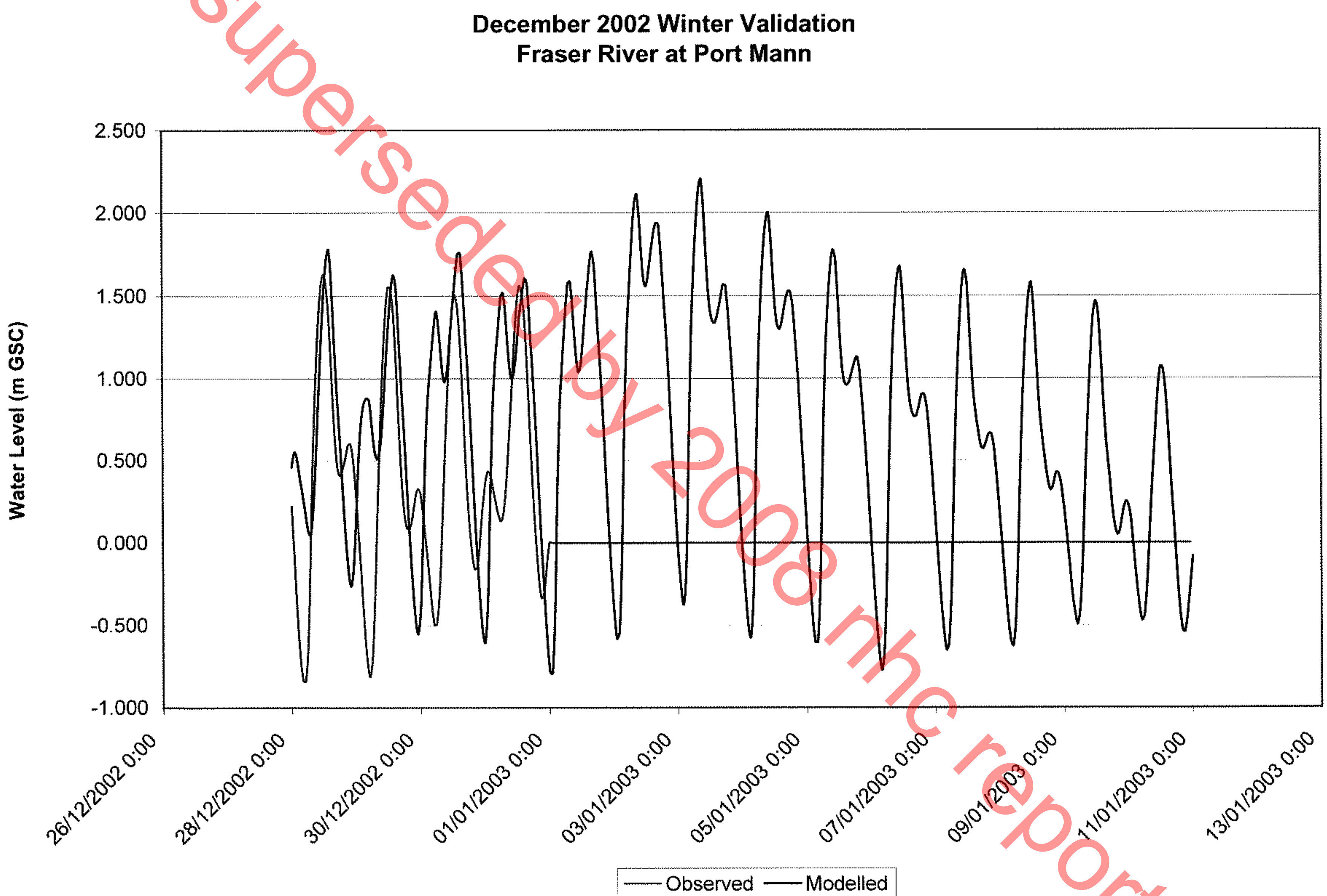




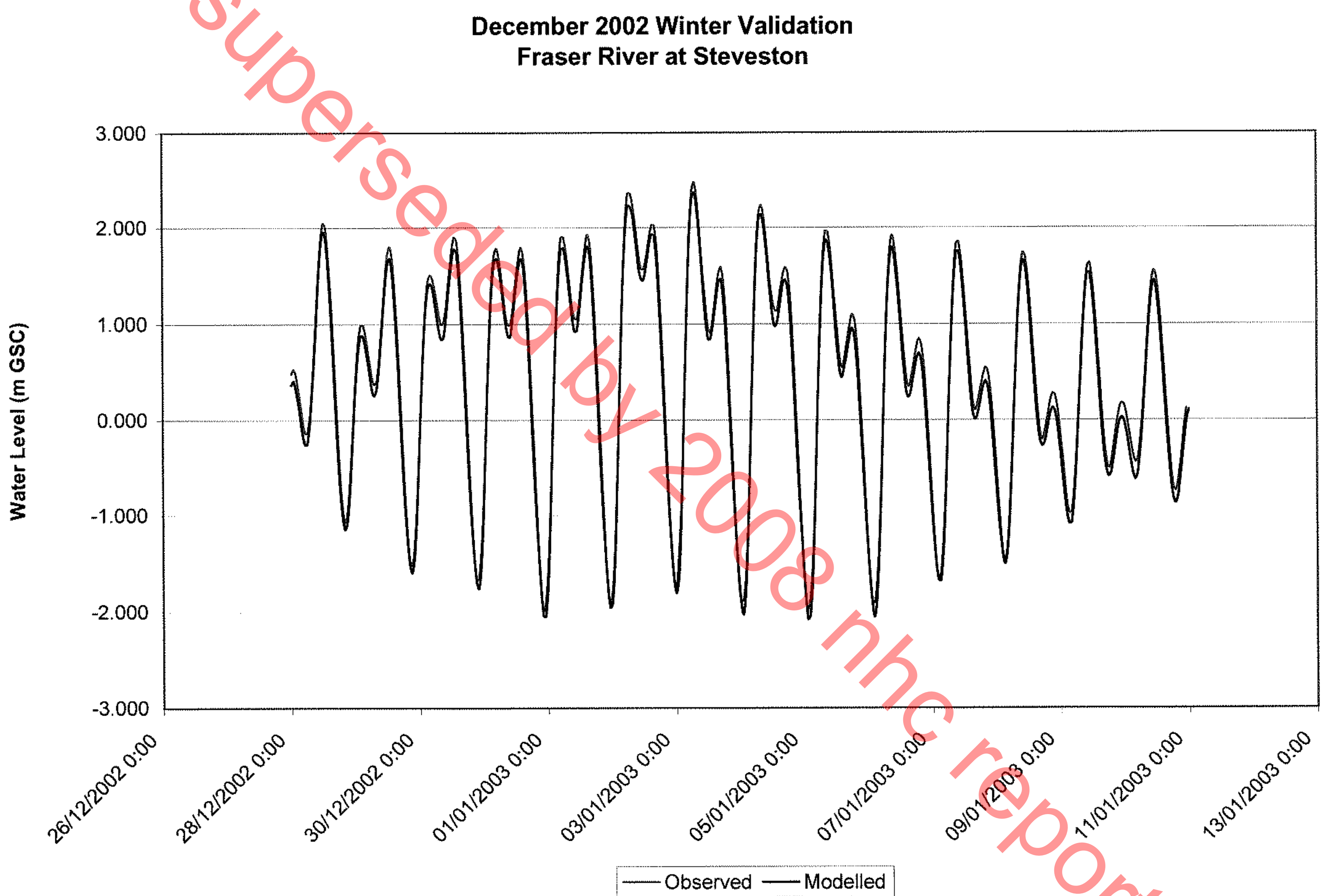
2002 November Calibration
Fraser River North Arm at Vancouver South
Water Level (m GSC)
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
-2
10/31/02 0:00
11/2/02 0:00
11/4/02 0:00
11/6/02 0:00
11/8/02 0:00
11/10/02 0:00
11/12/02 0:00
11/14/02 0:00
11/16/02 0:00
11/18/02 0:00
Observed
Modelled

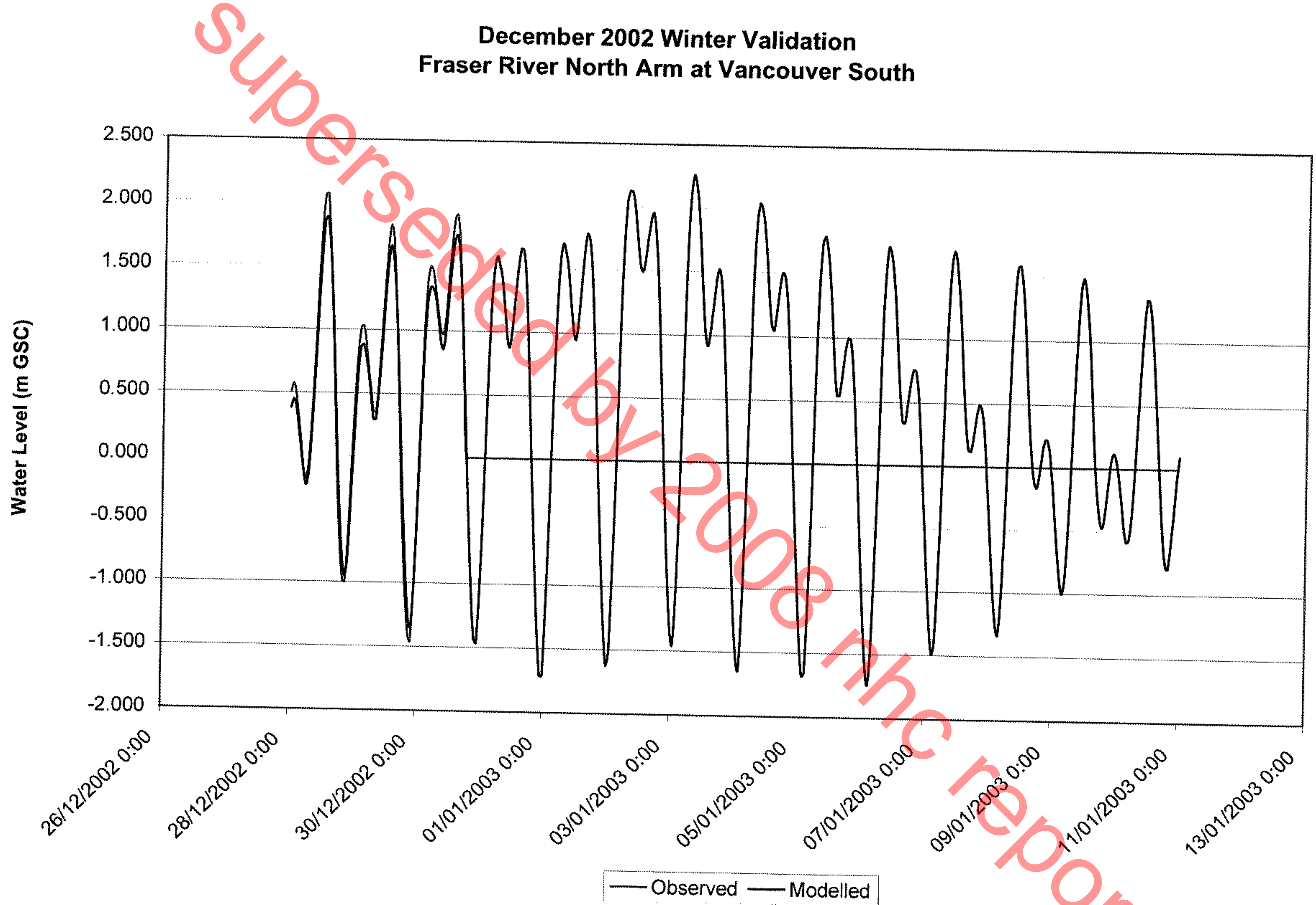

December 2002 Winter Validation
Fraser River at Mission
Water Level (m GSC)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
26/12/2002 0:00
28/12/2002 0:00
30/12/2002 0:00
01/01/2003 0:00
03/01/2003 0:00
05/01/2003 0:00
07/01/2003 0:00
09/01/2003 0:00
11/01/2003 0:00
13/01/2003 0:00
Observed
Modelled


December 2002 Winter Validation
Fraser River at Whonnock
Water Level (m GSC)
3.000
2.500
2.000
1.500
1.000
0.500
0.000
-0.500
-1.000
26/12/2002 0:00
28/12/2002 0:00
30/12/2002 0:00
01/01/2003 0:00
03/01/2003 0:00
05/01/2003 0:00
07/01/2003 0:00
09/01/2003 0:00
11/01/2003 0:00
13/01/2003 0:00
Observed
Modelled



December 2002 Winter Validation
Fraser River at Port Mann
Water Level (m GSC)
2.500
2.000
1.500
1.000
0.500
0.000
-0.500
-1.000
26/12/2002 0:00
28/12/2002 0:00
30/12/2002 0:00
01/01/2003 0:00
03/01/2003 0:00
05/01/2003 0:00
07/01/2003 0:00
09/01/2003 0:00
11/01/2003 0:00
13/01/2003 0:00
Observed
Modelled


December 2002 Winter Validation
Fraser River at Steveston
Water Level (m GSC)
3.000
2.000
1.000
0.000
-1.000
-2.000
-3.000
26/12/2002 0:00
28/12/2002 0:00
30/12/2002 0:00
01/01/2003 0:00
03/01/2003 0:00
05/01/2003 0:00
07/01/2003 0:00
09/01/2003 0:00
11/01/2003 0:00
13/01/2003 0:00
Observed
Modelled


December 2002 Winter Validation
Fraser River North Arm at Vancouver South
Water Level (m GSC)
2.500
2.000
1.500
1.000
0.500
0.000
-0.500
-1.000
-1.500
-2.000
26/12/2002 0:00
28/12/2002 0:00
30/12/2002 0:00
01/01/2003 0:00
03/01/2003 0:00
05/01/2003 0:00
07/01/2003 0:00
09/01/2003 0:00
11/01/2003 0:00
13/01/2003 0:00
Observed
Modelled

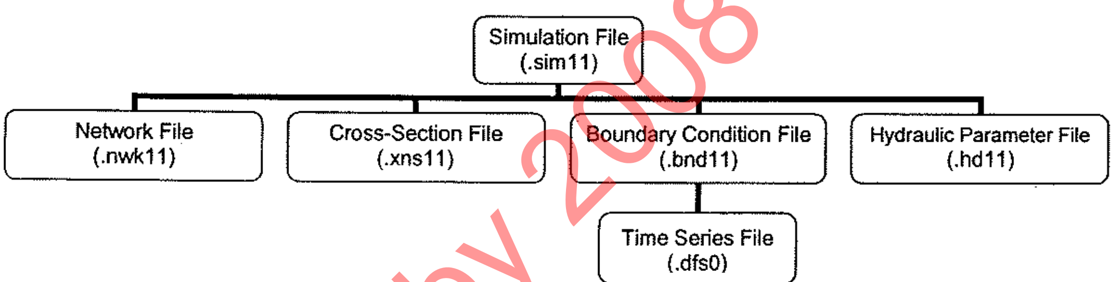
APPENDIX H
MODEL OPERATION MANUAL

Using Mike11 and MikeVIEW for the Lower Fraser River Model

This appendix briefly describes how to edit, run and view results of the Lower Fraser Mike11 models. It includes basic instructions for both the **nhc** model (Ocean to Sumas Mountain) and the combined model that includes the UMA model and extends from the ocean to near Hope. Two programs are required to create, edit and view 1-D model results: Mike11 and MikeVIEW.

MIKE11

A Mike11 model is formed of 4 distinct input files, which are combined in a simulation file. The Lower Fraser River model has numerous input files which are configured in various ways to assess different scenarios. Each of the file types is described below. Individual files created for the Lower Fraser River model project are also described.



Network File (.nwk11)

The network file describes the planform of the model. It includes information on the channel network including river and reach names and junctions. Structure information, such as the hydraulic geometry of bridges is also found in this file.

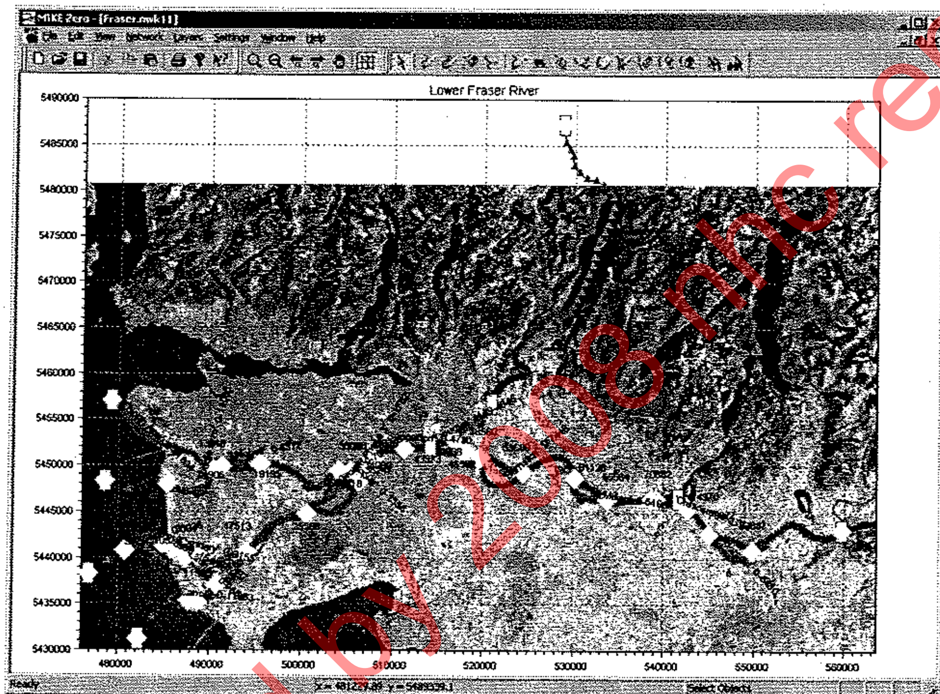
The Lower Fraser River model uses only one network file (Table 1), and the combined model uses a separate network file.

Table 1: Lower Fraser River Model Network Files

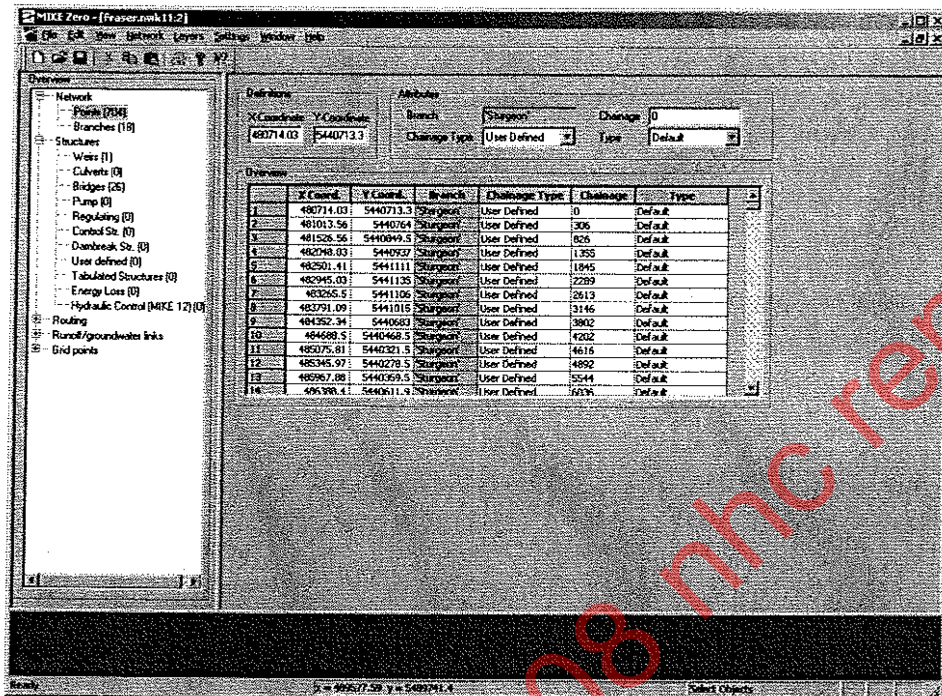
Filename	File Description	File Use
Fraser.nwk11	Universal network Ocean to Sumas Mountain	All model scenarios
Combined.nwk11	Universal network Ocean to Hope	Forecasting with flows at Hope

To look at or edit the network file:

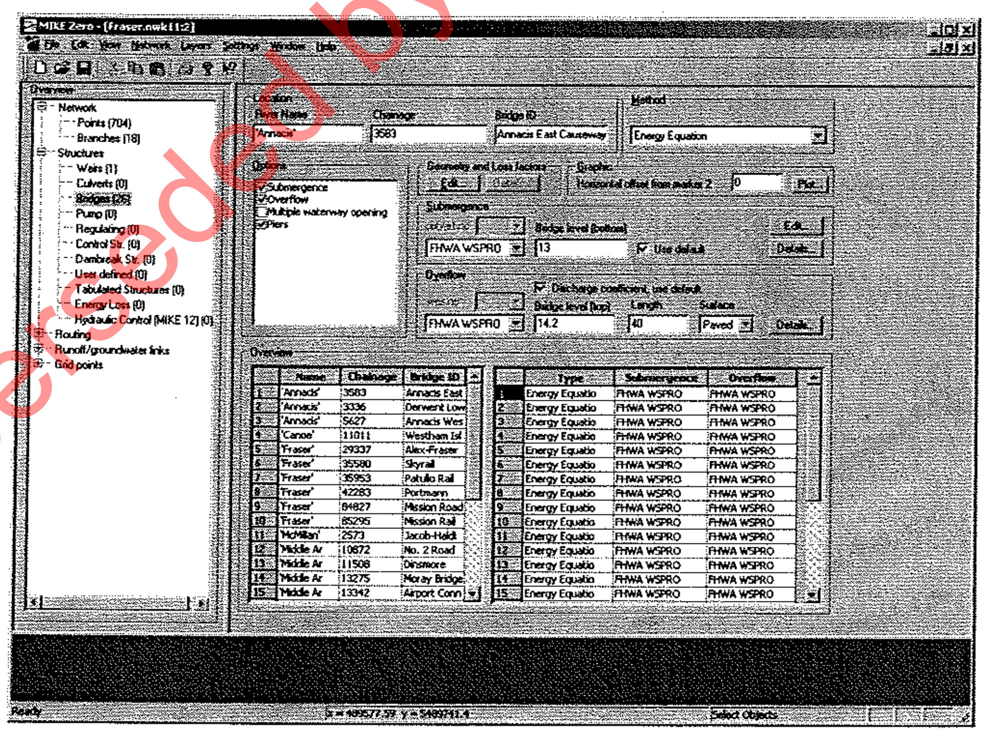
1. Open the network file by either going to File|Open in MikeZero and selecting the appropriate network file, or by clicking on the "Edit" button next to the network file on the input window of the simulation file.
2. A screen showing the network will appear. The background image can be turned on or off by going to Layers| Add/Remove Layers| Overlay Manager.



3. Any edits to the network should be made in the Tabular View, which is accessed by going to View| Tabular View. No changes are expected to be required to this file.



4. Bridges or other structures can be edited or added through the bridge menu in the tabular view. Please refer to the dhi Mike11 documentation for details regarding the input of bridges or other structures. The existing model contains 26 bridges that can be used as templates for any new bridges.



Cross-Section File (.xns11)

The cross-section file describes the geometry of the individual channels within the model. It includes information on the channel shape, the location of the banks and dikes, and the relative channel roughness.

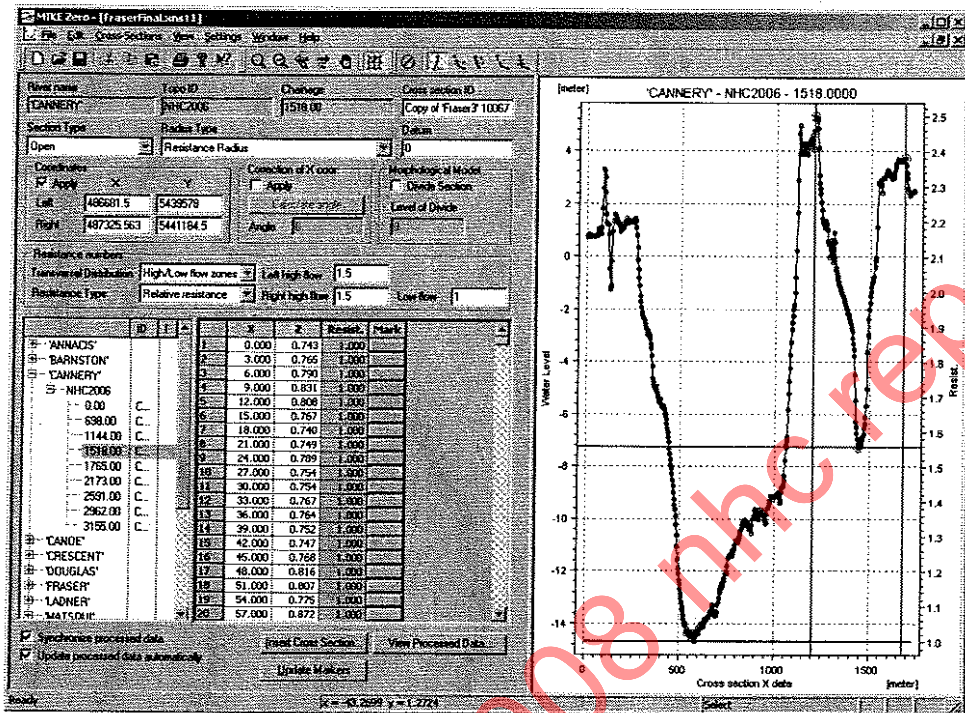
The Lower Fraser Model includes 7 cross-section files, which describe different geometric scenarios (Table 2).

Table 2: Lower Fraser River Model Cross-Section Files

Filename	File Description	File Use
Fraser.xns11	Base 2006 geometry	Most model runs
FraserScour.xns11	Base 2006 geometry with local scour between Whonnock and Mission	Scour sensitivity run
FraserInfill0to11.xns11	Base 2006 geometry with infill on mainstem between km 0 and km 11,	Dredging model scenarios
FraserInfill0to27pt5.xns11	Base 2006 geometry with infill on mainstem between km 0 and km 27.5	Dredging model scenarios
FraserInfill0to35pt5.xns11	Base 2006 geometry with infill on mainstem between km 0 and km 35.5	Dredging model scenarios
FraserInfill0to40.xns11	Base 2006 geometry with infill on mainstem between km 0 and km 40	Dredging model scenarios
Combined.xns11	Geometry for Ocean to Hope	Forecast model with flows at Hope

To look at or edit the cross-section file:

1. Open the network file by either going to File|Open in MikeZero and selecting the appropriate cross-section file, or by clicking on the "Edit" button next to the cross-section file on the input window of the simulation file.



2. Individual cross-sections can be viewed or edited by selecting the appropriate section from the menu tree on the bottom left-hand corner.
3. Please refer to the dhi Mike11 user manual for instructions on editing the cross-section files. No changes are expected to be required to these files in order to run the model in forecast mode.

Boundary Conditions File (.bnd11)

The boundary conditions file describes the flow and water level conditions at the model boundaries.

The Lower Fraser Model includes numerous boundary condition files, which describe different input scenarios (Table 3).

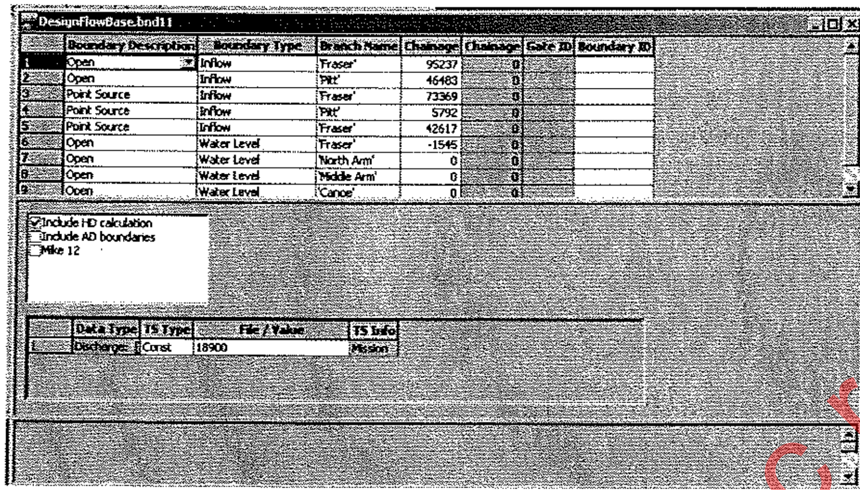
Table 3: Lower Fraser River Model Boundary Condition Files

Filename	File Description	File Use
2002Calibration.bnd11	2002 observed conditions	Freshet calibration
1999Validation.bnd11	1999 observed conditions	Freshet validation
1997Validation.bnd11	1997 observed conditions	Freshet validation
2002NovCalib.bnd11	2002 observed conditions	Winter calibration
2002DecValid.bnd11	2002 observed conditions	Winter validation
1997WinterValid.bnd11	1997 observed conditions	Winter validation
DesignFlowBase.bnd11	Design inflows, 2002 ocean levels	Freshet design run

DesignFlowBase200YearTribs	Increased tributary inflows, 2002 ocean levels	Freshet flow sensitivity
DesignFlow*****.bnd11	Varying mainstem inflows, base tributary inflows, 2002 ocean levels	Freshet flow sensitivity
DesignFlowOcean+10%swing	Design inflows, increased swing on ocean levels	Freshet ocean level sensitivity
WinterDesignBase.bnd11	Design inflows, design water levels	Winter design run
WinterDesignBaseGW.bnd11	Design inflows, global warming scenario water levels	Winter ocean level sensitivity
WinterDesign20Year.bnd11	Reduced inflows, design water levels	Winter flow sensitivity
WinterDesign20YearGW.bnd11	Reduced inflows, global warming scenario water levels	Winter flow/ocean level sensitivity
WinterDesign100Year.bnd11	Reduced inflows, design water levels	Winter flow sensitivity
WinterDesign100YearGW.bnd11	Reduced inflows, global warming scenario water levels	Winter flow/ocean level sensitivity
Combined.bnd11	Design inflow at Hope, 2002 ocean levels	Forecast model with flows at Hope

To look at or edit the boundary condition file:

1. Open the hydraulic parameter file by either going to File|Open in MikeZero and selecting the appropriate hydraulic parameter file, or by clicking on the "Edit" button next to the hydraulic parameter file on the input window of the simulation file.
2. The following screen will open.



3. The Lower Fraser River model includes 9 boundaries, each of which requires inputs. There are 5 inflow inputs and 4 water level inputs (Table 4). The combined UMA and **nhc** model includes additional upstream boundaries that are described in the original UMA model documentation.

Table 4: Lower Fraser River Model Boundaries

Boundary Type	Location	Input Number
Inflow	Mainstem (Sumas Mountain)	1
Inflow	Pitt River	2
Inflow	Stave River	3
Inflow	Alouette River	4
Inflow	Coquitlam River	5
Water Level	North Arm at ocean	7
Water Level	Middle Arm at ocean	8
Water Level	Main Arm – North at ocean	6
Water Level	Main Arm – South at ocean	9

4. Individual boundaries can be edited by selecting a boundary from the top table and changing the values in the table at the bottom of the screen. The MIKE11 interface can sometimes by finicky; it is often necessary to click over boxes more than once, and it is important to click the screen outside the fields where changes were made before closing the screen.
5. Inputs are either constant or time-series. Constant values can be edited directly in the boundary file by clicking on the File/Value field. Time-series are linked to separate time-series files (.dfs0) in the boundary file. The link can be changed by clicking on the "…" button adjacent to the File/Value field.

TimeSeries File (.dfs0)

The timeseries file is a database that holds water level and flow data.

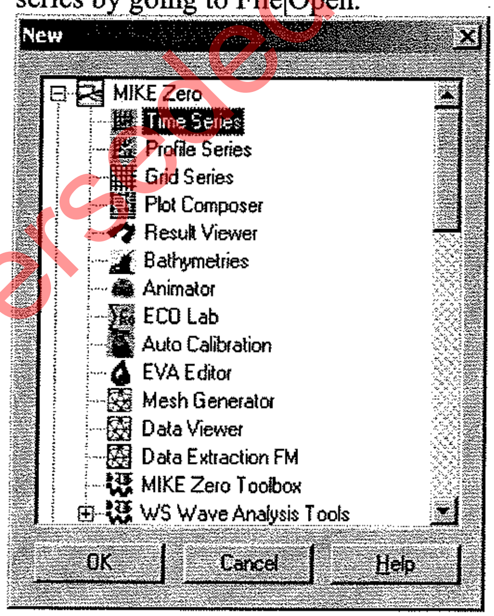
The Lower Fraser Model includes 7 timeseries files, which describe different time periods (Table 5).

Table 5: Lower Fraser River Model Timeseries Files

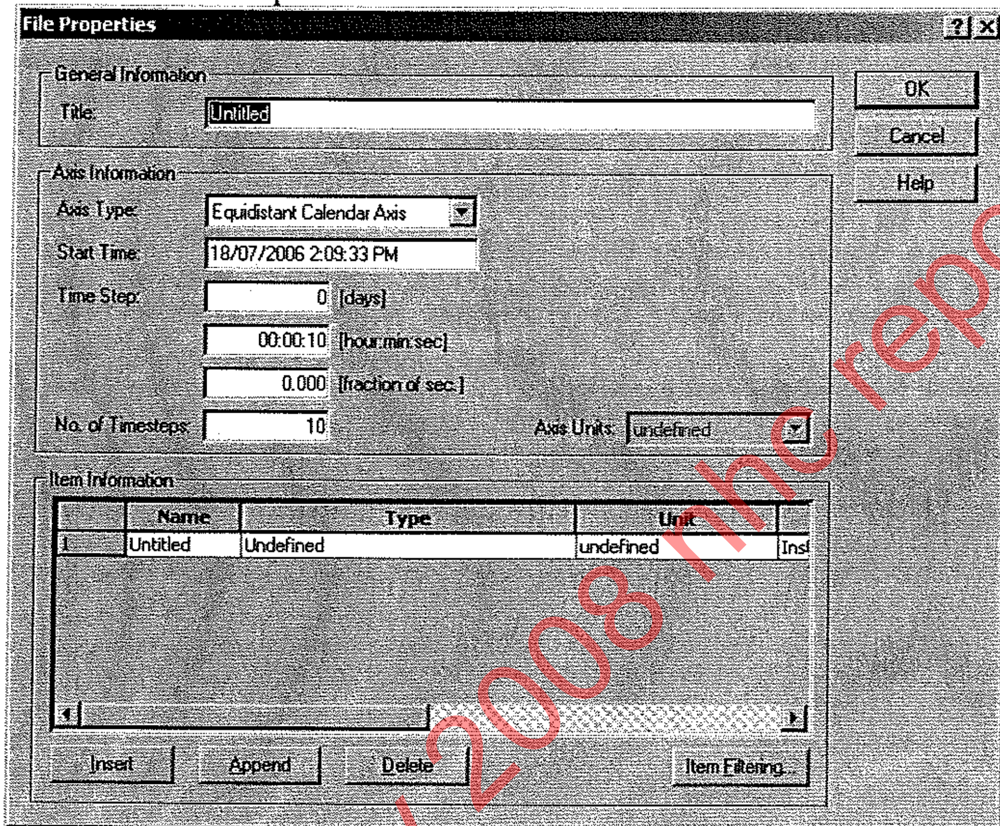
Filename	File Description	File Use
2002Calibration.dfs0	Observed inflows and water levels for Spring 2002	Freshet Calibration
1999Validation.dfs0	Observed inflows and water levels for Spring 1999	Freshet Validation
1997Validation.dfs0	Observed inflows and water levels for Spring 1997	Freshet Validation
2002NovCalib.dfs0	Observed inflows and water levels for Winter 2002	Winter Calibration
2002DecValid.dfs0	Observed inflows and water levels for Winter 2002	Winter Validation
1997WinterValid.dfs0	Observed inflows and water levels for Winter 1996-1997	Winter Validation
WinterDesign.dfs0	Predicted water levels for winter design events	Winter design runs

New timeseries files can be created by:

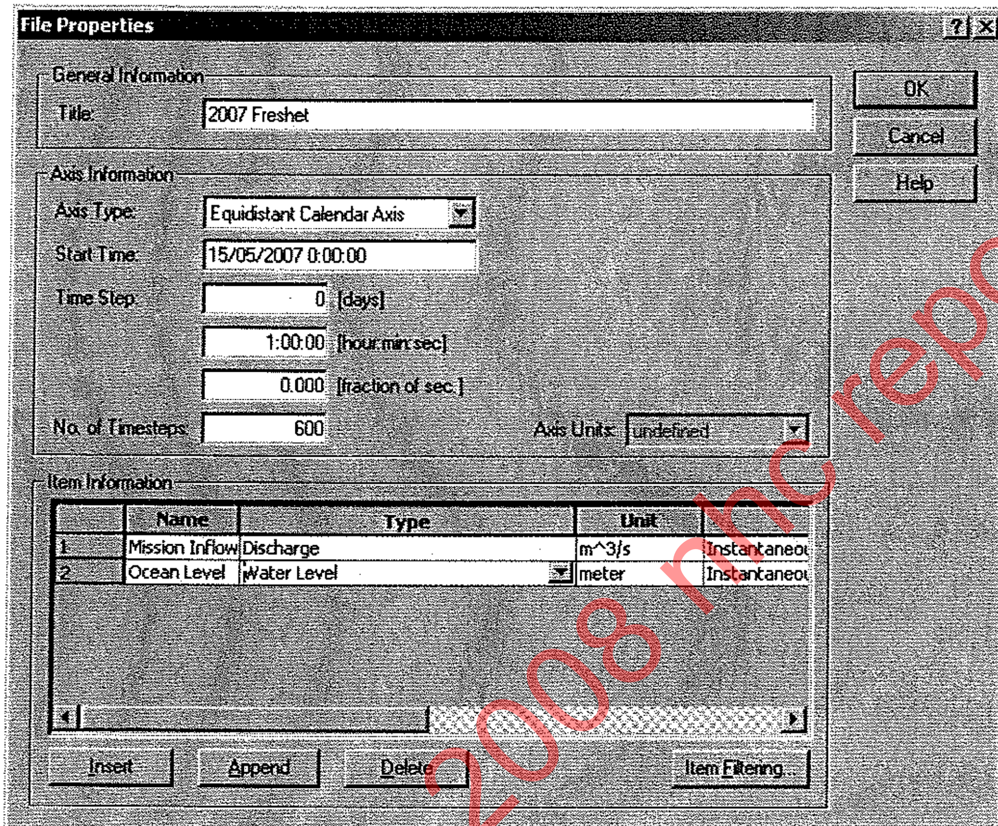
1. In MikeZero go to File|New and select *Time Series*. Then selecting *Blank TimeSeries* and clicking OK. Files can be viewed and edited by selecting a time series by going to File|Open.



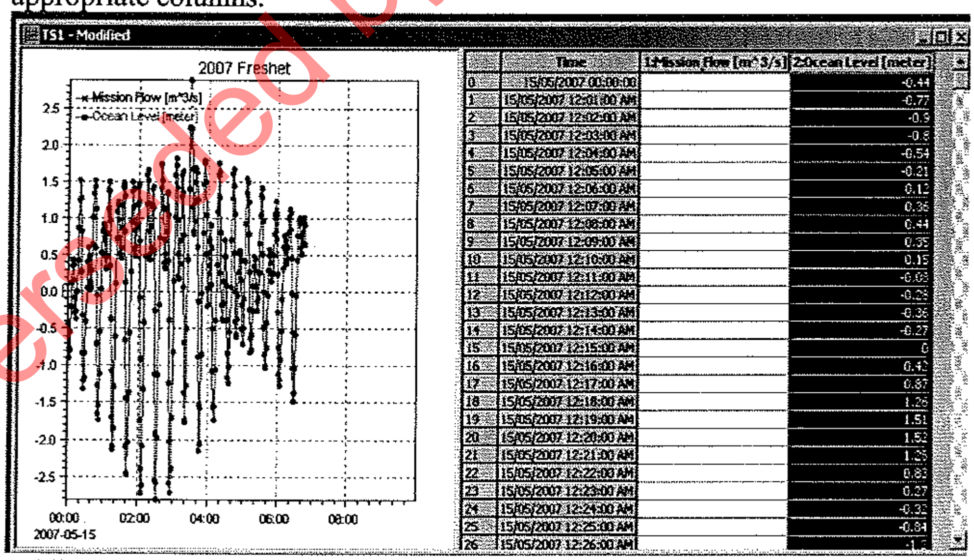
2. A new window will open.



3. Fill in the Title as appropriate, "2007 Freshet" for example.
4. Keep the default axis type, but change the start time, time steps and number of time steps based on available data.
5. Add data fields by inserting or appending new rows in the table at the bottom of the screen.
6. A new timeseries file to be used for freshet forecasting will likely include an inflow at the upstream boundary and water levels at each of the four outlet boundaries. Alternatively, a single ocean water level could be used for all four outlets.
7. Appropriate data types must be selected for each field.



8. Once the File Properties window is complete click OK.
9. You can then copy and paste an inflow hydrograph and predicted tide levels to the appropriate columns.



10. Additional fields can be added to the file by right-clicking the graph and selecting "properties". This will return you to the File Properties window.
11. When all the data is input save and close the file by going to File|Save.

12. The timeseries must then be linked into the model through the boundary condition file.

Hydraulic Parameter File (.hd11)

The hydraulic parameter file assigns hydraulic conditions to the model. It includes information on initial conditions, channel roughness and default model input parameters.

The Lower Fraser Model includes 9 hydraulic parameter files, which describe different scenarios (Table 6).

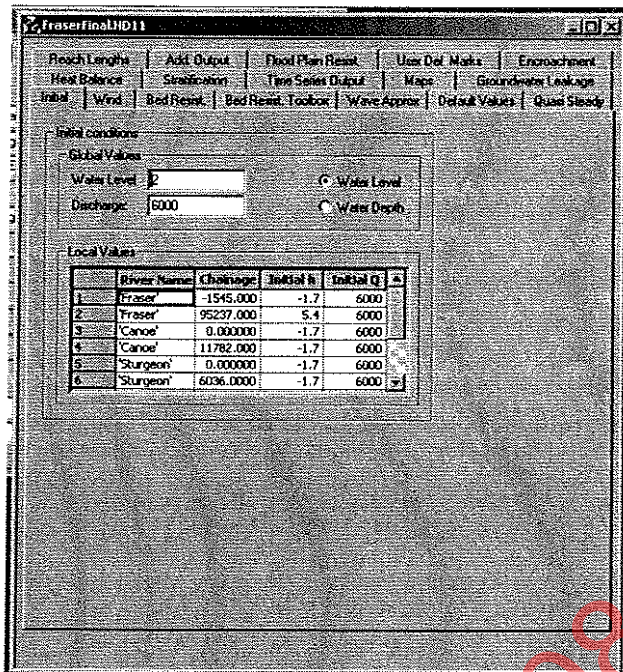
Table 6: Lower Fraser River Model Hydraulic Parameter Files

Filename	File Description	File Use
FraserVarRough.hd11	Base variable roughness file*	Most scenarios
Frasern027DougtoMission	Base calibration downstream of Douglas Island, n=0.027 Douglas Island to Mission	Flows greater than 15500 m^3/s
Frasern028DougtoMission	Base calibration downstream of Douglas Island, n=0.028 Douglas Island to Mission	Flows between 14000 m^3/s and 15500 m^3/s
Frasern029DougtoMission	Base calibration downstream of Douglas Island, n=0.029 Douglas Island to Mission	Flows between 12500 m^3/s and 14000 m^3/s
FraserBasen	Base Calibration	Flows less than 12500 m^3/s
Fraser-10%n.hd11	10 % reduced roughness file	Roughness sensitivity
Fraser+10%n.hd11	10% increased roughness file	Roughness sensitivity
Fraser+20%n.hd11	20% increased roughness file	Roughness sensitivity
Combined.hd11	HD parameters for Ocean to Hope	Forecast model with flows at Hope

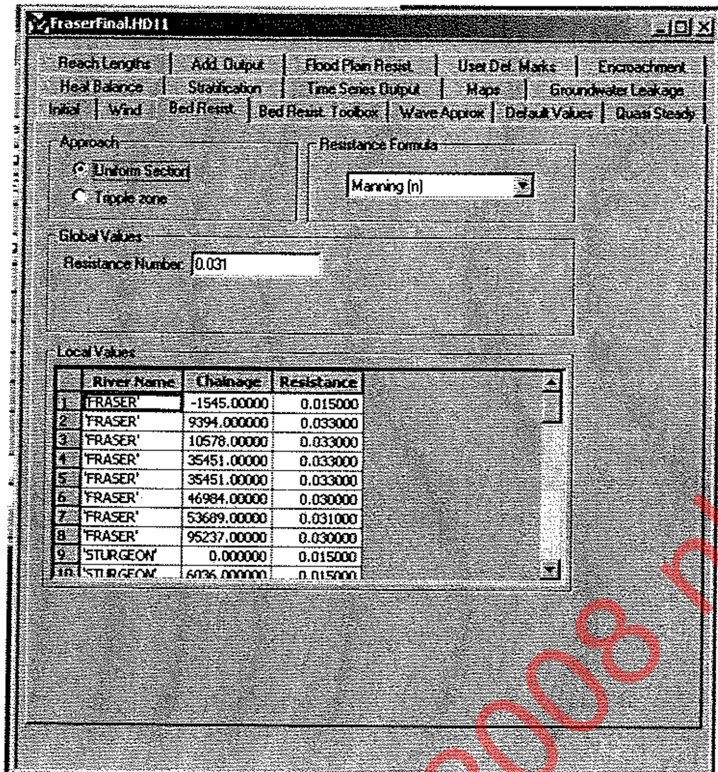
*Includes variable roughness parameters for Douglas Island to Mission. DHI's current version of the MIKE11 software does not properly support this feature. DHI aims to correct the software for later versions, and this HD11 file could be used. However, this file has not been fully checked and should be used with caution.

To look at or edit the hydraulic parameter file:

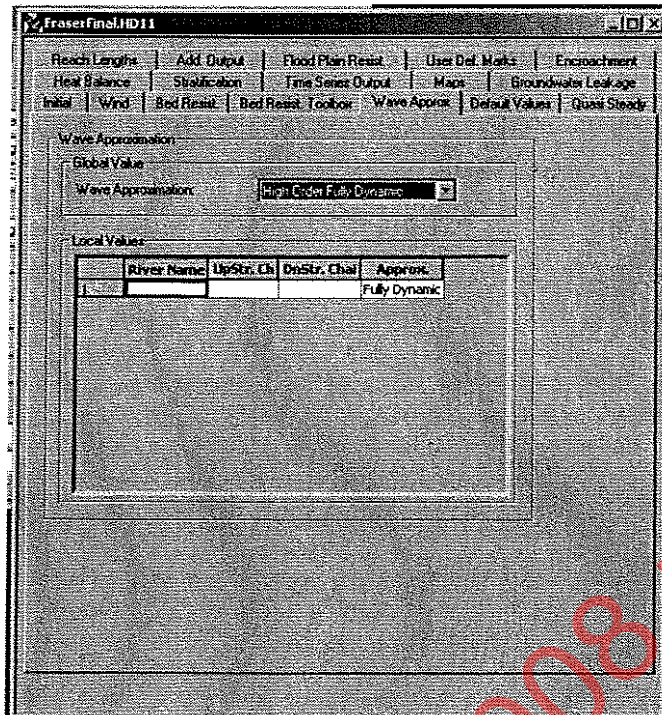
1. Open the hydraulic parameter file by either going to File|Open in MikeZero and selecting the appropriate hydraulic parameter file, or by clicking on the "Edit" button next to the hydraulic parameter file on the input window of the simulation file.



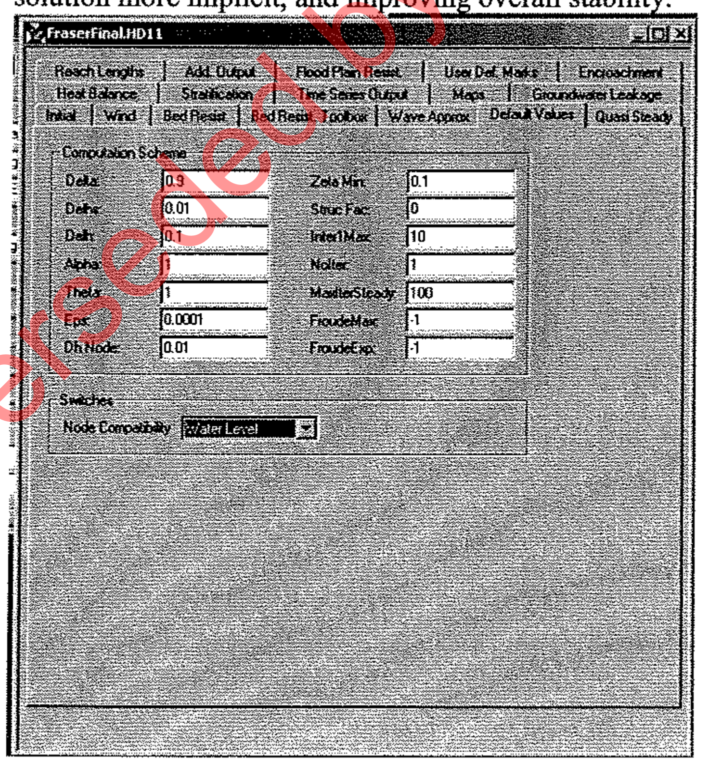
2. This file includes numerous types of input that are accessed by clicking on the appropriate tabs. Only certain of these tabs are relevant to the Lower Fraser River model.
3. The "Initial" tab screen includes information on the initial water levels and discharges. These values may have to be adjusted to better reflect estimated initial conditions if the boundary conditions of the model are changed significantly.
4. The "Bed Resist." tab screen includes information on the Manning's n roughness assigned to different sections of the model. The Global Value is the roughness assigned to all reaches not specifically described in the local values window. In the Lower Fraser River model, all roughnesses are defined in the local values window. These values were assigned based on careful calibration and generally should not be changed. No changes are expected to be required to this file in order to run the model in forecast mode.



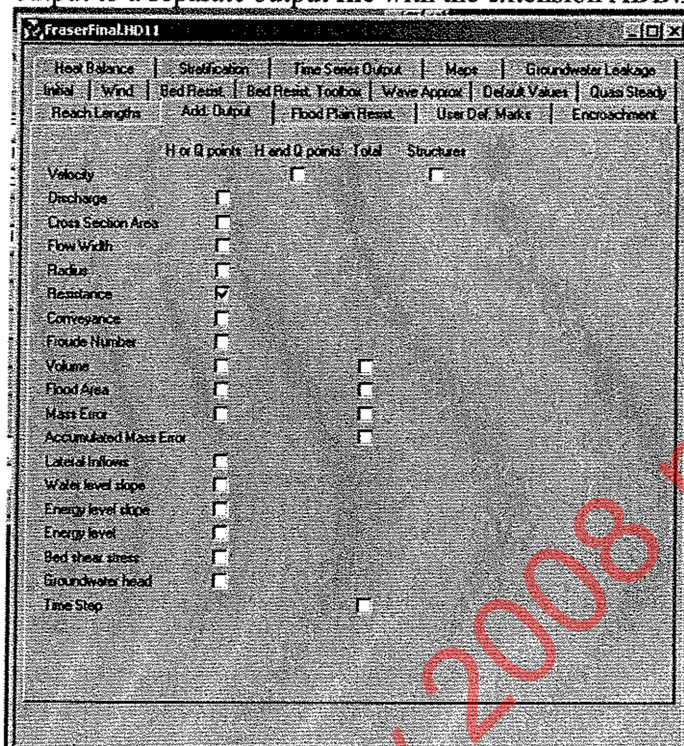
5. The "Wave Approx" tab screen describes the algorithm type used to reach a solution. For the Lower Fraser Model the *High Order Fully Dynamic* method has been used for all parts of the model. When using the combined model, some reaches of the upstream model use the simpler wave approximation method, please see UMA's original documentation for details.



6. The "Default Values" tab includes input boxes for various parameters used in the solution of the model. For the Lower Fraser Model, default values were used in all cases except for *Delta*, where a value of 0.9 was substituted, making the model solution more implicit, and improving overall stability.



7. The "Add. Output" tab inputs are used to get the model to output additional information beyond water levels and discharges. These additional results are output to a separate output file with the extension ADD.RES11.



Simulation File (.sim11)

The Mike11 model is controlled by the simulation file. This file calls all the relevant input files and includes information on simulation periods.

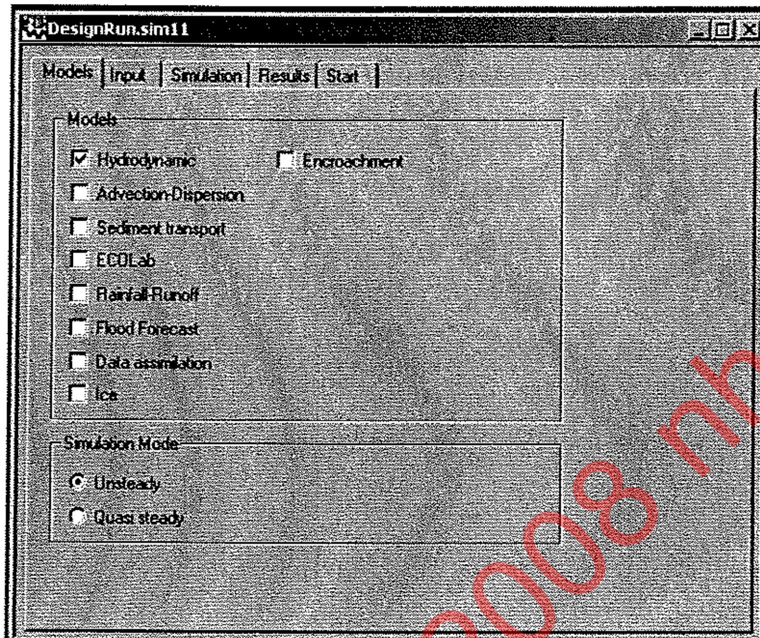
The Lower Fraser Model includes 8 simulation files, which describe different scenarios (Table 7).

Table 7: Lower Fraser River Model Simulation Files

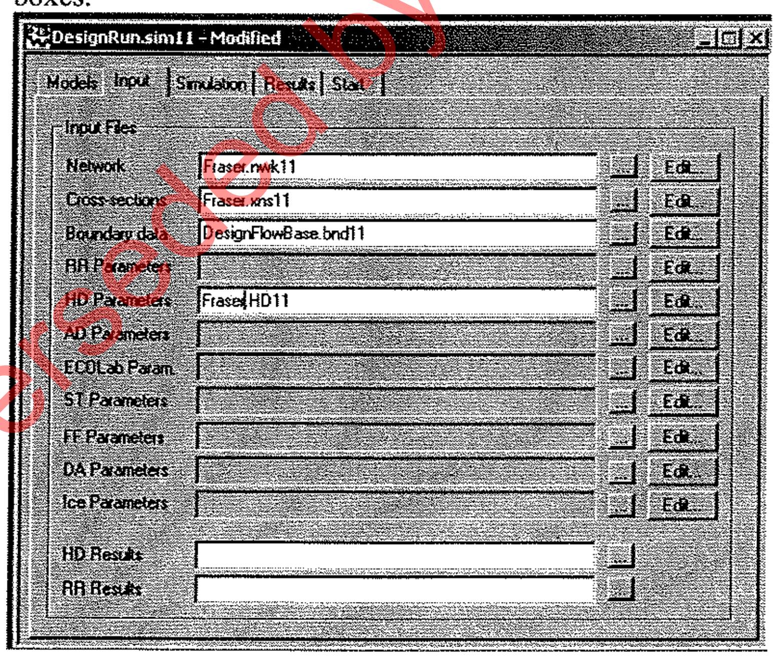
Filename	File Description	File Use
2002Calibration.sim11	2002 Calibration	Freshet Calibration
1999Validation.sim11	1999 Validation	Freshet Validation
1997Validation.sim11	1997 Validation	Freshet Validation
2002DecValid.sim11	2002 Calibration	Winter Calibration
2002NovValid.sim11	2002 Validation	Winter Validation
1997DecValid.sim11	1997 Validation	Winter Validation
FreshetRun.sim11	Freshet Design	Freshet Design Runs
WinterRun.sim11	Winter Design	Winter Design Runs
Combined.sim11	Ocean to Hope Forecast	Forecast mode with flows at Hope

To look at or edit the simulation file:

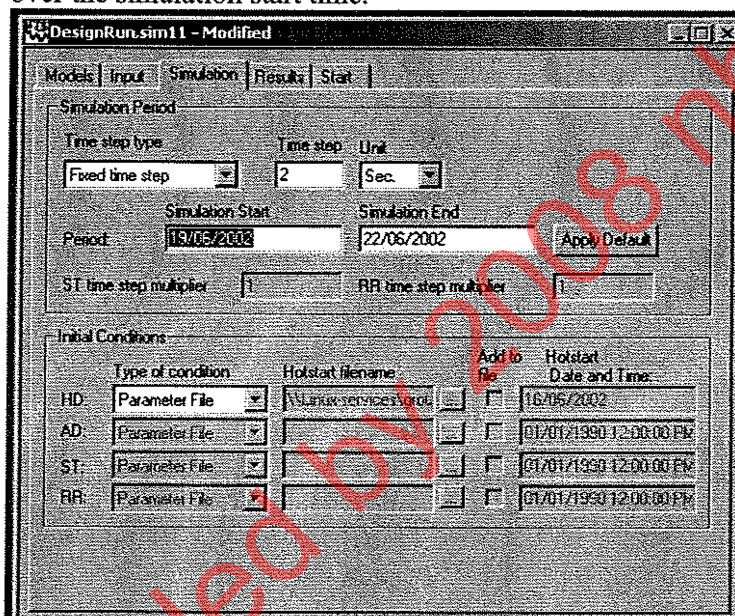
1. Open the simulation file by going to File|Open and selecting the appropriate simulation file.



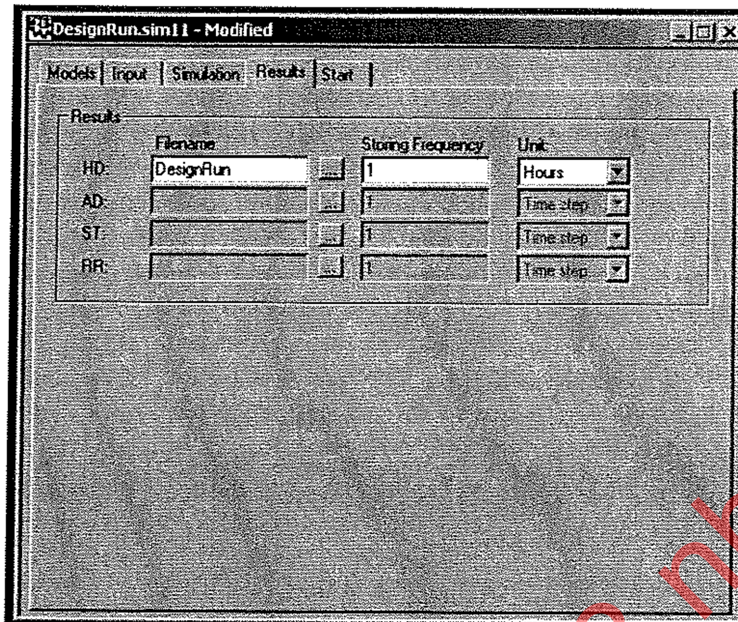
2. Click on the "Input" tab and type-in or select appropriate input files by clicking "…" for the *Network, Cross-sections, Boundary data,* and *HD Parameter* input boxes.



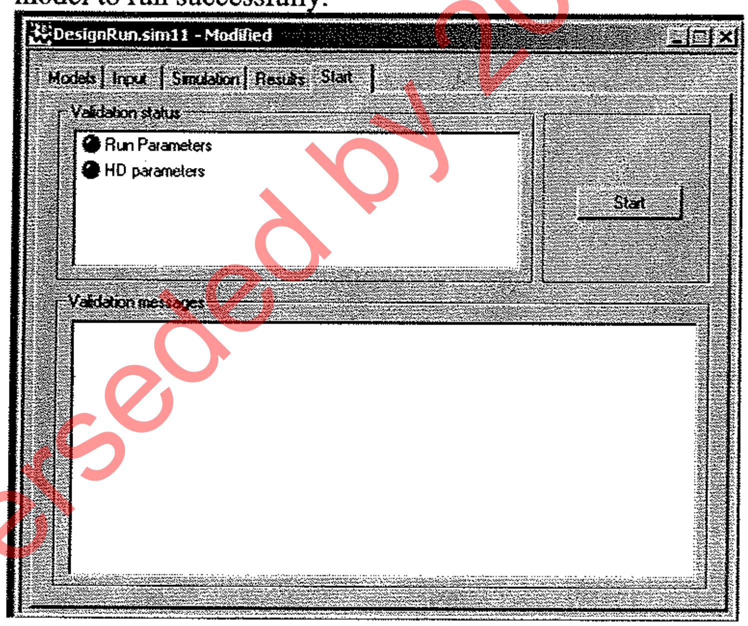
3. Each of the input files can be accessed by clicking the "Edit…" button to the right of the screen.
4. The "Simulation" tab requires inputs to define the simulation period and timestep. The Lower Fraser River Model needs to be run at a minimum of a 2 second timestep to ensure model stability. A fixed timestep is used for the Lower Fraser Model. The simulation period is dependant on available data and result requirements; a run-up period of several hours is required before getting valid results, and therefore the simulation should start 6-12 hours before model results are required. In the Lower Fraser River model initial conditions are defined in the parameter file (.hd11). A hotstart file is not required, but Mike11 results files (.RES11) can be used for initial conditions as long as the results date and time fall over the simulation start time.



5. The output file name and storing frequency is input in the "Results" tab window. A short storing frequency can result in very large, unreadable output files; a storing frequency of 1 hour is generally suitable for output.

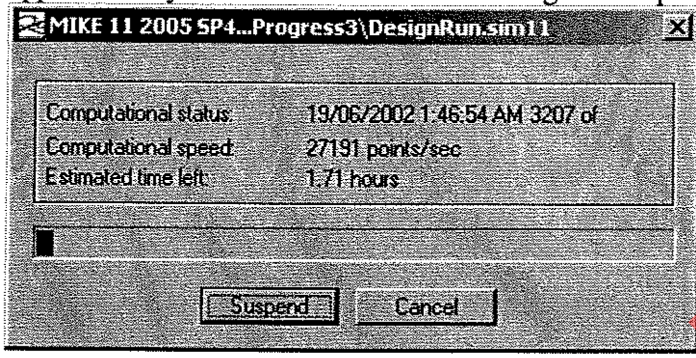


6. The model is run by clicking the *Start* button in the "Start" tab window. Two green lights for the Run Parameters and HD Parameters must be shown for the model to run successfully.



7. The Mike11.ini file must be placed in the same directory as the .sim11 file for a successful model run. This file includes information on default run methods, some of which have been adjusted for the Lower Fraser River model; these include the number of maximum iterations, the re-use of bridge hydraulic tables, and the suppression of error message boxes. None of these affects the outcome of the model, but they speed up processing time.

8. A warning message may appear which can be ignored by clicking NO. The messages include warnings that will not affect the model run.
9. A final window will appear showing the model calculation progress. It can be closed once the model run is complete. A week-long simulation will take approximately 45 minutes to run on an average desktop computer.

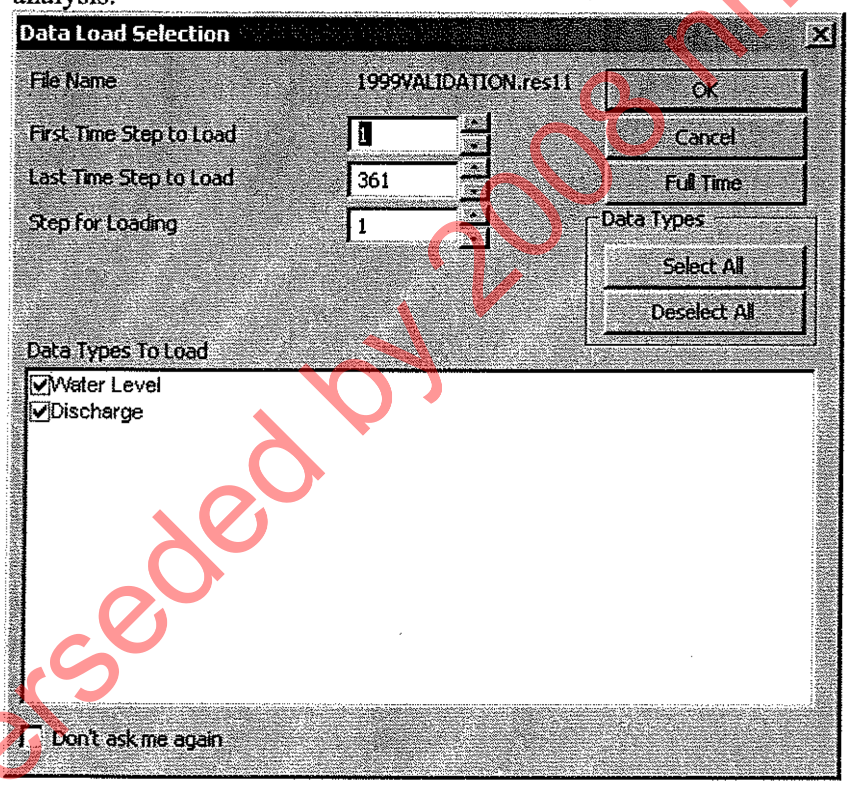


10. Mike11 will create a results file (.RES11) that is stored in the project directory or under the directory specified on the "Results" tab, which can be viewed in MikeView.

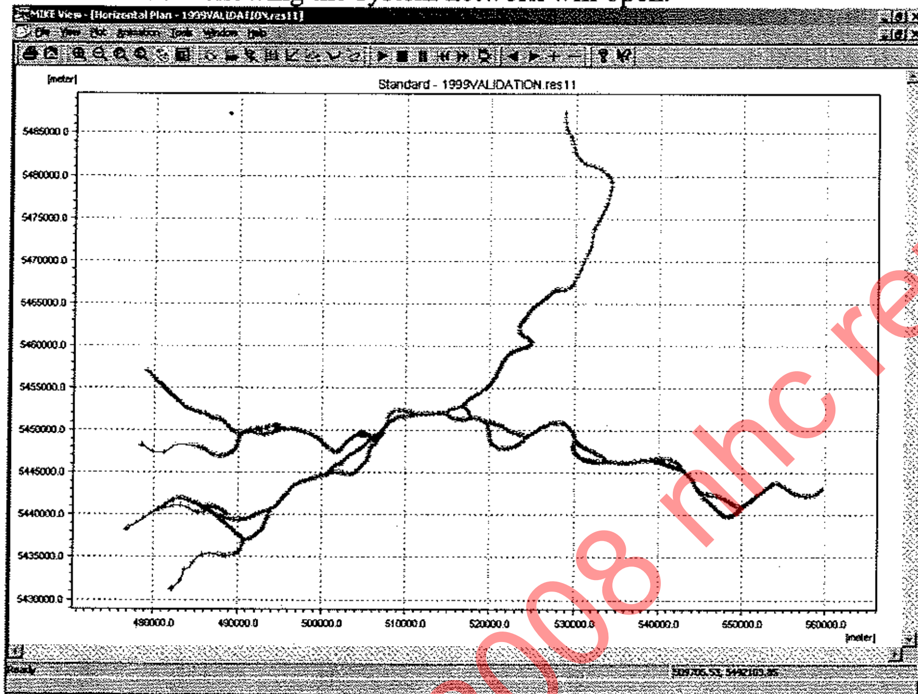
MikeView

MikeView is a program that is used to view the results of numerous dhi models including Mike11. To view the results of the Lower Fraser River model:

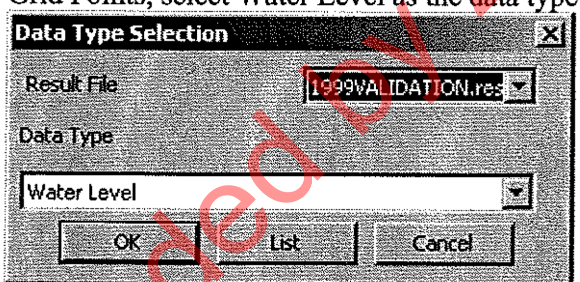
1. Launch MikeView and select File|Open.
2. Change the "Files of Type" field to MIKE11 DFS – files (*.res11)
3. Select the appropriate results file. Many results files were created for the Lower Fraser River model project; these are summarised in Table 8.
4. The full data set or a shorter time period can be loaded using the Data Load Selection window. Both water level and discharge should be loaded. Once the appropriate period has been selected, click OK. Keep in mind that the first 6 hours or so of a run will likely show instabilities. If you are looking at an additional output file (*ADD.res11) load whatever data types are relevant to your analysis.



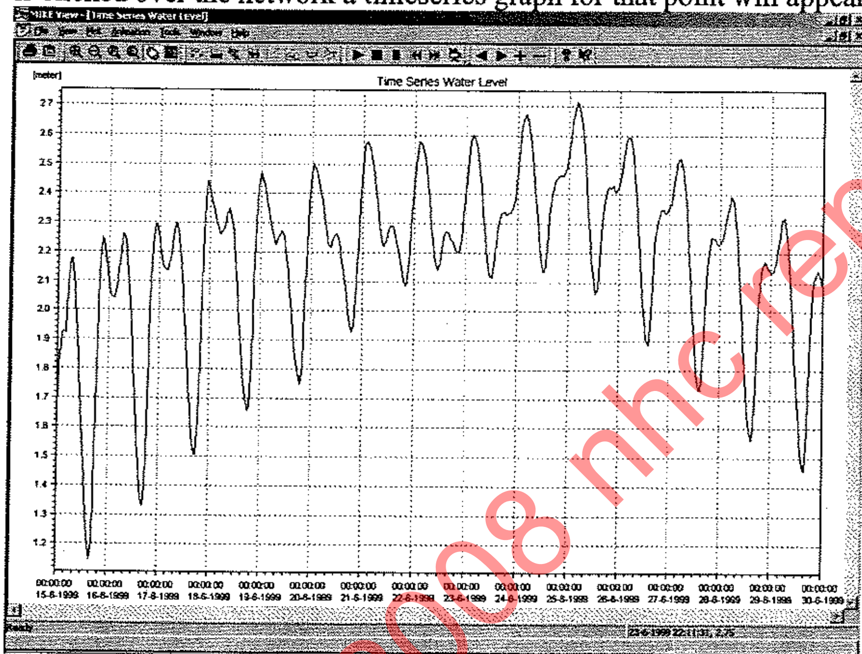
5. A new window showing the system network will open.



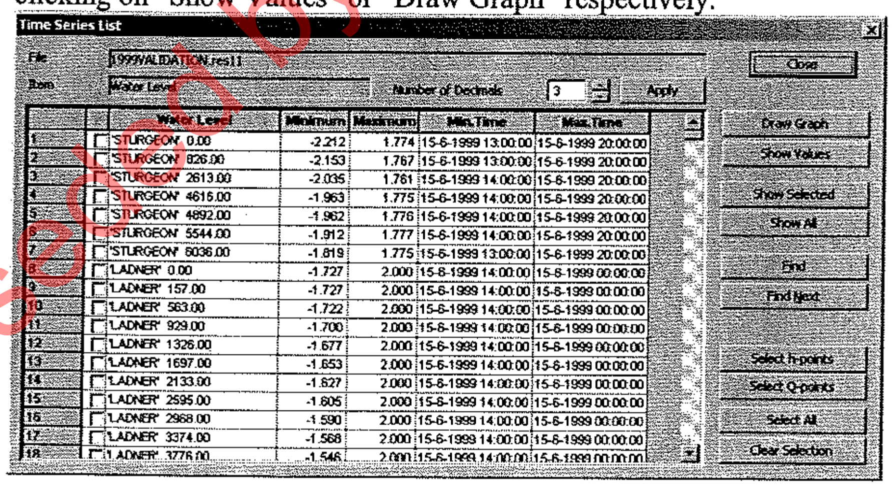
6. To look at water levels at any particular point on the network select Plot| TS in Grid Points, select Water Level as the data type and click OK or LIST.



a. If OK is clicked the mouse icon will change to a graph. When the mouse is clicked over the network a timeseries graph for that point will appear.



b. If LIST is clicked a table of cross-sections will appear. Individual cross-sections can then be selecting using the tick-boxes. Results from these cross-sections can then be viewed either as a table or as a graph by clicking on "Show Values" or "Draw Graph" respectively.

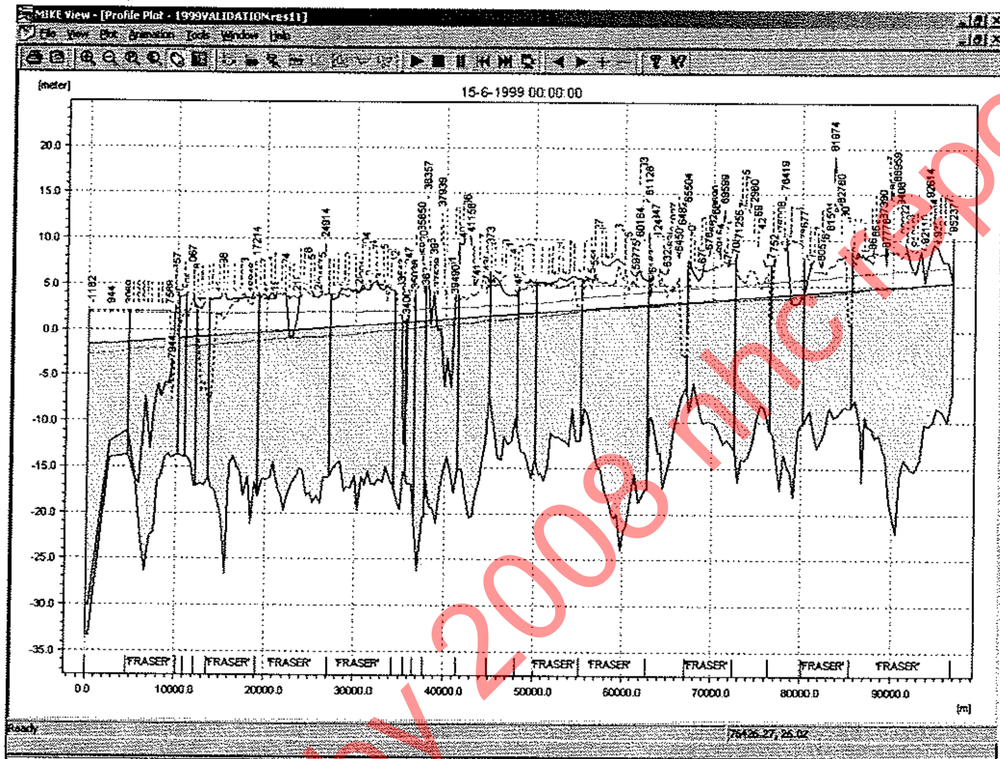


	Water Level	Minimum	Maximum	Min.Time	Max.Time
1	'STURGEON' 0.00	-2.212	1.774	15-6-1999 13:00:00	15-6-1999 20:00:00
2	'STURGEON' 826.00	-2.153	1.767	15-6-1999 13:00:00	15-6-1999 20:00:00
3	'STURGEON' 2613.00	-2.035	1.761	15-6-1999 14:00:00	15-6-1999 20:00:00
4	'STURGEON' 4616.00	-1.963	1.775	15-6-1999 14:00:00	15-6-1999 20:00:00
5	'STURGEON' 4892.00	-1.962	1.776	15-6-1999 14:00:00	15-6-1999 20:00:00
6	'STURGEON' 5544.00	-1.912	1.777	15-6-1999 14:00:00	15-6-1999 20:00:00
7	'STURGEON' 6036.00	-1.819	1.775	15-6-1999 13:00:00	15-6-1999 20:00:00
8	'LADNER' 0.00	-1.727	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
9	'LADNER' 157.00	-1.727	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
10	'LADNER' 563.00	-1.722	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
11	'LADNER' 929.00	-1.700	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
12	'LADNER' 1326.00	-1.677	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
13	'LADNER' 1697.00	-1.653	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
14	'LADNER' 2133.00	-1.627	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
15	'LADNER' 2595.00	-1.605	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
16	'LADNER' 2968.00	-1.590	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
17	'LADNER' 3374.00	-1.568	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00
18	'LADNER' 3776.00	-1.546	2.000	15-6-1999 14:00:00	15-6-1999 00:00:00

7. Alternatively a water profile can be viewed by going to Plot|Longitudinal Profile. The mouse icon will change to a graph. Individual reaches can be selected by clicking on top of them. A series of branches can be selected as long as they are connected. The branch selection can be finicky; it is often necessary to make several attempts. It is possible to undo the last selection by holding down the SHIFT button while clicking on the network. Once you have selected the appropriate branches hold down the CTRL button while clicking the left mouse

button. Then click YES when you are asked whether or not you would like to "Close profile selection and draw profile?"

8. Select water level or discharge depending on what you would like to view then click OK.



9. The time-series results can be viewed by clicking on the play button at the top of the screen. The maximum modelled water level is shown as a red-dashed line in the profile view.
10. Please refer to the MIKEView reference manual for further information on how to access data.