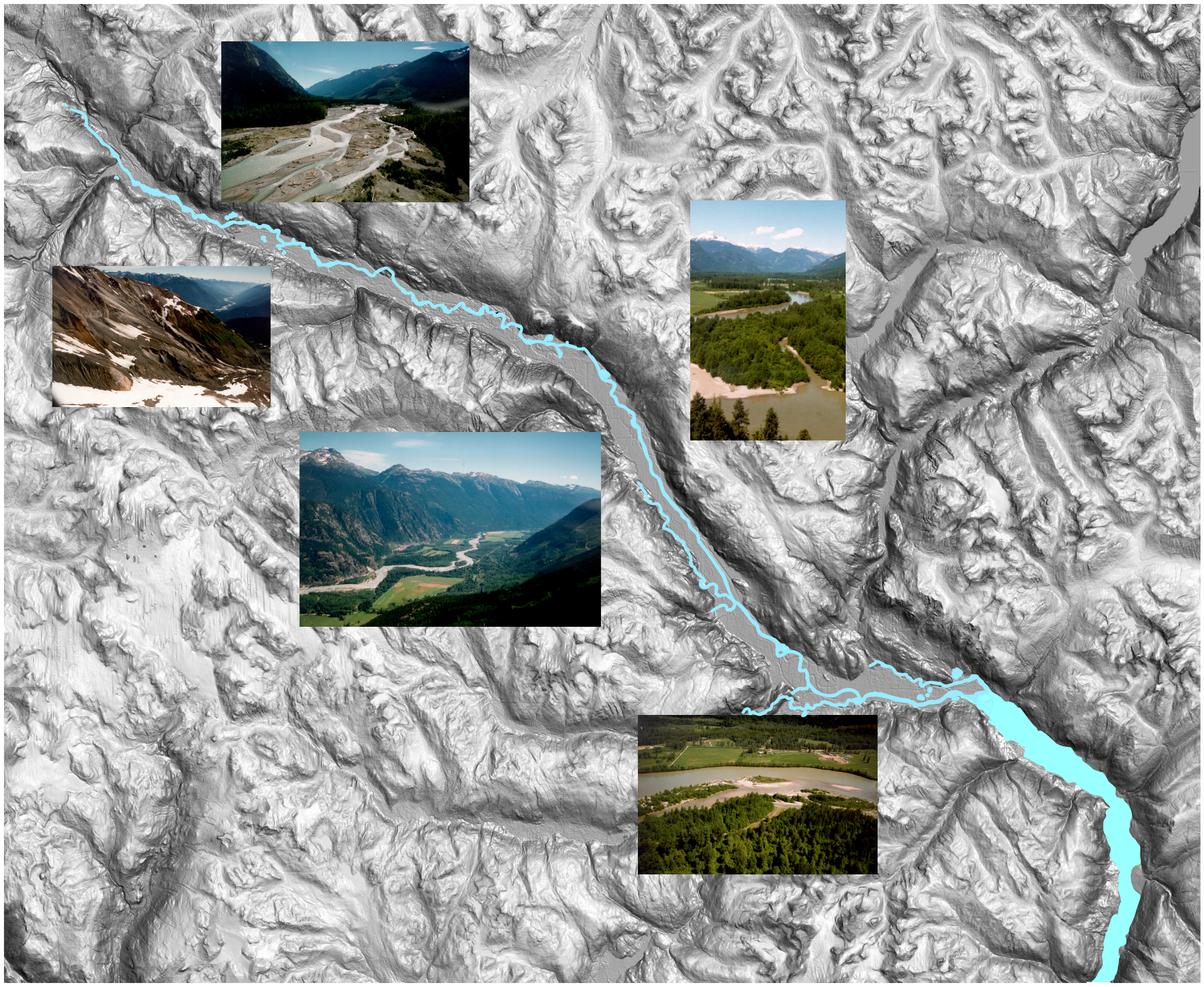

Engineering Study for

Lillooet River Corridor

Final Report
December 2002

Pemberton Valley Dyking District

Mount Currie Band

Canada

British Columbia

December 23, 2002

Mr. John Pattle, P.Eng.
B.C. Ministry of Water, Land and Air Protection
10470 - 152nd Street
Surrey, B.C.
V3R 0Y3

Dear Mr. Pattle:

RE: ENGINEERING STUDY FOR LILLOOET RIVER CORRIDOR
Submission of Final Report
Our File713.002

We are pleased to submit 3 copies of the *Engineering Study for Lillooet River Corridor Final Report*. This report presents current conditions and up-to-date hydraulic modelling results, with a backdrop of historical data and analysis of long-term geomorphological changes within the Pemberton Valley.

This report will assist the Steering Group, and communities at large, in understanding and documenting the problem areas. Further, this report will form the foundation of a flood mitigation and management plan for the Pemberton Valley.

We have very much enjoyed working on this project with you, and hope we can be of service to you again.

We trust this is satisfactory.

Yours truly,

KERR WOOD LEIDAL ASSOCIATES LTD.

Jonathon Ng, P.Eng., PMP
Project Manager

JN/am
Encl. (3)

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Engineering Study for

Lillooet River Corridor

Final Report
December 2002

KWL File No. 713.002

KERR WOOD LEIDAL
associates limited
CONSULTING ENGINEERS

STATEMENT OF LIMITATIONS

This document has been prepared by Kerr Wood Leidal Associates Ltd. (KWL) for the exclusive use and benefit of the Mount Currie Band, the Pemberton Valley Dyking District, B.C. Ministry of Water, Land and Air Protection, Public Works and Government Services Canada, and Indian and Northern Affairs Canada. No other party is entitled to rely on any of the conclusions, data, opinions, or any other information contained in this document.

This document represents KWL's best professional judgement based on the information available at the time of its completion and as appropriate for the project scope of work. Services performed in developing the content of this document have been conducted in a manner consistent with that level and skill ordinarily exercised by members of the engineering profession currently practising under similar conditions. No warranty, expressed or implied, is made.

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Executive Summary

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EXECUTIVE SUMMARY

River engineering, dyking and drainage efforts in the Pemberton Valley have been ongoing since the 1940s. Despite these efforts, the Pemberton Valley has been subject to regular flooding from Lillooet River and other tributaries. Recent damaging floods occurred in 1984 and 1991.

Two significant populations inhabit the Pemberton Valley: the Mount Currie Band, with a population centred in the flood-prone confluence area of the Lillooet River and Birkenhead River; and the non-native population, which is centred in the Village of Pemberton. Most of the previous flood protection work has focussed on the off-reserve areas where the Pemberton Valley Dyking District is the local dyking authority. The Mount Currie Band has jurisdiction over flood protection works on its reserves.

This study was guided by a Steering Group comprising the Pemberton Valley Dyking District, Mount Currie Band, Indian and Northern Affairs Canada, Public Works and Government Services Canada, and the BC Ministry of Water, Land and Air Protection. Input was also obtained from other stakeholders, including local landowners, environmental organizations, provincial and federal agencies, and the Village of Pemberton.

This study builds on previous surveys, technical reports, and scientific studies to provide a comprehensive geomorphologic perspective on Lillooet River. This includes consideration of sediment input from the Mount Meager volcanic complex, sediment transport characteristics, the effects major river works that were undertaken in the late 1940s (river diversions, river straightening, and lowering Lillooet Lake), and the effects of more recent dyking and bank protection works.

The most significant results of this study are:

- a summary of engineering works to date, along with an assessment of the effects of these works;
- updated surveys for 103 river cross sections;
- revised 200-year return period design flood levels for Lillooet River and tributaries;
- a gravel management plan for Lillooet River;
- an implementation plan for further flood protection improvements;
- recommendations for the Pemberton Valley Dyking District and the Mount Currie Band to proceed with flood protection improvements individually; and
- joint recommendations where it is necessary for the Pemberton Valley Dyking District and Mount Currie Band to work together in implementing flood protection improvements.

This study provides a strong basis for reducing flood risks in the Pemberton Valley.

Section 1

Introduction

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1. INTRODUCTION

1.1 BACKGROUND

STAKEHOLDERS

Two significant populations inhabit the Pemberton Valley: the Mount Currie Band, with a population centred in the flood-prone confluence area of the Lillooet River and Birkenhead River, and the non-native population, which is centred in the Village of Pemberton.

With respect to river management activities, the most significant stakeholders are the Mount Currie Band and the Pemberton Valley Dyking District (PVDD), which represent the majority of the populations and land affected by the Lillooet River and tributaries in this region. Active government stakeholders include the B.C. Ministry of Water Land and Air Protection (MWLAP), Indian and Northern Affairs Canada (INAC), and Public Works and Government Services Canada (PWGSC). Other government agencies with specific interests in management of the Lillooet River include Fisheries and Oceans Canada and the B.C. Ministry of Forests.

The PVDD is responsible for maintaining dykes and other flood protection works in accordance with provincial MWLAP guidelines. The PVDD's jurisdiction extends from the head of Lillooet Lake northwest to above Pemberton Meadows, but excludes the Mount Currie Band reserve lands.

Similarly, the Mount Currie Band is responsible for dykes and other flood protection works on their reserve lands. Funding for construction and maintenance of the majority of flood mitigation works on reserve lands is typically provided by INAC. Construction activities are generally undertaken by the Band after completion of feasibility studies and design, in accordance with the Band's Physical Development Plan (PDP). Projects on reserve lands are subject to federal guidelines, and review by PWGSC. Several Mount Currie reserves are located adjacent to lands under the jurisdiction of PVDD, necessitating a co-ordinated approach to river management and flood protection.

NEED FOR STUDY

In the past century, the Pemberton Valley has been subject to regular flooding despite ongoing engineering efforts. Part of the problem lies in the complex hydrology of the region with a number of large tributaries discharging into Lillooet River in the vicinity of the town of Pemberton (Figure 1-1). The highly dynamic geomorphological processes that are characteristic of Lillooet River and its tributaries have resulted in a constant struggle to wrest stable, arable land from the low-lying and often swampy floodplain.

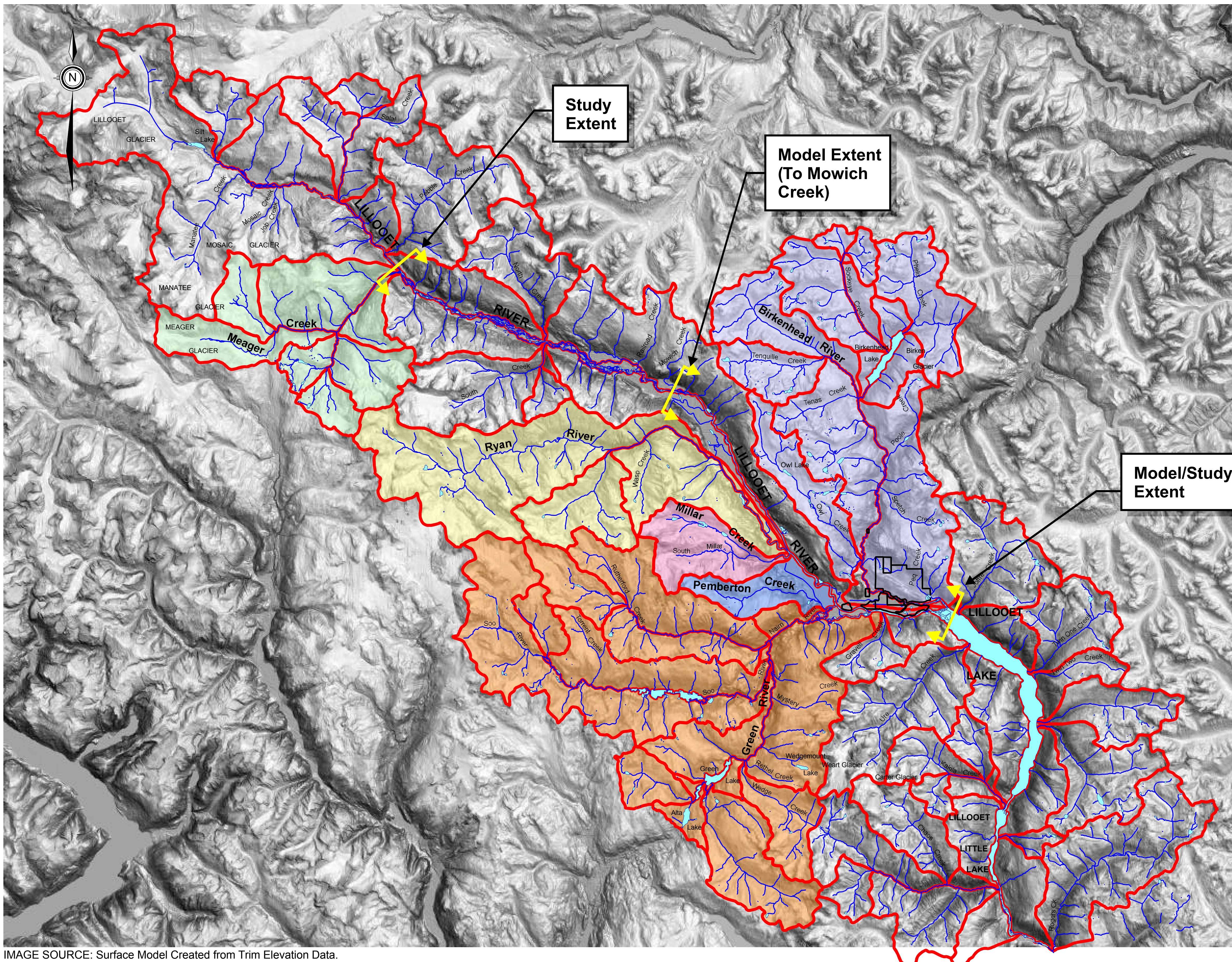

IMAGE SOURCE: Surface Model Created from Trim Elevation Data.
BOUNDARY SOURCE: B.C. Watershed Atlas: Lillooet Watershed Group - Captured from 1:50,000 NTS Mapsheets.

713-002 \ Report \ Drawings \ 713002SFig1-1.CDR

Engineering Study for Lillooet River Corridor

Legend

- Birkenhead River Watershed ($666\ km^2$)
- Green River Watershed ($869\ km^2$)
- Millar Creek Watershed ($74\ km^2$)
- Pemberton Creek Watershed ($51\ km^2$)
- Ryan River Watershed ($411\ km^2$)
- Meager Creek Watershed ($279\ km^2$)

Location Map

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10000 0 10000
Scale in Metres

Project No.	Date
713-002	December 2002

Map of Study Area

Figure 1-1

Since approximately 1946, flood control and drainage efforts in Pemberton Valley have resulted in varying degrees of flood protection for off-reserve lands and populations (detailed in Chapter 3). Only relatively minor flood and erosion protection projects have been implemented on Mount Currie Band lands.

The Mount Currie Band has ten reserves situated in the Pemberton Valley. The majority of the reserves are situated near the head of Lillooet Lake, several are bisected by the Birkenhead River, and most are located on floodplain near the confluence of the Birkenhead River (to the north) and the Lillooet River (to the south). The six reserves of greatest concern to the Band represent an on-reserve population totalling approximately 1500 people (2000 estimate), and include Mount Currie Nos. 1, 2, 6, 8, and 10, and Nesuch No. 3.

Because there are relatively few (and minor) flood or erosion mitigation works constructed on Mount Currie lands, and because some reserves are situated in the most flood-prone areas of the valley, the relatively unprotected reserve lands are subject to significant flooding and erosion damage.

The stakeholders all understand that flood mitigation works and management measures do not function in isolation, but rather produce long-term effects that propagate both upstream and downstream. All also realise the benefits of a co-ordinated analysis and flood hazard mitigation strategy that considers trans-jurisdictional effects.

This study seeks to develop a greater understanding of the extremely complex processes that shape the Pemberton Valley, and to also recommend practical, effective mechanisms for reducing flood risk for all who live and work in the study area.

1.2 STUDY OBJECTIVES

The primary objectives of this study are to provide:

1. an updated ‘snapshot’ of existing geomorphologic, hydraulic and hydrologic conditions;
2. a summarised history of all complete works and reports to date;
3. revised cross section survey data and comparative plots;
4. updated 200-year return period flood (the ‘design flood’) profiles;
5. a long-term river management plan, including floodplain management recommendations for the Pemberton Valley; and
6. summary and recommendations.

1.3 ENGINEERING WORK PROGRAM AND PROJECT DELIVERABLES

The work program consists of ten primary tasks. The task descriptions and associated major deliverables are summarized in the following table:

Task	Description	Major Deliverables
1	Project Initiation	▪ Background information ▪ Table of previous works
2	Base Maps and Field Work Prep	▪ Watershed map ▪ Study area maps
3	Initial Field Visit	▪ Field maps: photo/video locations, problem areas ▪ Reconnaissance photos
4	Survey Program	▪ Survey control network ▪ Cross section data and plots for 103 surveyed cross sections
5	Hydrology	▪ Report chapter ▪ Hydraulic model boundary conditions
6	Sediment Process Analysis	▪ Report chapter ▪ Maps of channel changes over time ▪ Delta advance map
7	River Engineering/ Geomorphology	▪ Report chapter ▪ Comparative section plots, river bed profiles
8	Backwater Analysis	▪ Report chapter ▪ Q_{200} and Q_{50} flood profile and associated design flood levels
9	Assessment of Management Options	▪ Report chapters ▪ Flood mitigation options and gravel management plan
10	Report	▪ Project report

Funding for this project was provided equally by PVDD and the Mount Currie Band, through the financial support of MWLAP and INAC respectively.

1.4 ENVIRONMENTAL ISSUES

The most significant environmental issues surround impacts on habitat of floodplain management measures and further urban development. There are limited regions within Pemberton Valley that remain undisturbed, and these regions are critically important to resident and migratory species.

A report identifying important habitat locations and dependent species is included as Appendix A.

1.5 PROJECT TEAM

The KWL project team includes:

- Mike Currie, M.Eng., P.Eng., Senior Water Resources Engineer;
- Jonathon Ng, P.Eng., Project Manager;
- Jeff Friesen, P.Eng., Project Engineer;
- Matthias Jakob, Ph.D., P.Geo., Senior Geoscientist; and
- Hamish Weatherly, M.Sc., P.Geo., Fluvial Geomorphologist.

The KWL project team was assisted by Mr. Donald Reksten, P.Eng., who was responsible for estimating peak flows, reviewing previous studies and data, and contributing a significant portion of Section 6. Cascade Environmental Resource Group addressed environmental issues, including preparation of Appendix A.

The project was generally directed by a Steering Group with the following representation:

Mount Currie Band:	Mr. Leonard Andrew, Capital Projects Manager
PVDD:	Ms Kathie Bergen, Administrator
	Ms Pia Fotsch, Administrator
	Mr. Sandy McCormack, Foreman
MWLAP:	Mr. John Pattle, P.Eng., Flood Hazard Specialist
PWGSC:	Mr. Don Burns, P.Eng., District Engineer
INAC:	Ms Mary Lui, Capital Program Specialist
	Mr. Brian Shantz, Capital Program Specialist

Input to the study was also obtained from a wide range of stakeholders, including other provincial and federal agencies, the Village of Pemberton, local business interests and area residents.

1.6 DEFINITIONS

The following definitions are used in this report, and are based on information from MWLAP (personal communication: Mr. John Pattle, September 12, 2002).

DESIGN FLOOD

A flood with a return period of 200 years (either the instantaneous flood or the maximum daily flood), based on a frequency analysis of unregulated historic flood records or by regional analysis where there is inadequate hydrometric data.

DESIGN FLOOD ELEVATION

The observed or calculated elevation of the design flood.

FREEBOARD

A vertical distance added to the actual calculated design flood elevation to allow for some of the following uncertainties:

- hydraulic and hydrologic variables, i.e. potential for waves, surges, etc.;
- other natural phenomena (sediment, ice & debris blockages, etc); and
- limited error in the design calculations.

The minimum freeboard is 0.3 m for the instantaneous design flood level or 0.6 m for the maximum daily design flood level – whichever is the greater.

DYKE DESIGN ELEVATION

The minimum crest elevation for a Standard Dyke, based on the design flood elevation plus freeboard. In the case of a Non Standard Dyke, such as an agricultural dyke, the minimum crest elevation may be based on a reduced flood event, such as a design flood with a return period of 50 years, plus freeboard.

Section 2

Study Area

2. STUDY AREA

This section provides an overview of the study area, starting with a general watershed description and settlement history. This is followed by discussion of bedrock and surficial geology, and watershed geomorphology. Climate and hydrology are discussed later in Section 6. These latter variables all have a major influence on the morphology, sedimentology, and discharge of Lillooet River.

2.1 LILLOOET RIVER WATERSHED

The study area encompasses the Lillooet watershed upstream of Lillooet Lake (Figure 1-1). Lillooet River drains an area of approximately 3,150 km^2 upstream of Lillooet Lake. Approximately 500 km^2 or 16% of the basin is glacierized. The mean elevation of the basin is 1,580 m, and about half of the basin is above the timberline. The maximum local relief (mountain top to adjacent valley) is 2,500 m, but average local relief is typically 1,500 to 2,000 m.

Lillooet River flows in an alluvial channel progressing from braided and cobble-gravel bedded channel in its upper reaches to a single-thread, sand-bedded channel for the lower 8 km. Except for the extreme upstream end, there are no terraces along the valley, suggesting that Lillooet River is aggrading throughout the valley and has done so since deglaciation of the valley approximately 10,000 years ago. Pleistocene glacial valley fill (more than 10,000 years old) are absent from most of the main valley. Commonly steep bedrock slopes covered by a thin colluvial or morainal veneer meet the floodplain abruptly. Alluvial fans are interfingered with the floodplain deposits, again suggesting that mass movements from adjacent hillsides occurred in an aggradational environment. In the alpine areas of the valley, neoglacial deposits are a common occurrence.

Approximately 65% of the discharge to Lillooet Lake is from the Upper Lillooet River, with the remaining 35% from Green River and Birkenhead River. Runoff from glaciers provides a high and continuous supply of fine and coarse sediment to Lillooet River and its tributaries.

2.2 SETTLEMENT HISTORY OF THE PEMBERTON VALLEY

Pemberton Valley was first inhabited by the Lillooet Tribe, living in “Slalok”, now the town of Mount Currie. The Lillooet Tribe belongs to a branch of the Interior Salish, who live between Fraser River and the Coast. The name Lillooet is believed to come from “lil’uet” meaning “Onion People”, which refers to the abundance of wild onion in the area. Another interpretation is a chief by the name of “A-ihl-ooet” who lived in the area. Wherever he camped, the site became known as: “A-ihl-ooet’s”. The first mention of non-natives in the area appears in a Hudson’s Bay Company census dated 1839, and both

Lillooet Lake and Lillooet River appear on a map prepared by the land surveyor Alexander Caulfield Anderson between 1832 and 1851.

The Mount Currie Band was the most westerly of the Interior Salish inhabiting an area of approximately one hundred square miles. Their main settlements were on the lower Lillooet River, on Lillooet Lake at Port Douglas and upper Harrison Lake. A small settlement still exists today at Skookumchuck. It appears that the area was never very densely populated and there is little archaeological evidence of people. The Lillooet people had to periodically defend themselves from intrusions of other tribes onto their territory. The Shuswap even dammed Lillooet River some 13 km upstream of Lillooet Lake to deprive the Lillooet people of their primary food source: salmon.

The first non-natives to come through the Lillooet River valley were employees of the Hudson's Bay company who were searching out a new fur trading route to Fort Langley. Francis Ermatinger of the Hudson's Bay Company was probably the first non-native who arrived via the Seton and Anderson Lake route.

In the 1850's, British Columbia Governor James Douglas ordered that a trail be built through the Squamish - Pemberton corridor to avoid the treacherous Fraser Canyon route. This trail led from Fraser River to Harrison Lake, up Lillooet River to Lillooet Lake, further along Birkenhead River and across Anderson and Seton Lakes to Lillooet. Five hundred miners eager to reach the gold-bearing Fraser River bars volunteered to construct the trail. For four years in the 1850s this route served as the main supply corridor to the gold fields in the Cariboo Mountains.

In 1881, approximately 5,000 acres of land were reserved for the native community from the area of Mount Currie to Lillooet Lake. Commissioner Peter O'Reilly of the Department of Indian Affairs had originally proposed that the entire valley be designated as a reserve. However, some land had been developed by the early 1890s and the proposal was dropped from consideration. When O'Reilly visited the village he was told that the 160 acres on which the village of Mount Currie is located had been given to the band by Governor Douglas twenty years earlier. While this could not be confirmed in the land office, another 1,200 acres were added to the reserve over the next 30 years.

In the early 1900s, the possibility of a railway connection with Squamish sparked interest in the agricultural potential of the valley. The richness of the soil and the uncontrolled meandering of the Lillooet River were well known, and in 1912, the many residents were petitioning the Federal Government for help in river control.

By 1914, the first train connected Lillooet with Squamish and a permanent non-native community was established in Pemberton. For many years the erratic course changes of Lillooet River allowed only a small percentage of the land to be used for agriculture. This situation did not change until 1946 when river works began under the auspices of the Prairie Farm Rehabilitation Administration (PFRA). The first complete survey of Lillooet River below at Tenas Narrows as well as upstream of the lake delta to Pemberton

Meadows began in 1944, and was completed in April 1945 by the then Pemberton Drainage District.

2.3 BEDROCK GEOLOGY

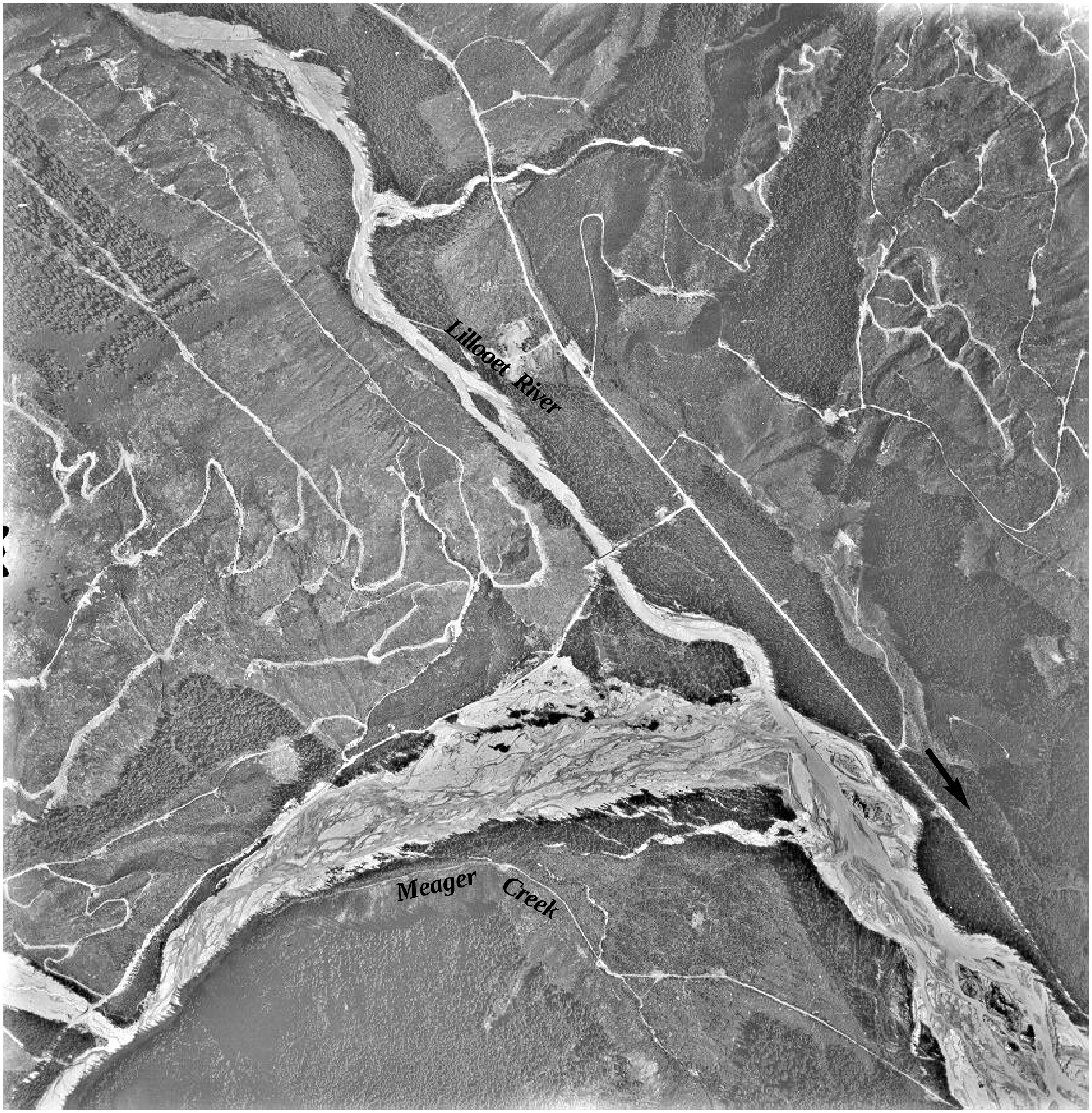
The geology of the Lillooet River watershed is as diverse as its geomorphic history. The major geologic structures are mid Cretaceous to early Tertiary in age. An important geological feature exists north of Meager Creek, the largest tributary to Lillooet River upstream of Ryan River: the Meager Creek volcanic complex. This area consists of early Pleistocene dacite and rhyolitic lava, and pyroclastic deposits, which have undergone hydrothermal alteration (Read, 1978).

The northern parts of the complex are underlain by mid Pleistocene dacitic and andesitic rocks. This area determines the morphology and sedimentology of Meager Creek because of frequent mass movements between 10^4 to 10^6 m^3 in volume. Meager Creek itself influences the sedimentology of Lillooet River, which is clearly demonstrated in Figure 2-1. Below the Meager Creek confluence, the single-thread Lillooet River abruptly transforms to a multi-channel, braided stream. As detailed further in Section 5.3, the high frequency and magnitude of mass movements in the Meager Creek watershed have caused a dramatic steepening of the channel gradient over the last 10 km of its course.

The Lillooet River on the north side of the Meager Creek volcanic complex is filled with tephra, pyroclastic flows and debris avalanche and flow deposits resulting from the most recent eruptive period. Jordan (1994) estimated their total volume at 10^9 m^3. The Meager Creek valley is remote from the site of Holocene eruptions, but is also deeply filled with debris avalanche and debris flow deposits ranging in age from 4,100 years before present to the present (Jordan and Slaymaker, 1991).

The Lillooet River valley is flanked by the Coast Crystalline complex to the south and Mesozoic metamorphic rocks with minor Tertiary volcanic rocks to the north. The Coast Crystalline complex or Coast Range Batholith reflects the dominance of plutonic rocks within the orogen (mountain-building period). In the Lillooet River watershed it consists mainly of upper Jurassic to lower Tertiary granodiorite and quartz diorite. Sections of fault-bound stratified rocks and various metamorphic rocks can also be found on the north side of the valley.

Both the plutonic and metamorphic rocks produce enough material to feed debris flow channels that transport sediment to the Lillooet River floodplain where they have formed alluvial fans over the past 10,000 years since deglaciation. Since the floodplain of Lillooet River is aggrading, these alluvial deposits interfinger with the floodplain deposits. Particularly in the braiding reaches of Lillooet River, alluvial fans are regularly truncated by fluvial erosion and therefore become part of the river bedload.



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Engineering Study for
Lillooet River Corridor

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Not to Scale

1994 Air Photo of Meager Creek Confluence with Lillooet River (30BCC 94012 No. 80)

Figure 2-1

Apart from rockfall, debris slides and debris flows, the majority of plutonic and metamorphic rocks in the Lillooet River watershed are not prone to the high frequency - high magnitude landslides commonly observed in the Meager Creek watershed. Nevertheless there have been at least four large rock avalanches: the Nairn Falls rock avalanche in Cretaceous diorite, the Birkenhead rock avalanche in Cretaceous granodiorite, the Hotsprings Creek rock avalanche in Jurassic quartz diorite, and an unnamed rock avalanche in the Cretaceous Gambier Group. Presuming that these rock avalanches occurred in the last 10,000 years, an average return interval of 2,500 years for $>10^7$ m^3 rock avalanches can be assumed. However, rock avalanches cannot be reliably predicted. A large rock avalanche in Lillooet River valley would at least change the river's course, and possibly dam it. However, due to their relative infrequency, rock avalanches are not specifically addressed in this report.

2.4 GEOMORPHOLOGY AND SURFICIAL GEOLOGY

Quaternary glaciation has strongly influenced geomorphic processes in the Coast Mountains of British Columbia. An examination of geology can therefore not be confined to bedrock characteristics only, but must include glacial effects.

The present valley alignment has developed from prolonged Cenozoic fluvial erosion exploiting zones of weakness created by earlier tectonic events. Re-elevation of the Canadian Cordillera occurred in the later Cenozoic time, leading to a renewed cycle of fluvial incision that further accentuated earlier erosional alignments. This was followed by multiple glaciations during the Pleistocene period, in which most valleys were widened and deepened. Deep U-shaped river valleys separate rounded ridges below 2,200 m, and sharp peaks at higher elevations. In the central part of the range, major river valleys such as the Squamish and Lillooet lie between sea level and about 500 m, with the elevations of adjacent peaks typically at 2,500 to 2,800 m. Present ice cover is extensive; large icefields occupy high elevation areas (above about 2,100 m), and feed valley glaciers that are the sources of most rivers.

Morainal materials accumulated in alpine areas during Quaternary time, with basal and ablation till blankets accumulating at lower elevations on most valley side slopes. Deglaciation resulted in the exposure of large amounts of this unconsolidated material, much of which was subsequently eroded by mass movement processes such as rockslides, rockfalls and debris flows, as well as continuous agents of denudations such as creep, slope wash, and solution transport. The rate of delivery of this material seems to have intensified immediately after deglaciation, causing a peak of sediment delivery to the fluvial system during the early Holocene time referred to as the paraglacial cycle (Church and Ryder, 1972). Neoglacial advances in the past 6,000 years have resulted in renewed morainal accumulations, many of which are highly unstable and are actively feeding mass movement processes today.

Section 3

Previous Work

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3. PREVIOUS WORK

3.1 ENGINEERING STUDIES AND WORKS

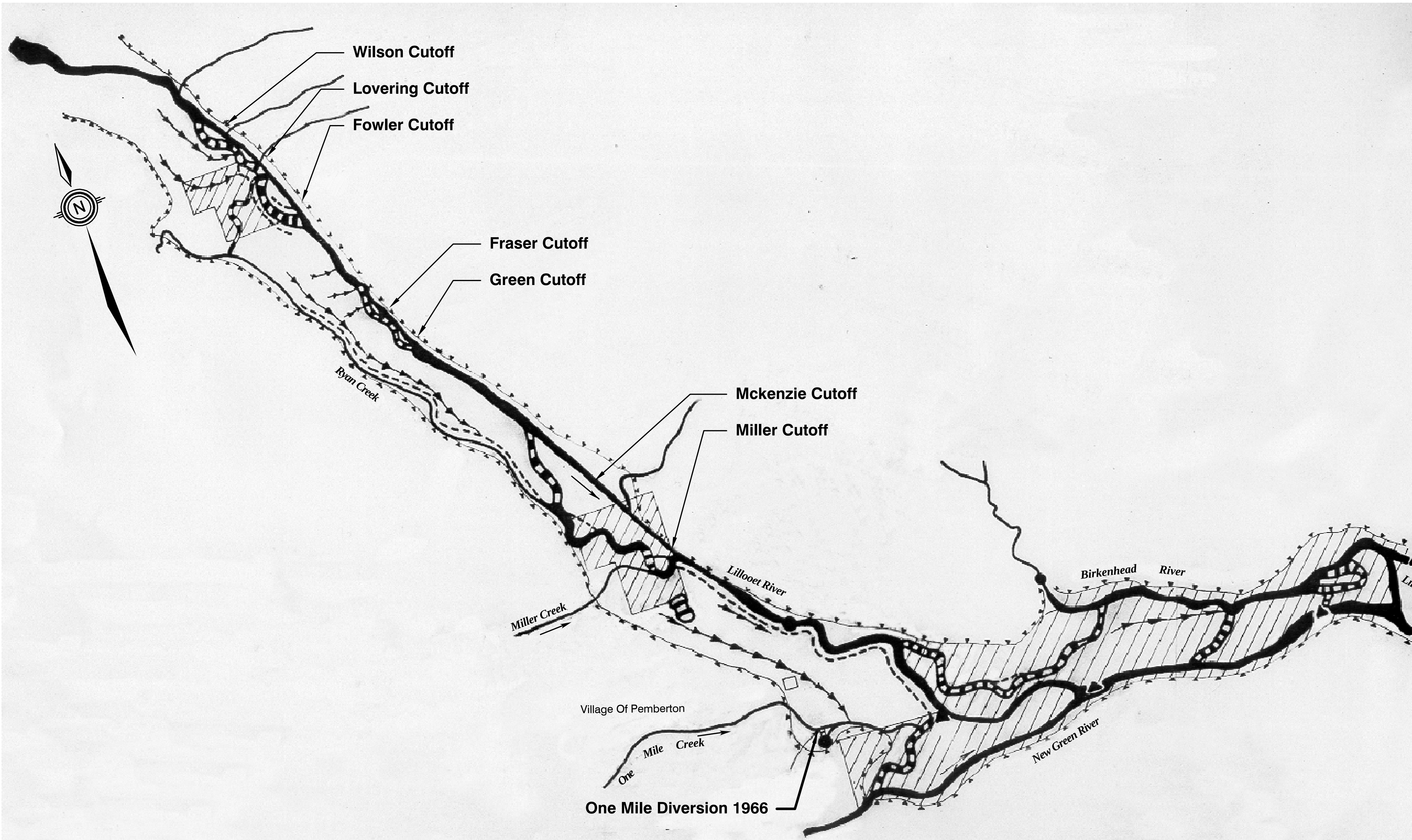
Lillooet River and its tributaries have undergone several periods of major engineering works. This section presents a summary of these works and a chronology of their implementation. Principal data sources were the floodplain maps of 1987 for works prior to this date and personal communications with Mr. Sandy McCormack of the Pemberton Valley Dyking District (PVDD) for works completed between 1987 and 2002. The chronology is not complete, but lists the most important works. More detailed descriptions are available at the offices of the PVDD where correspondence files on engineering works are archived.

Specific locations for most of the engineering works noted in this section are shown on Sheets 1 to 12 in Appendix B. Brief summaries of the completed works are also provided on these figures. The principal agencies and programs under which river engineering and floodproofing measures were carried out are the Prairie Farm Rehabilitation Administration (PFRA), the Agriculture Resource Development Subsidiary Agreement (ARDSA), the Provincial Emergency Program (PEP), the PVDD, the provincial River Protection Assistance Program (RPAP) and the Ministry of Transportation and Highways (MoTH).

1946 TO 1952

Due to recurring property damage caused by flooding, extensive engineering works were implemented after World War II to reclaim agricultural land and prevent future floods. Between 1946 and 1952, 14 km of river meanders were cut off and 38 km of dykes were constructed upstream of Lillooet Lake under the auspices of the PFRA. These works shortened the mainstem channel of Lillooet River by 5.5 km. Significant cutoffs are shown on Figure 3-1, including the 4.3 km long MacKenzie Cut (Figure 3-2) which drastically altered the confluence of Lillooet River and Ryan River. Flooding of land next to the MacKenzie Cut was encouraged for a number of years so that overbank silt deposition would raise the land elevation. In addition to the cutoffs, the water level in Lillooet Lake was lowered by 2.5 m in 1946. This occurred through dredging of the channel at Tenas Narrows and Lillooet Narrows. Volumes removed from these reaches were 3,000 m^3 and 470,000 m^3 respectively.

Green River, which used to enter Lillooet River at a right angle (where Pemberton Creek, formerly called One Mile Creek, now enters Lillooet River), was diverted along the foot of the mountain to join the mainstem several kilometres below the old confluence (Figure 3-1). This channel was initiated by excavating a pilot channel approximately 6 m wide and 2 m deep. This remedied extreme backwater effects that occurred at the confluence of Green River and Lillooet River during floods.



Engineering Study
for
Lillooet River Corridor

Legend

Present Water Course
Former Water Course
Dyke
Internal Drainage System
Limit of the Valley Floor
Areas Flooded 1968

Note

The drawing was scanned from Dwg. No. 0242512-27 prepared by B.C. Water Resources Service.

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Not to Scale

Project No.	Date
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Meander Cutoffs of Lillooet River Constructed in Late 1940s

Figure 3-1

713-002\Report\Drawings\713002StFig3-1.dwg

Dec.11/02

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Engineering Study for
Lillooet River Corridor

Project No.	Date
713-002	December 2002

Not to Scale

1947 Air Photo of MacKenzie Cut (BC 409 No. 19)

Figure 3-2

713-002\Report\Drawings\713002StFig3-2.dwg Dec.11/02

Throughout the valley, drainage canals were constructed so that land owners could drain their land with lateral ditches. A significant portion of the land base could not be used prior to the engineering works due to swampy conditions and in some cases, the drained land was situated several feet below the river banks.

Other significant changes to Lillooet River included simplification of the channel in the vicinity of Pemberton and Mount Currie. As shown on Figure 3-1, flow was divided in two channels upstream of the confluence with Pemberton Creek, with the northern channel flowing into Birkenhead River. Following the reclamation project, the north channel was cut off and abandoned.

Starting in 1951, the left bank of Pemberton Creek was dyked by the PFRA and extended upstream over the following years. At Birkenhead River, a riprapped dyke was constructed under the PFRA in 1950.

1953 TO 1978

Table 3-1 summarizes engineering works completed on Lillooet River and its tributaries between 1953 and 1978. Cross section locations noted in the table can be cross-referenced with the figures in Appendix B.

Table 3-1
Pemberton Valley Engineering Works, 1953 to 1978

Location	Date	Description
Ryan River		Approximately 1,900 m of riprap were placed by the PVDD and one owner along the north side of Ryan River.
Miller Creek		1,670 m of dyke constructed by local land owner, M. Miller.
All sections below refer to Lillooet River		
XS 56 – XS 53	<1970	600 m of riprap was placed sometime before 1970 by an owner on the north side of the river cutting off an oxbow, creating a slough 3 km upstream of the present Outdoor School. A total of 360 m of riprap was placed along the south bank of the river upstream of XS 56 under the RPAP.
XS 31.1 - XS 39	1975	Riprap was upgraded by the RPAP along the south side of the river.
XS 26	< 1970	Prior to 1970, 200 m of riprap were placed on the south side of the river between the confluences with Ryan and Miller Creeks.
XS 17 – XS 18	1977	700 m of riprap upgraded between XS 17 and XS 18.
XS 16	< 1970	Prior to 1970, 700 m of riprap were placed upstream of XS 16.
XS 9.2 – XS 0.1	< 1980	Before 1980, 380 m of riprap were placed by MacGillis & Gibbs Ltd. between the Green River confluence and Lillooet Lake.

1979 TO 1985

The 1946 to 1953 works were followed by rehabilitation and further improvement works (mostly bank protection) with ARDSA funding over a five-year period beginning in 1979. These works included bank protection and some dyking works on the Mount Currie Band lands. The purpose of these measures was to protect against the 1:50 year instantaneous flood level.

During this period, the Lillooet River dykes were raised to a crest elevation of 0.6 m above the observed December 27, 1980 peak flood profile. This resulted in raising of the dykes by about 0.15 m to a gauge reading of 6.25 m. Subsequently, the Water Survey of Canada determined that had the 1984 flood not overtopped upstream dykes, the flood would have reached a stage of 6.48 m.

A summary of engineering works in the Pemberton Valley for the period 1979 to 1984 is provided in Table 3-2.

Table 3-2
Pemberton Valley Engineering Works, 1979 to 1984

Location	Date	Description
Ryan River	1981 - 1985	Riprap and dyke sections were repaired under the Provincial Emergency Program and another 250 m of dyking was added just upstream from the Lillooet River confluence. Additional riprap was placed by MoTH, RPAP and PVDD.
Miller Creek	1984	330 m of dykes were replaced under PEP.
	1980 - 1987	A total of 91,000 m^3 of gravel bedload was removed between Talbot's Bridge and approximately 100 m downstream of the Highway Bridge.
Birkenhead River	1979 - 1983	Existing bank protection was improved and additional bank protection as well as a short dyke along Grandmother Slough were constructed under the ARDSA program.
Pemberton Creek	1980	Riprap replaced along a 130 m reach and rock was placed across the stream to prevent upstream erosion.
	1981	510 m of dyke were constructed and 880 m^3 of riprap was replaced along a 120 m reach. A damaged culvert through the existing dyke was also replaced.
	1980 - 1986	17,000 m^3 of gravel bedload was removed from a reach of Pemberton Creek bounded by the B.C. Rail bridge and the Underhill Bridge.
XS 56 – XS 42.1	1981 - 1984	Riprap was placed alg several sections on the north side and south side of Lillooet River by MoTH and two local land owners, Mr. N. Smith and Mr. J. Smuk. Riprap was placed where the road was being affected by bank erosion.

Location	Date	Description
	1981 - 1982	Riprap ranging in length between 80 m and 560 m was placed on the south side of Lillooet River up to XS 55. This work was carried out by the ARDSA project as well as PEP. Some of this riprap was subsequently extended by RPAP as well as PVDD.
XS 42.1 – XS 25	1981 - 1982	Sections of riprap were placed on the south side of Lillooet River under the ARDSA project.
	1984 - 1985	Riprap repairs on the north side of the river were funded by PEP and MoTH.
	1984 - 1986	In 1984 and 1986, 760 m^3 of riprap were replaced by PEP.
XS 25 – XS 9.2	1980	3,000 m of riprap were placed on the north side of the river under the ARDSA project.
	1979 - 1983	In 1979/80 and again in 1983 approximately 3,500 m of dyke were built on the north side of the river.
	1983	150 m of riprap placed by the Ministry of Lands, Parks and Housing.
	1980 - 1981	13,000 m^3 of gravel were replaced along dykes by the PEP in addition to culvert and flap gate replacement between XS 10 and XS 11.
	1980 – 1984	Sections of riprap were replaced or repaired under the PEP.
XS 18	1984	B.C. Rail placed 220 m of riprap at XS 18 to protect their tracks.
XS 17		Under the ARDSA program the old flood channel on the left bank of Lillooet River near the Pemberton airport was closed by a short dyke known as the North Arm Plug. This plug was later raised and extended downstream to the highway bridge.
XS 11	1980	700 m of ditching were carried out near XS 11 on the north side of the river. Some 2,800 m of dyking (used as public road) were constructed on the south side of the river.
XS 9.2 – XS 0.1	1979	Approximately 6 km of access roads constructed to reach Lillooet River from the highway and along the river for its length along Mount Currie Lands.
	1979 - 1983	Over 3 km of riprap placed along the north side of the river under ARDSA.
XS 6	1979	On the north side of Lillooet River, a 160 m dyke was constructed to prevent flooding of an old slough that lead to Birkenhead River across Mount Currie Land.
XS 1 – XS 2	1979	1,150 m of riprap placed between XS 1 and XS 2 on the north side of Lillooet River.

1985 TO 1990

In 1985, the Ministry of Environment, Water Management Branch, issued a flood study for the Pemberton Valley. The study was preceded by floods in December 1980, October 1981, and the October 8, 1984 flood, which is now the third highest on record, only exceeded by the 1940 and the August 1991 floods. Funding for these works was provided by the Provincial Emergency Program to repair damages to protective works and to return river channels blocked by sediment and organic debris to their pre-1984 conditions. Significant gravel was removed during this time, particularly in Ryan River. This gravel was used for dyking in 1985 when 600 m of dyke were constructed at the location where Ryan River meets the Lillooet River floodplain.

After the 1984 flood it was suggested that further lowering of Lillooet Lake could lower the flood risk in the lower Pemberton Valley. Calculations by Nesbitt-Porter (1985) demonstrated that the effects of a 3 m lake level lowering would be negligible upstream of the Green River confluence, and therefore have virtually no effects on flooding in the Pemberton Valley. It was also recognized that increasing flow velocities in Birkenhead and Lillooet River would cause accelerated bank erosion, bed scour and subsequent undercutting of the toe of existing riprap.

The 1985 report suggested new dyking from granular fill and on-site materials, replacement or relocation of suspect dykes, a dyke setback of at least 30 m to the top of the river bank and the use of public roadways for flood protection. Dykes were to be built with a minimum crest width of 4 m and 2:1 side slopes. Dykes would average 1.1 m in height for the 1:200 year flood. Dykes vulnerable to erosion were to be protected by riprap. In addition, floodproofing was suggested by constructing ring dykes, raising houses above the flood construction level (FCL) on gravel pads, and elevating buildings by adding concrete or masonry foundation walls. For the section between Miller Creek and Pemberton, it was proposed that the Miller Creek dykes be raised by an average of 2.1 m and the Lillooet River dykes be raised by an average of 0.5 m to protect against the 1:200 year flood, excluding allowances for gravel deposition.

Following the report, flood protection works were carried out at Miller Creek in 1986 and 1988. Under the RPAP, 4 km of dykes were constructed together with riprap placement and lock block installation. At Pemberton Creek, 3.6 km of dykes were constructed in 1987. In addition, three 1,200 mm corrugated metal pipe (CMP) culverts with flap gates were installed between the Lillooet River confluence and the town of Pemberton.

Along Lillooet River, RPAP provided funding for the construction of 3,450 m of dykes upstream of the forest road bridge on the south side, while approximately 8 km of dykes were constructed downstream of XS 25 in the mid to late 1980s.

1988 HEC2 MODELS

A series of HEC2 models were developed by the Provincial Ministry of Environment (now MWLAP) based on the 1985 cross section survey. Separate models of Lillooet

River, Birkenhead River, Green River, Ryan River, Miller Creek and Pemberton Creek were developed, with a total of 237 cross sections. With the exception of the Lillooet River model which used 91 cross sections, the tributary models extended significantly further upstream than the 2002 modelling exercise.

Cross sections were considered to be confined between dykes where standard dykes existed, and were extended across floodplain where dykes did not exist. Floodplain data for cross section extensions were extracted manually from 1 m contour mapping where required.

Peak flow estimates from the 1984 flood were used as input. The models were calibrated to the 1984 recorded HWMs, to within an average of 0.13 m departure from observed HWM elevations.

These modelling results were used to derive the current (1990) FCLs and floodplain limits for Pemberton Valley.

1990 TO PRESENT

In January 1995, the B.C. Ministry of Environment, Lands and Parks (MELP, now MWLAP) Water Management Branch completed a review of floodplain mapping for Lillooet River near Pemberton. The following conclusions were drawn from the study:

- the estimated 200-year return period peak instantaneous discharge at WSC gauge 08MG005 was increased from 1,170 m^3/s to 1,400 m^3/s, based on the August 1991 flood. This translates to a 20% increase;
- flood levels shown on the October 1990 floodplain mapping were greater than the flood levels observed during the August 1991 flood upstream of the Miller Creek confluence. In contrast, the area from Lillooet Lake to Miller Creek confluence showed flood levels equal to those observed in 1991;
- the 200-year return period Lillooet Lake level was increased to 200.3 m GSC; and
- a comparison of cross sections XS 6 to XS 16 for the years 1969, 1978, 1985 and 1993 indicated that the reach had stabilized with little change in channel capacity having taken place since 1978.

In response to these findings, several recommendations were made:

- the flood levels from the 1988 studies were to be retained since they proved to be adequate and accurate;
- a pledge was made to several agencies to seek co-operation with the Water Survey of Canada in re-establishing hydrometric station 08MG003 (Green River near Pemberton); and

- the Ministry should encourage local authorities to educate landowners with regard to the threat posed by floods.

In September 1998, the Ministry of Forests (MoF) proposed an upgrade of the bridge crossing for Tenas Narrows on Lillooet River due to structural problems and weight restrictions. Forest road requirements are for passage of the 100-year return period flood with 1 to 1.5 m of freeboard. Maintaining the existing span of 86 m was not considered a desirable option since the bridge constricted the natural channel during normal flood events.

To assist MoF in their planning, a preliminary hydraulic analysis of the crossing was completed by MELP. Lillooet Lake levels for the existing Lillooet Narrows configuration were found to be relatively insensitive to losses at the bridge crossing (i.e. net increase of 0.15 m to Lillooet Lake levels). However, Tenas Lake levels were very sensitive to the bridge configuration due to backwater effects. MELP concluded that assuming Little Lillooet Lake is not backing up into Tenas Narrows, losses for the 200-year flow (1,800 m^3/s) would be reduced by 1 m to 3 m by replacing the existing bridge with a 120 m span and excavating the infilled material. Because the analysis was based on 1970 soundings of the narrows, MELP recommended the hiring of a hydraulic consultant to assess backwater conditions based on updated soundings. We are aware of no further action on this at the time of writing.

In May 2000, flood construction levels (FCLs) in the village of Pemberton were adjusted. An updated flood frequency analysis conducted by MELP determined that the 200-year return period peak instantaneous flow estimates for Lillooet River should be increased by 25% over the flow estimates used to produce the 1990 floodplain maps (1,170 m^3/s to 1,470 m^3/s). As a condition of MELP consent for all proposed subdivisions in Pemberton, 0.3 m was added to the FCL determined from the floodplain maps. The adjustment factor applied from XS 21, near the WSC gauging station, to the confluence with Pemberton Creek.

It was further noted that lower reaches of Lillooet River had experienced relatively greater increases in the flood profile (as shown by 1991 flood high water marks) and hence required an adjustment greater than 0.3 m. Adjustments for those areas were to be determined on a site-specific basis pending subdivision referrals. The adjustment factor of 0.3 m was intended as an interim measure only, in lieu of updated floodplain mapping. To properly assess the updated flood frequency analysis, MELP recommended that a comprehensive review of floodplain mapping by hydrological mapping and new channel surveys be conducted – a recommendation being addressed partially by this study.

In addition to these studies, engineering works continued on Lillooet River and tributaries, as summarized in Table 3-3.

Table 3-3
Pemberton Valley Engineering Works, 1990 to Present

Location	Date	Description
Miller Creek	1991	In 1991, PEP funded riprap repairs and dyking improvements along the outer bend of Miller Creek between the road bridge and the Lillooet River confluence. One year later, more riprap was placed along the right channel bank of Miller Creek upstream of the road bridge, again funded by PEP.
Ryan River	1991 – present	PEP funded and repaired riprap on the right channel bend just upstream of the road bridge across Ryan River in 1991. A setback dyke and riprap repairs were carried out in 1991 and 1993 at XS 22. Further riprap repair was carried out in 1993 between XS 42 and XS 43. In 1994, the dyke on the north side of Ryan River between XS 36 and XS 38 was raised by 30 cm (PEP). Riprap was repaired by the PVDD on the left channel bank in 1997. In 1999, riprap was repaired at XS 38 by the PVDD.
Pemberton Creek	1991	PEP funded the construction of a dyke on the south side of Pemberton Creek between XS 2 and XS 4.
Lillooet River		
XS 18 – XS 20	2002	Dyke upgrade on right bank to new dyke design elevation; funded by FPAF.
XS 56	1990's	Throughout the 1990s, MoTH repaired riprap along their road upstream of XS 56, where the river was eroding the road embankment.
XS 53	1997 – 1998	A spur dyke was placed by the PVDD upstream of XS 53 on the south bank of the river.
XS 52.2	1999	PVDD carried out riprap repairs downstream of XS 52.2.
XS 52 – XS 51		PEP funded riprap repairs on the right (south) bank of the river.
XS 51 – XS 49		Riprap placed by private owner (Mr. Los) on the north bank.
XS 46 – XS 45	1993	PEP funded the repair of riprap upstream of XS 45 and toe repair on the right channel bank of Lillooet River at McLeod Creek ranch (downstream of XS 46).
	1996 – 1997	The schoolboard added riprap to existing riprap upstream of the Wolverine Creek confluence.
XS 28 – XS 31		A dyke with a 30 m setback was constructed along MacKenzie Cut approximately from XS 28 to XS 31.
XS 27	1991 - 1992	Riprap was repaired and the dyke raised upstream of Ryan River confluence up to XS 27.
XS 26 – XS 24	1999 - 2000	Riprap was repaired downstream of the Miller Creek confluence.

Location	Date	Description
XS 16.1	1991	PVDD funded and completed riprap repair downstream of XS 16.1
XS 14	1997 - 1998	In 1997, the MoTH placed new riprap along the southern bank of the river paralleling the dyke (which is used as the access road to Pemberton airport). In 1998, another short section of riprap was upgraded at the Pemberton Creek confluence.
XS 11 – XS 10	1995	To protect their helicopter site and camp, MoF placed riprap on the south side of Lillooet River.

3.2 SCIENTIFIC STUDIES

LILLOOET RIVER DELTA FORMATION

Lillooet Lake is 22.2 km long and covers an area of 20.5 km^2. Its mean water surface elevation is 196 m, but varies seasonally by as much as 4 m. Its maximum depth is 137 m. Dredging at the lake outlet in 1946 lowered the average water level by 2.5 m to decrease the effects of flooding of Lillooet River. The drainage area upstream of the lake outlet is 3,850 km^2. The mean elevation of the watershed is 1,580 m and about half the basin is above the timberline.

Studies by Gilbert (1973, 1975) and Desloges and Gilbert (1994) have shown that several centimetres of fine sediment (larger than sand sized) accumulate annually near the inflow point. The rate of accumulation is dependent on seasonal variations in runoff. The main conclusions of these works were:

- Light and dark couplets in the lake sediments represent summer and winter laminae respectively, with each couplet representing a varve (total yearly deposition). This finding demonstrates the distinct difference in summer and winter deposition.

- ^{137}Cs (cesium) concentrations in lake sediments showed a one year lag between atmospheric peak (rainfall, spring runoff) and sediment peak, which is typical for other glacial lakes. Three types of varves were identified: simple varves, multiple layered varves and anomalously thick varves. Simple varves constitute slow uniform deposition throughout the year. Multiple layered varves express frequent intraseasonal deposition, which can be traced back to individual runoff-generating processes. Anomalously thick varves occur infrequently and reflect rare flood events such as the October 1984 flood. Other thick laminae were deposited in 1906, 1932, 1940, 1957, 1958, 1967, 1980 and 1984.

- The eruption of Mount Meager 2,430 years before present some 50 km upstream of the lake left a distinct acoustic reflector surface in the lake sediments. A second reflector surface may be linked to higher sediment yields at the onset of the Little Ice Age some 400 years ago.

- About 83% of the total annual sediment yield (7.2 x 10^5 m^3) is contributed by headwater glaciers.

- In a 124-year sediment record cored from the lake, the period between 1965 and 1985 shows exceptionally high rates of sedimentation. Desloges and Gilbert associated the high rates to a higher frequency of extreme runoff-generating events during this interval.

Two models were proposed by Jordan and Slaymaker (1991) to explain the observed aggradation in Lillooet River valley (i.e. lack of terraces) and delta formation. Both models assume a constant discharge and constant sediment delivery over time. However, there is a high likelihood that both discharge and sediment input will increase over the next century.

A principal tenet of the models is that Lillooet River will attempt to preserve channel slope as the delta advances. This is due to the tendency of rivers to maintain equilibrium conditions and achieve channel stability. Channel slope can only be preserved if upstream reaches aggrade as the delta advances. In the first model, the aggradation in the delta reach is applied evenly along the length of the valley. This would result in an extension of the delta reach, while the other reaches do not change in length. However, as Jordan and Slaymaker (1991) point out, differential sorting and deposition of coarse bed material would cause an extension of the upper reaches as well. Their second model assumes the length of each reach remains constant, except for the uppermost reaches, and results in greater rates of aggradation for the upper reaches. The authors conclude that the most likely pattern of aggradation probably lies between the two extremes.

SEDIMENT BUDGET FROM DELTA DEPOSITS

A central problem in sediment budgeting is the role of storage of sediment between the point of its provenance and the mouth of the basin. The conventional model states that the greatest denudation occurs in the smallest headwater tributaries, and that denudation is gradually reduced with increased basin size and a decrease in precipitation intensity. Church and Slaymaker (1989) have proposed an alternative model that takes into account the reworking of paraglacial (post-glacial) sediment. This model suggests that sediment yields actually increase downstream.

Several studies conducted in California have used a sediment budget approach to investigate the response of a basin to, or its recovery from, disturbances resulting from extreme meteorological events (Kelsey, 1980, 1982; Madej, 1987). They found that most of the debris produced during an extreme rainstorm was stored as channel deposits along the upper channel and that the zone of aggradation migrated downstream of a time scale of decades. Similar findings were made by Roberts and Church (1986) in the Queen Charlotte Islands.

The modern accumulation record in Lillooet Lake allows inference to some of the sediment sources in the watershed. The total sediment yield was estimated by Desloges and Gilbert (1994) to approximately 16 x 10^5 m^3/year. The main sediment sources include landslides and debris flows in the upper basin as well as glacial runoff. Table 3-4 summarizes the main sediment sources and their estimated input in 10^5 m^3/year. The significant differences between estimated and expected sediment outputs are due to the difficulty and lack of precision in quantifying the sediment budget.

Table 3-4
Sediment Sources and Input Rates to Lillooet River

Sediment Source	Input (x 10^5 m^3/year)
Input	(all fractions)
Debris flows	1.4 - 4.3
Rock avalanches, debris slides, slumps	0.3 - 1.1
Glacial runoff	1.0 - 5.0
Total	***2.7-10.4***
Storage	
Floodplain	4.0 – 16.0
Outputs	
Estimated	6.3 - 13.3
Expected	15.5 – 16.5

If the regime of sediment delivery and transport prevails, the amount of sediment deposited on the floodplain must equal the amount removed and finally deposited in Lillooet Lake. Basic calculations have shown that the fine fraction (larger than 63µm) in the overbank floodplain sediments may vary between zero during low flow years and 1.9 x 10^5 m^3 if significant overbank flooding and vertical accretion occurs.

Not all sedimentation events are related to extreme flows (e.g. 1980, 1932), and not all extreme flows produce large sediment accumulations (e.g. 1968, 1946). It is interesting to note that three of the seven largest sediment yield events in a 124-year record occurred after 1979 and five of the largest seven occurred after 1965. It is estimated that there is less than 1% chance of this result occurring randomly. The post-1965 trend is coincident with a well-documented shift to wetter conditions in southwestern British Columbia.

MASS MOVEMENTS AND THEIR HYDROLOGIC LINK

Mass movements play an important role in the morphologic development of gravel bed rivers. Although over 80% of the total sediment load of sediments delivered to Lillooet

Lake are derived from glacially fed streams, coarse sediments are almost exclusively integrated from mass movements such as debris flows, debris avalanches, debris slides, rockfall, rockslides and rock avalanches. These processes either discharge their load directly into Lillooet River, or form fans impinging on the floodplain, which are eroded over longer time periods.

Unlike many other forested mountain slopes in coastal British Columbia, debris slides from open slopes are not a significant sediment source in the Lillooet River basin. This is likely due to the shallow soil cover and the absence of glacial till over large sections of the valley.

As stated earlier, mass movements in the Lillooet River valley are a common occurrence. The rate of mass movements or landslide activity increases sharply with the vicinity to the Mount Meager volcanic complex. Large landslides in this area occur at highly irregular intervals and sediment routing is therefore problematic. From a sedimentological point of view, however, only landslides during the past 100 years or so may be relevant for the formation of the Meager Creek floodplain and the planform of Lillooet River downstream. The most notable landslides that have occurred in this time period are the 1931 Devastation Creek debris flow (5×10^6 m^3), and the 1975 Devastation Creek debris avalanche (1×10^6 m^3), as well as the July 1998 debris flow and landslide dam at Capricorn Creek (1.2×10^6 m^3). These landslides occurred in response to recent glacial retreat and suggest that similar events will likely reoccur in the Devastation Creek and Capricorn Creek basins as well as other sites where steep colluvial and weak bedrock slopes have been glacially debuttressed.

The Mount Meager volcanic complex is a high-relief area comprising several deeply eroded, dormant-to-extinct stratovolcanoes of late Pliocene to Holocene age. The most recent eruption occurred 2,400 years ago at Plinth Peak, and deposited tephra over a wide region of south-central British Columbia. The volcanic rock assemblage is generally poorly consolidated, and steep slopes are prone to catastrophic collapse (Photo 1, Appendix C). The complex is drained by 15 sub-basins, which deliver debris flows and debris avalanches of variable sizes to Meager Creek. At least six events have caused temporary impoundments of Meager Creek in the past 60 years. The unusually high level of landslide activity is due to the existence of steep, high relief areas carved into relatively weak volcanic assemblages, to the humid climate, and to the very rapid recession of glaciers in the past 150 years. The high frequency of landslides within basins on the south side of Meager Creek volcanic complex causes a disproportionately high sediment load relative to its drainage area and discharge. It is therefore the dominant coarse-sediment source for Lillooet River. This is manifested morphologically by an abrupt change from a single thread, straight channel to a wide, multiple channel, braiding system at the Meager Creek confluence (Photo 2, Appendix C).

It is predicted that glaciers will further recede in coastal British Columbia. Accompanied with this retreat will be a regional increase in mass movement activity, which will be particularly pronounced in areas with steep unstable rocks that have to date, been buttressed by glacial ice. At Mount Meager, the rate of mass movement is expected to increase over the next several hundred years, should glacial retreat continue as expected. This development would mean that more and more sediment will be released into the Lillooet River system, which will result in local aggradation in the braided reach and potentially increase sediment transport rates to reaches below the forestry bridge.

Section 4

Work Program Implementation

KERR WOOD LEIDAL
associates limited
CONSULTING ENGINEERS

4. WORK PROGRAM IMPLEMENTATION

The work carried out for this study consisted of field work and analysis conducted at KWL's North Vancouver offices, and at the Department of Geography at the University of British Columbia. Field work consisted of a large survey component, gravel sampling on bars of Lillooet River, bank erosion investigations, as well as resident interviews. The analysis focused on river geomorphology and modelling. The overall study process involved various levels of community and stakeholder involvement, and was guided by a Steering Committee.

4.1 FIELD WORK

SURVEY

Cross section surveys were conducted on Lillooet River, Birkenhead River, Green River, Miller Creek, Pemberton Creek, Ryan River, and Lillooet Lake Narrows. The survey had the following objectives:

- to create a hydrodynamic model of the Lillooet River and main tributaries for the purposes of establishing updated flood levels and assessing mitigative alternatives;
- to assess river aggradation/degradation over time; and
- to provide accurate NAD83 UTM co-ordinates for surveyed cross sections to aid in future location.

A total of 103 cross sections were surveyed for this study, most in November 2000. The table below details the number of sections surveyed for each river and creek:

Location	Sections Surveyed
Lillooet River	56
Birkenhead River	18
Miller Creek	7
Ryan River	4
Green River	9
Lillooet Lake Narrows	3
Pemberton Creek	6
Total	**103**

The survey data obtained from this exercise were used for two purposes:

- as the basis for a Mike 11 hydrodynamic model of the Lillooet River and main tributaries; and

- as a basis for comparison with previous cross section surveys in the geomorphological analysis explained below.

Each channel cross section was surveyed, and later positioned both horizontally and vertically via GPS survey. This GPS control survey tied together the individual cross section surveys, and the existing benchmarks in the Valley.

The survey data and more detailed reference material is included in a companion binder to this report: *November 2000 Survey, Lillooet River and Tributaries*. A brief explanation of horizontal and vertical datums and accuracy is given below.

Horizontal and Vertical Datums

The horizontal datum is NAD 83. Co-ordinates are UTM projection (Zone 10). The vertical datum is Geological Survey of Canada CVD 28, referred to geodetic benchmarks 599J and 601J, (1975). Elevations correspond with the ‘old’ (unadjusted) levels, so that elevations may be compared without adjustment to prior survey results.

Survey Control – Horizontal

Horizontal control was established between November 6 and 11, 2000, by Real Time Kinematic (RTK) GPS. Initial static vectors were measured to establish UTM co-ordinates for two temporary key stations in Pemberton Valley, which were then used as base stations for RTK measurements to the remaining points. The static measurements were post-processed using data from three Provincial Active Control Service (ACS) receivers located in North Saanich, Port McNeil, and Summerland.

The RTK GPS system was then used to establish NAD 83 UTM (Zone 10) co-ordinates for most cross sections.

The horizontal accuracy of the GPS-located cross section points is 0.12 m relative to the provincial ACS stations.

Survey Control – Vertical

Elevations are referenced to British Columbia Water Resources Monuments in the CVD28, 1927 adjustment. Vertical control was adjusted to the following monuments based on published elevations:

601J, 839, 840, 842, 843, 844, 849, 850, 851, 852, 854, 856, 858, 1068, 1069.

The published elevations of these monuments were held as correct, and all surrounding points were adjusted so that the GPS-produced elevations correlated with the published elevations of the monuments. The required adjustment of surveyed benchmark results was minimal: the GPS survey found that all but one benchmark was within 0.02 m of published elevation, which is considered to be excellent given the age of the benchmarks.

The vertical accuracy of the GPS-located cross section points is 0.02 m relative to the ‘best fit’ of the above benchmarks.

Real Time Kinematic (RTK) GPS equipment was also used to obtain dyke crest, road, and ground spot elevation data at various locations as needed between Lillooet Lake and the top of the model study area.

SUPPLEMENTAL FIELD INVESTIGATION

In December 2001, a survey crew obtained detailed geometry information for eight bridges spanning the Lillooet River and tributaries. This information was used to update the bridge geometry in the Mike 11 model.

Also in December 2001, a separate crew used RTK GPS equipment to obtain dyke crest, road, and ground elevation data a various locations between Lillooet Lake and the top of the model study area (adjacent to Lillooet River cross section 54). Information was gathered where required to clarify modelling results (such as possible dyke or bank overtopping), or to provide more detailed input to the model (such as road elevations at the upstream end of Seymour Slough).

GRAVEL SAMPLING

Gravel sampling consisted of surface and subsurface sampling of gravel from the Meager Creek confluence to the bridge across Highway 99. Determining the size distribution of surface and subsurface material is important because they enter into competence and sediment transport calculations, and hence into the consideration of channel form and stability (Church et al., 1987).

Grid sampling was used to determine the size distribution of the surface layer. Using this method a grid or transect is established on a bar and grain sizes of 400 individual stones are measured at equal intervals. To remove bias due to differential sorting towards the surface of the gravel bar, subsurface samples were also carried out. Bulk or subsurface sampling involved sieving approximately 300 kg of sediment in order to obtain a representative grain size distribution. As subsurface sediment reflects all grain sizes transported during flood events, it is a more appropriate measure of average sediment in transport. In contrast, surface sampling is a better measure of the tractive forces required to mobilize the coarser pavement. Details on gravel sampling procedures can be found in Kellerhals and Bray (1971) and Church et al. (1987).

BANK EROSION INVESTIGATION

Lower reaches of Lillooet River were investigated by boat in October 2000. Eroding banks were photographed and videotaped for documentation and for later comparison with erosion rates determined from air photography (Section 5.4). Bank erosion has been limited largely to those sites that have not been riprapped or otherwise protected.

RESIDENT INTERVIEWS

Residents were interviewed with regard to flood damage. Most interviewees are long-terms residents and therefore have a good knowledge of the flood history of Pemberton Valley. Questions asked during interviews were the location and magnitude of flooding and loss of land through erosion. During interviews floodplain maps and air photos were shown to pinpoint sites of interest.

4.2 ANALYSIS

RIVER GEOMORPHOLOGY

The analysis of channel geomorphology had five principal components. First, the river was characterized morphologically. River morphology is important because it provides an interpretable impression of sediment movement and floodplain development, even before any further analysis is conducted.

Second, sediment transport and deposition were analyzed for upper reaches of Lillooet River. This was accomplished by linking observed grain size distributions with channel changes digitized from air photographs. A sediment budget for upper reaches was calculated from this exercise.

Third, changes in channel geometry were documented for lower reaches from repeated cross section surveys in 1969, 1978, 1985, 1993 and 2000. Changes were analyzed at each cross section site and as longitudinal changes following the river thalweg. This information was supplemented by limited survey data from 1945. A sediment budget was derived from this analysis and linked with results from the upper river.

Fourth, bank erosion rates for the 1971 to 2000 period were determined for the lower river based on floodplain maps and cross sectional data. These changes in planform were used to predict potential sites of future channel instability.

Fifth, changes in delta advance were analyzed from existing data and recent air photography. Changes were determined in longitudinal advance, aerial advance and volume increase.

BACKWATER ANALYSIS

Approximately 58.4 km of river were modelled, including Lillooet River (43 km), Birkenhead River (8.3 km), Green River (3.0 km), Ryan River (1.8 km), Miller Creek (1.2 km), and Pemberton Creek (1.1 km). The model was created in the Mike 11 (version 2000b) one-dimensional hydrodynamic modelling software, written and marketed by DHI Software (Denmark).

Data from the 103 cross section surveys were imported into the model, and extended by adding floodplain data digitised from 1 m contour maps. Floodplain limits for each cross section were determined following the same general approach used in the 1988 Provincial Government HEC2 modelling exercises.

Upper boundary conditions consisted of six discharge hydrographs incorporating 40 days of hourly discharge values. The lower boundary condition consisted of fixed lake levels, with the specific level depending on the type of model run being performed (calibration run, Q_{50} run, or Q_{200} run).

The model was calibrated to recorded 1991 high water marks using hydrographs derived from the shape of the 1991 flood event, as recorded by the Water Survey of Canada (WSC) Gauge 08MG005. This gauge is located at cross section 20.1 on Lillooet River.

Estimated Q_{200} and Q_{50} hydrographs were then run through the calibrated model. Resulting water levels were extracted and compared to surveyed bank and dyke elevations. Design flood levels (in 0.5 m increments) were interpolated for Lillooet River and Birkenhead River, and plotted on the colour digital orthophoto (flown in 1999) map figures in Appendix B.

4.3 STUDY PROCESS

The Steering Group met with the project team periodically to review study progress and interim deliverables, and to provide direction to the project. In addition to the primary Steering Group members listed in subsection 1.5, additional representatives from their respective organizations would attend meetings from time-to-time.

The Steering Group meeting dates were as follows:

Date	Location
July 19, 2000	KWL – North Vancouver
January 10, 2001	KWL – North Vancouver
August 15, 2001	KWL – North Vancouver
November 27, 2001	KWL – North Vancouver
January 15, 2002	Mount Currie
June 26, 2002	KWL – North Vancouver

In addition to the formal Steering Group meetings, numerous impromptu meetings and minor reviews were held with PVDD and Mount Currie representatives throughout the project.

A stakeholder meeting (by invitation) was held on January 15, 2002 at Mount Currie. The following organizations were represented at the meeting by a total of 24 attendees:

- Village of Pemberton (Mayor Elinor Warner and Mr. Bryan Kirk)
- Department of Fisheries and Oceans Canada
- School District No. 48 (Howe Sound)
- BC Rail
- Public Works and Government Services Canada
- Squamish Lillooet Regional District
- Lil'wat Fisheries Commission
- Creekside Resources
- Ministry of Water, Land and Air Protection
- Pemberton Valley Dyking District
- Mount Currie Band
- Kerr Wood Leidal

The following organizations were invited, but were not able to attend:

- Indian and Northern Affairs Canada
- BC Asset and Land Corporation
- Ministry of Forests
- Ministry of Transportation
- BC Hydro

A public open house was also held on January 15, 2002, also at Mount Currie. A total of 29 individuals signed the guest register and provided comments and feedback on preliminary study results. Feedback from the stakeholder meeting and open house was incorporated into the final report.

KWL attended a PVDD Board meeting on October 30, 2001 (in Pemberton) to present and review preliminary study results, and to hear the concerns of Board members and local residents. A presentation summarizing study results was made to Mount Currie

Band Council on October 22, 2002, and feedback on the draft report was obtained from Council for inclusion in the final report.

Section 5

Fluvial Geomorphology

KERR WOOD LEIDAL
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5. FLUVIAL GEOMORPHOLOGY

A primary objective of the study is to quantify the flood hazard posed by Lillooet River and tributaries, particularly where there is significant development on the floodplain. Although the river is dyked for approximately 45 km upstream of Lillooet Lake, it is intended to review the adequacy of the dykes to protect the developed areas from extreme flood events.

A primary tool for flood hazard assessment is hydraulic modelling. As documented in Section 7, such a model was used for this study. Hydraulic modelling of river systems, however, is limited in its use because it only provides a snapshot in time. Information on sediment transport, which governs channel form and morphology, and rates of channel change are generally not specifically addressed in modelling. For example, the level of protection afforded by dykes tends to be compromised over time due to the accumulation of gravel-sized sediment along the channel bed. Such a situation is presently occurring in lower reaches of Fraser River near Chilliwack, where gravel deposition over the past fifty years is up to 1 m in some areas, thus reducing the margin of protection provided by dykes (Church et al., 2000). This section investigates whether a similar situation is occurring at Lillooet River.

The section begins with a morphological characterization that provides an overview of the planform of Lillooet River and its main tributaries. Following is a discussion on sediment transport and deposition in upper reaches of Lillooet River, including an estimate of sediment transport rates. Then an analysis of changes in the channel geometry in lower reaches is discussed. These changes were determined from repeated surveys of river cross sections. Sediment transport rates are also estimated from the repeated surveys and compared with estimates from the upper reach. Following is an analysis of stream bank erosion along lower Lillooet River and its tributaries, which is an important exercise in evaluating potential changes in river alignment and the necessity for future river works. The section ends with an analysis of the advance of the Lillooet River delta into Lillooet Lake.

An analysis of sediment transport rates can identify long-term trends in sedimentation and, therefore, provide a tool for floodplain management. Simple and localized observations of channel geometry changes are insufficient to allow conclusions over future changes because neither sediment input nor outputs are known. Of particular concern in the Pemberton Valley is that increases in sediment input, caused principally through landslides in response to 20th century glacial recession, will cause an increase in the rate of sediment storage on the Lillooet River floodplain and aggradation of the river channel. This will ultimately cause an increase of river stage and the necessity for additional engineering measures to protect development on the floodplain.

5.1 MORPHOLOGICAL CHARACTERIZATION

The present morphology of the river is a single-thread, irregularly meandering channel that extends upstream of Lillooet Lake for about 43 km. The average channel width is about 110 m. Upstream of 43 km the channel gradient increases and the morphology of the river changes to a broad, braided system with an active channel that is up to 500 m wide (Photo 3, Appendix C). Lillooet River is gravel-bedded for most of its length, except for six to eight kilometres upstream of the lake where the channel gradient is no longer sufficient for gravel transport (some pea gravel does reach the delta but the volume is not significant).

For the purpose of this study, the river has been separately analyzed as upper and lower reaches. The upper river is defined as the reach from the confluence with Meager Creek (80 km) to the forest road bridge (40 km). Meager Creek was chosen as the upstream limit of the study because it is a very significant source of gravel to Lillooet River due to unusually high landslide activity in tributaries of Meager Creek (Photo 2, Appendix C). To analyze changes in the upper river over time, this reach has been divided into seven reaches herein referred to as Reaches 4 through 10. The lower river extends from the forest road bridge to the Lillooet Lake delta, and is separated into gravel (Reaches 2 and 3) and sand reaches (Reach 1). These separations are useful because of the distinct differences in channel morphology, availability of data and rural development between the reaches. Figure 5-1 shows the long profile of the river and its effect on channel morphology, while Figure 5-2 shows the locations of the various reaches.

Brierly and Hickin (1991) have described a downstream sequence of braided-wandering-meandering along lower Squamish River, which is very similar to Lillooet River. High sediment inputs in the upper Lillooet valley have contributed to its unique braided morphology in upper reaches (Photo 4, Appendix C). Braided rivers consist of a series of broad, shallow channels and bars, with elevated areas active only during floods, and dry islands (Miall, 1977). The characteristic feature of the braided pattern is the repeated division and joining of channels, and the associated divergence and convergence of flow, which contributes to a high rate of fluvial activity relative to other river types (Knighton, 1998). In general, there are four factors that contribute to the development of a braided planform:

- *Large volumes of sediment* either from upstream or channel banks. Braiding results when the stream has insufficient capacity to transport all the sediment supplied or grain sizes are too large for further downstream transport. Deposition of the excess material redirects flow against the banks, resulting in bank erosion and a wide, shallow channel.

- *Erodible banks* are an important source of sediment and allow a characteristic widened channel to develop. In laterally confined rivers, flow tends to become entrenched in a single channel, unlike braided systems where several zones of deeper flow develop.

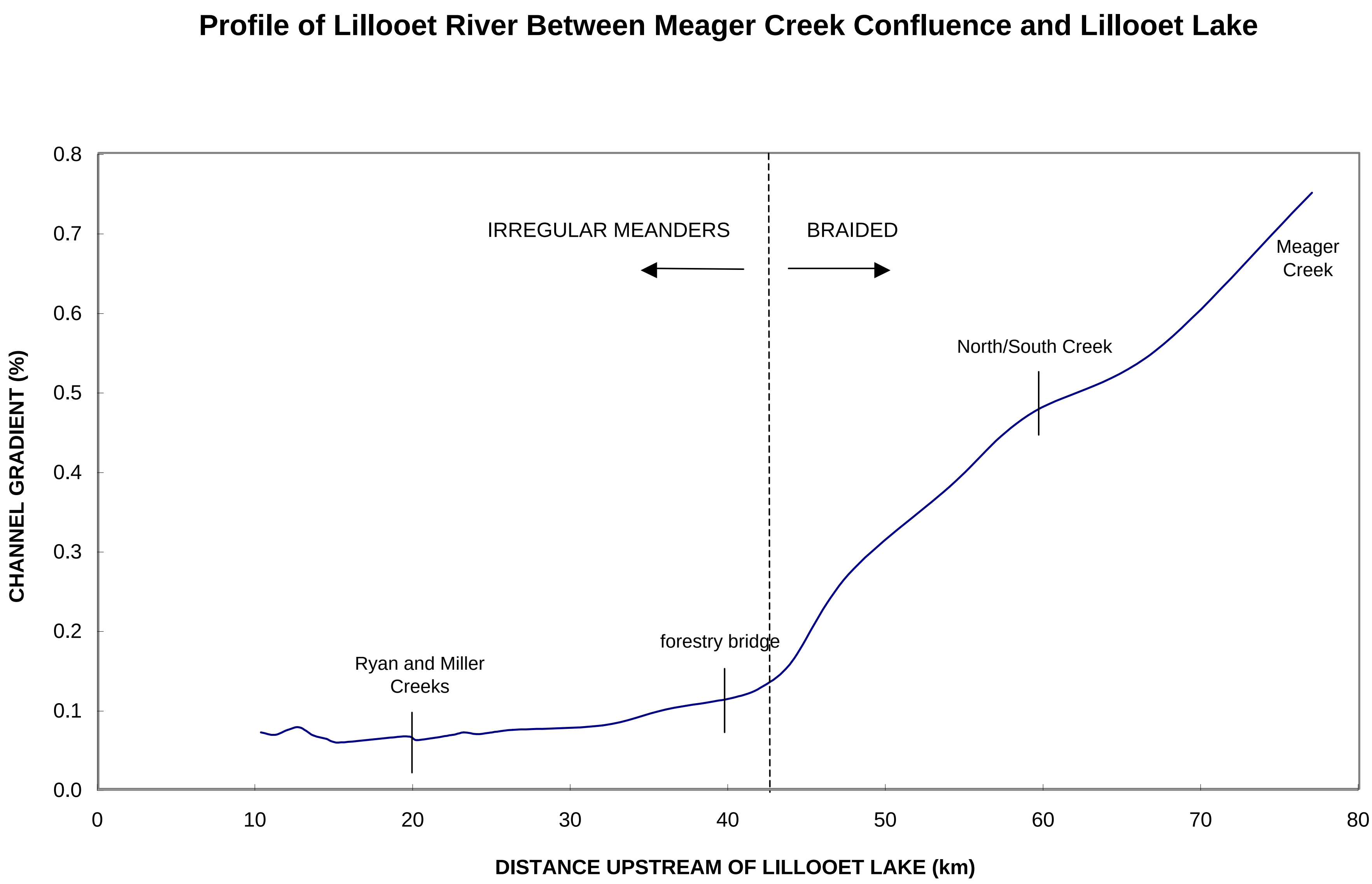
Profile of Lillooet River Between Meager Creek Confluence and Lillooet Lake
IRREGULAR MEANDERS
BRAIDED
Meager Creek
North/South Creek
forestry bridge
Ryan and Miller Creeks
CHANNEL GRADIENT (%)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0
10
20
30
40
50
60
70
80
DISTANCE UPSTREAM OF LILLOOET LAKE (km)

Figure 5-1

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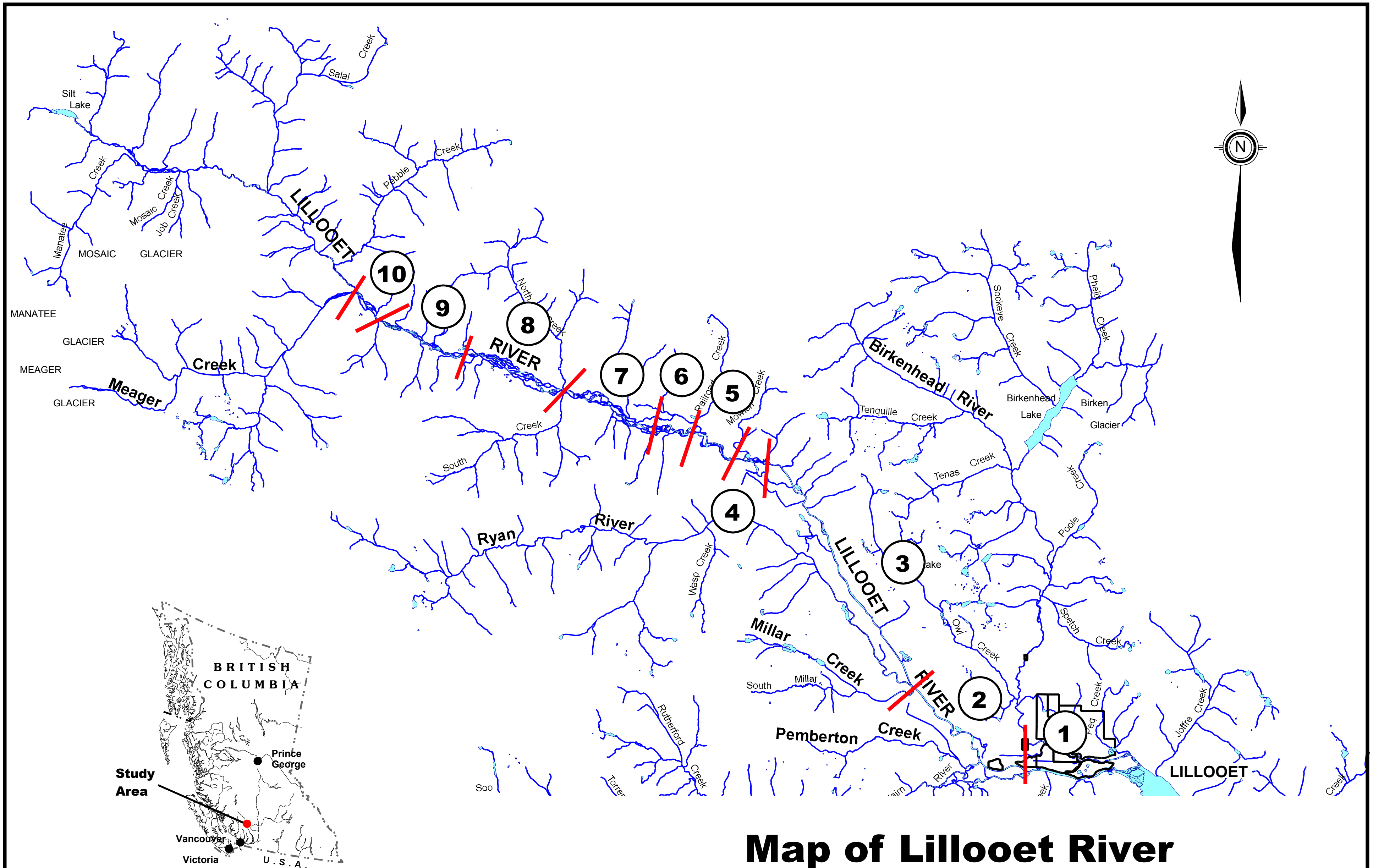

Location Map

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Map of Lillooet River Watershed Showing Reach Locations

Figure 5-2

- A *highly variable discharge* is closely associated with high rates of sediment supply, and hence braided systems. Rapid fluctuations in flow contribute to irregular sediment transport and bank erosion, both of which are characteristic of braided rivers.

- Braiding tends to develop when the *slope* is above some threshold value. Brierly and Hickin (1991) have described a downstream sequence of braided-wandering-meandering along the lower Squamish River where valley slope decreases from 0.0058 to 0.0015 and 0.0013 m/m respectively. However, high stream power (γQs) appears to be a more appropriate measure of the potential onset of braiding since braiding can occur at low slopes in very large rivers (Knighton, 1998).

All of these criteria appear to be met in the upper, braided reach of the Lillooet River, resulting in its characteristic planform. As with Squamish River, the transition to a braided morphology occurs at channel gradients in excess of 0.0013 m/m (Figure 5-1). Below this transition, the channel gradient decreases from about 0.0011 m/m at the forestry bridge to 0.00055 m/m at the delta.

Significant tributaries that supply gravel to the river include Meager Creek (80 km), North and South Creeks (60 km), Ryan River (20 km), Miller Creek (20 km), and Pemberton Creek (11.5 km). Gravel has been removed from the latter three creeks over the last few decades due to aggradation and reduced flood conveyance.

A lack of sediment larger than a few millimetres in the delta deposits indicates that gravel is not being transported into the lake. Gravel transport is therefore confined within a closed system and should accumulate along the channel bed of Lillooet River below the forestry bridge – given a continuous supply of gravel to the system. In other words, the gravel-sand transition represents the front of a large gravel fan. Significant upstream gravel deposition is required for the front of the gravel fan to migrate downstream. As such, the overall level of the channel bed should be increasing (in lieu of human intervention), particularly since extensive riprapping has effectively confined the river to its present course. The question is at what rate is the bed level rising.

The behaviour of gravel-sized sediment, which determines channel morphology, strongly contrasts that of finer sediment (sand and silt) which acts as wash load. Once entrained, this material moves primarily in suspension and has little impact on channel morphology, except as a superimposed deposit on floodplain surfaces and in backchannel areas. Most of the fine sediment transported by Lillooet River is deposited in Lillooet Lake, where it is responsible for a rapidly advancing delta front.

There have been no detailed studies on gravel transport in the river and the potential for significant gravel accumulations in the dyked reach raises several questions:

- Are there significant slugs of sediment being transported through the braided section of the river that could be transported into the dyked reach of the river in the next few decades?

- How has the level of the channel bed changed since dyke construction?
- On average, how much gravel is being transported into the dyked reach per year?

Channel mapping of historic air photographs is used to address the first question while an answer to the second question is provided in subsection 5.3 through the analysis of cross sectional data that exists for the dyked section of the river. Fortunately, the river has been surveyed regularly with surveys completed in 1969, 1978, 1985, 1993, and in November 2000. By comparing surveys, historical changes in bed level and estimates of sediment transport rates can be re determined.

A direct interpretation of bed level changes in the dyked reach is complicated by the removal of meanders in the late 1940s and lowering of Lillooet Lake (Figure 3-1). Because the total length of the river was shortened by approximately 5.5 km, the channel gradient of the river was increased. An increase in channel gradient usually causes a river to degrade since more shear stress is being applied to the river bed. Hence in the short-term surveys should indicate overall degradation (with downstream aggradation). In time, however, the river will achieve a new equilibrium and the channel bed will start to aggrade once again.

5.2 CHANNEL CHANGES AND SEDIMENT TRANSPORT IN UPPER REACHES OF LILLOOET RIVER

Historic cross sections allow an interpretation of changes in the dyked section of the river. However, there are no survey data for the braided reach of the river. This reach is of particular importance because it is a zone of active sediment transport and large volumes of gravel are in storage. Since river systems function in response to water and sediment inputs from the upstream catchment, the importance of the upper reach can not be overemphasized.

Due to their shallow nature, channels in braided rivers frequently shift positions and gravel transport within reaches is high. In this study it is important to analyze whether there is enough storage capacity in the braided section to contain a high proportion of the sediment supplied to the channel, or whether there are significant slugs of gravel being transported to the single-thread dyked reach. It is known that there are large inputs of gravel into upper reaches of the braided section due to unusually high landslide activity in tributaries of Meager Creek, which itself discharges into Lillooet River. If such a “wave” of sediment were to be eventually deposited in upper reaches of the dyked section, the bed would aggrade significantly and the dykes would provide considerably less protection during an extreme flood event. Determining the risk of sedimentation from upstream areas is crucial in developing an integrated flood hazard assessment.

The transfer of sediments through channel reaches is summarized by the general sediment budget equation:

$$V_0 = V_i - \Delta V \quad (1)$$

where V_0 is sediment output and V_i is sediment input into a reach. The storage term, ΔV, is a measure of the net difference between erosion of island and floodplain deposits into the active channel, reconstruction of islands and the floodplain by sediments deposited from the active channel, and scour and fill of gravel within the active channel (Ham and Church, 2000). Within the budget, a distinction is made between fine-grained sediment (wash material found in overbank deposits) and bed material. Changes in channel morphology reflect the transport of bed material, hence the wash load is ignored in this study.

The morphologic approach described in this section allows the approximation of sediment movement rates without the need of cross section measurements. Erosion and transport processes mobilize sediment stored in gravel bars and floodplain areas, which is then deposited downstream. Because this movement of sediment causes changes in channel morphology, measurements of planform changes can be used to estimate sediment transport rates. Popov (1962) and Neill (1971, 1987) were the first to develop this approach. Simply stated, the authors noted that material eroded along an outer concave bank was deposited on the next downstream bar (in the case of a simple meandering channel). In assuming that the average downstream travel distance of sediment was equal to one-half the meander wave length, estimates of bedload transport could be determined from planform changes.

Refinements of this technique have since been presented by Church et al. (1986), Carson and Griffiths (1989), Ferguson and Ashworth (1992), Martin and Church (1995), Lane et al. (1995), McLean and Church (1999), and Ham and Church (2000). In British Columbia, sediment budgets using this methodology have been carried out on Fraser River, Vedder River, and Chilliwack River. For this study the modified approach of Church et al. (1986) is employed, which is an extension of the simplified meander sediment transfer model to more complex morphologies. This concept is valid provided there is morphologic evidence for the sediment travel distance, L_t. Using this method, the sediment transport rate is estimated by the following equation:

$$G_b = (V_e / t)(L_t / L_r) \quad (2)$$

where G_b is the sediment transport rate, V_e is the volume of sediment eroded (or deposited), t is the time period of comparison, and L_r is the reach length.

The morphologic approach is in many instances favourable to conventional techniques of measuring sediment transport rates and volumes. It has been shown that bedload formulae that relate sediment transport to streamflow hydraulics and sediment characteristics have failed to predict sediment transport accurately (Gomez and Church, 1989), and to this date there is no satisfactory solution for predicting bedload transport.

For Lillooet River, changes in channel morphology have been related to bedload transport by estimating erosion and deposition rates from a sequence of historic air photographs. Channel features (bars, islands, banks) were mapped directly from historic air photographs and transferred to a Geographical Information System (GIS) to determine rates of channel changes over time. The GIS also provides measures of thalweg length, sinuosity, and rates of bank migration as well as showing trends of erosion and deposition along channel reaches. Areal changes in channel features (e.g. channel to floodplain) were converted to erosion and deposition volumes by estimating the depth of the mobile bed, which is the elevation difference between the bed surface along the thalweg and the top of adjacent bars and gravel banks. Bed material transport rates were then determined from the total scour (or deposit) over a length of reach equal to the average step distance of transport from one active bed zone to the next one downstream.

The morphologic approach assumes that a representative bed-material depth can be defined and that sites of erosion and deposition within study reaches remain distinct between surveys. The latter assumption does not always hold since compensating scour and fill is not an unusual occurrence, especially if the period between surveys is excessively long or a number of large flood events occur between surveys. This is especially true for braided rivers were the channels are very active and continually shifting.

Morphologic Approach to Sediment Transport

The first step of the morphologic approach involved channel mapping of old air photos. Photo records from five years were chosen for the study as shown in Table 5-1.

Table 5-1
Historic Air Photo Mapping of Upper Lillooet River

Year	Date	Scale	Flow (m^3/s)
1947	Aug. 12	1:35,000	294
1965	July 29	1:35,000	326
1977	Sept. 12	1:40,000	118
1988	Aug. 22	1:70,000	174
1994	July 28	1:20,000	315

Using an analytical stereoplotter (that allows the air photos to be mapped in three dimensions) and standard UTM georeferencing, planimetric features were mapped for all years. Mapping extends from the vicinity of the Meager Creek confluence to approximately 12 km downstream of the forestry bridge (and includes portions of Ryan

River above its confluence with Lillooet River). In total, approximately 65 km of channel were mapped for each year of aerial coverage. Planimetric features mapped include:

- gravel bars: depositional features that are devoid of vegetation and are inundated during relatively low magnitude floods;
- islands/floodplain: areas vegetated with woodlands or tall shrubs (taller than 3 m); these areas are generally dry even during high magnitude floods, with islands differentiated from the floodplain in that islands are situated within the active part of the channel (i.e. surrounded by water);
- mature bars: gravel bars that are at the same elevation as the floodplain and that show patchy or initial signs of vegetation;
- backchannels: wetted channels that are no longer connected to the main channel; and
- channels: the wetted perimeter of the river.

Examples of the channel mapping from 1947 and 1994 are included in Appendix D. Some of the 1947 air photos are of poor quality and channel features (principally gravel bars) could not be mapped precisely at all locations. Hence, some sections of the channel appear to consist of water only.

Having mapped the channel, the data was then transferred to a GIS environment (Arc/Info) where overlays were made of different years to accurately measure areal changes in channel features. Areal changes were converted to volumes by estimating the depth of the mobile bed, measured as the elevation difference between the bed surface along the thalweg and the top of adjacent bars and gravel banks. The depth from the gravel bar tops to the channel bed was remarkably consistent along the entire length of the braided section (approximately 35 km), and averaged about 3.0 m. At the downstream end of the braided section where the channel develops a meandering pattern this depth increases to 3.5 m. This consistency can be expected since there are no major tributaries situated between the confluence with Meager Creek and the forestry bridge. If a number of significant tributaries were situated downstream of Meager Creek, the scour depth of the river would be expected to increase downstream relative to the increase in discharge. Island and floodplain surfaces were generally situated about 0.5 m higher than bar tops. Table 5-2 shows how these depth measurements are used in conjunction with areal changes to estimate volumetric changes.

Table 5-2
Summary of Potential Channel Changes

1977 Feature	**1988 Feature**	**Process**	**Area** (m^2)	**Depth** (m)	**Volume** (m^3)
Channel	island/floodplain	deposition	A_1	3.5	$3.5^* A_1$
Channel	gravel bar	deposition	A_2	3.0	3.0^*A_2
Gravel bar	island/floodplain	deposition	A_3	0.5	0.5^*A_3
Island/floodplain	channel	erosion	A_4	-3.5	-3.5^*A_4
Gravel bar	channel	erosion	A_5	-3.0	-3.0^*A_5
Island/floodplain	gravel bar	erosion	A_6	-0.5	-0.5^*A_6
Note: Island/floodplain features also include mature bars.					

The process of determining volumetric changes is complicated by variations in discharge between each date of photography. A higher discharge will increase the average width of the water surface and correspondingly decrease the exposure of gravel bars, especially for the wide, shallow reaches found in this study. Between any two successive dates of photography, this leads to a false impression of erosion (higher water on later date) or deposition (lower water on later date). These changes must be accounted for prior to determining volumetric changes or transport estimates will be significantly biased. A separate exposure correction was determined for six major reaches that have been identified along the braided section. A number of correction factors were derived since variations in channel form and adjustments of flow depth and velocity to discharge may cause the correction gradient to change along the river (Church et al., 1986).

By sectioning the braided reach of the river into seven computational cells or reaches (designated as reaches 4 to 10 and shown on Figures D1 and D2), volumetric changes were determined for each reach for the following three periods: 1965 to 1977, 1977 to 1988, and 1988 to 1994. Unfortunately the 1947 air photos are of poor quality and channel features (principally gravel bars) could not be mapped precisely at all locations. Hence, volumetric changes could not be determined for the 1947 to 1965 period. However, the 1947 banklines and islands could be mapped with precision for most reaches allowing calculations of active channel width, from which useful comparisons could be made with other years.

Figure 5-3 is a simplified example of typical channel changes that occur in the river. Note the extreme channel changes that occurred in the 11 year period between 1977 and 1988. Changes of this magnitude are typical of braided rivers where channel shifting is a common occurrence due to the relatively shallow nature of the channels and large areas of exposed gravel bars. Erosion areas shown on the map either indicate a floodplain/island to channel or gravel bar to channel transition while deposition areas indicate the opposite transitions. To avoid unnecessary complexity, the two erosion and two deposition transitions have been delineated on the figure with the same fill symbols.

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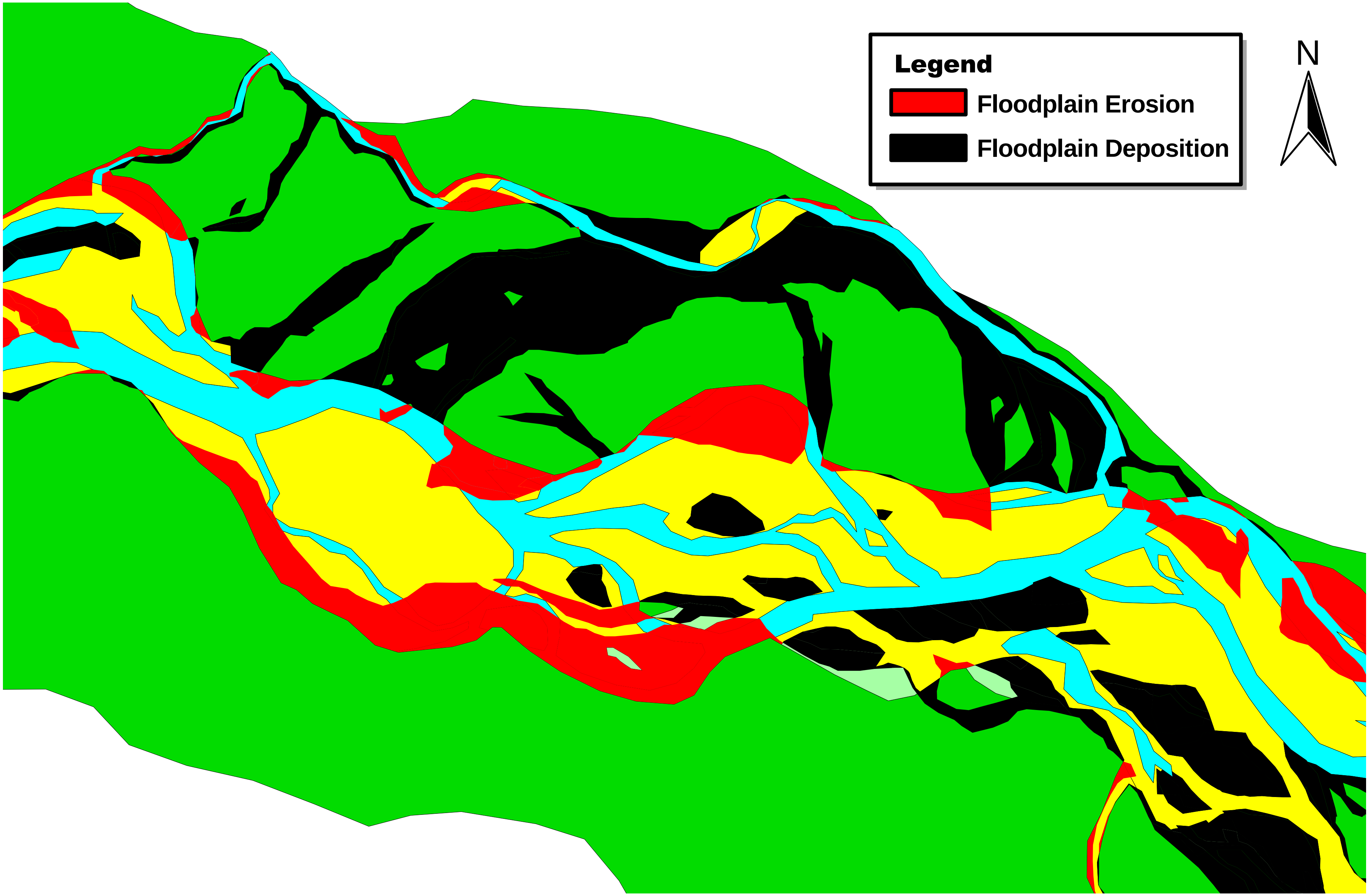

Typical Channel Changes in Lillooet River Downstream of North/South Creek Confluence, 1977 to 1988

Figure 5-3

RESULTS

Results for the morphologic analysis of the upper reach have been broken up into three sections: downstream variations in bed texture, downstream and temporal changes in channel width, and estimates of sediment transport rates.

Sediment Texture

A portion of the field work concentrated on measurements of sediment texture because grain size analyses can yield important information on sediment transport patterns. For example, coarsening of the substrate at tributaries is often an indication of significant inputs that should be accounted for in transport calculations. The rate of downstream fining is also a useful indicator of the river's ability to move coarser sediment at a specific location, thereby influencing channel form and stability. As such, surface and subsurface sampling was conducted in the gravel reach of the river (i.e. Reaches 3 to 10). Section 4 summarizes the methods used in sampling, with detailed results presented in Appendix E.

Results from the sampling program are shown in Figure 5-4. Plotted D_{50} values represent the mean grain size of the sample for which 50% of the sediment is finer-grained. D_{84} values (84% of the sediment is finer-grained) represent an upper size limit for the sample, with increasing grain sizes being much less common and hence less statistically significant. In geomorphic studies, both values are commonly used measures of grain size distribution.

For the most part, there are few surprises in the observed grain size distributions. An exponential decrease in sediment size in a downstream direction is obvious for both the surface and subsurface samples. Strong downstream fining trends are typical for gravel-bed rivers and indicate that a majority of the coarsest sediment is confined to upper reaches. Although this downstream fining trend can be interpreted as a result of particle abrasion, it generally represents differential sorting (i.e. shear stresses are insufficient to move the largest particles further downstream).

There is no apparent coarsening of sediment immediately downstream of the confluence with North and South Creeks (Photo 5, Appendix C). Both of these creeks have large debris flow fans resulting in significant channel constriction. If either of these systems had been active within the past 50 years, an increase in grain size would be expected immediately downstream, as the river would be incapable of transporting the largest introduced sediment.

Downstream of the forestry bridge, bulk sampling indicates an average grain diameter smaller than 20 mm. Although relatively fine, this grain size is sufficiently coarse to accumulate along the channel bed and not be transported through to the lake. Approximately 25 to 30% of the subsurface material is sand-sized below the forestry bridge with percentages decreasing in an upstream direction (Figure 5-5). Measures of sediment texture are important for both potential instream channel work and for future

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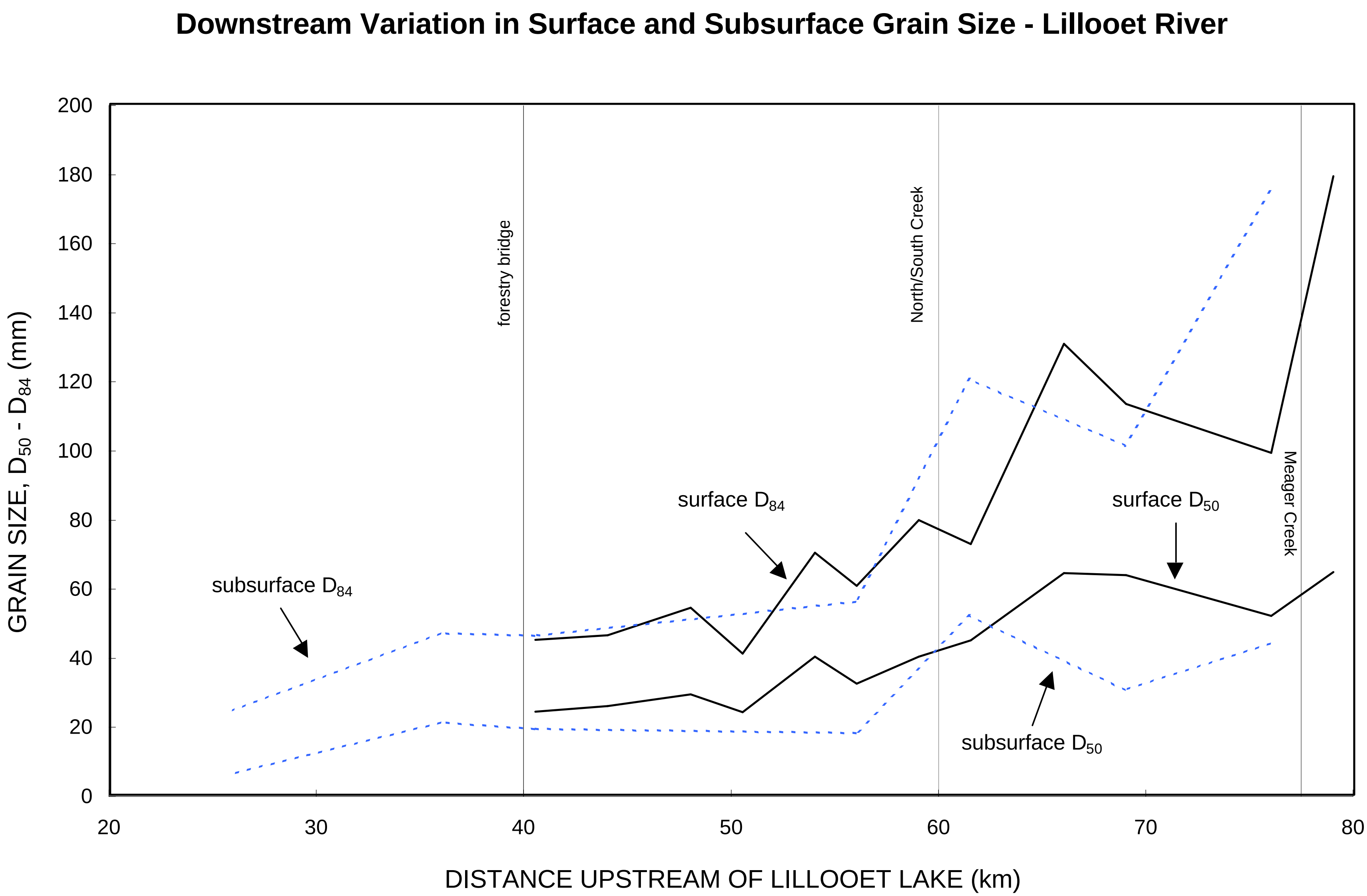


Figure 5-4

studies that may wish to monitor a potential downstream increase in grain sizes (i.e. because the channel gradient has been steepened, gravel may be moving further downstream).

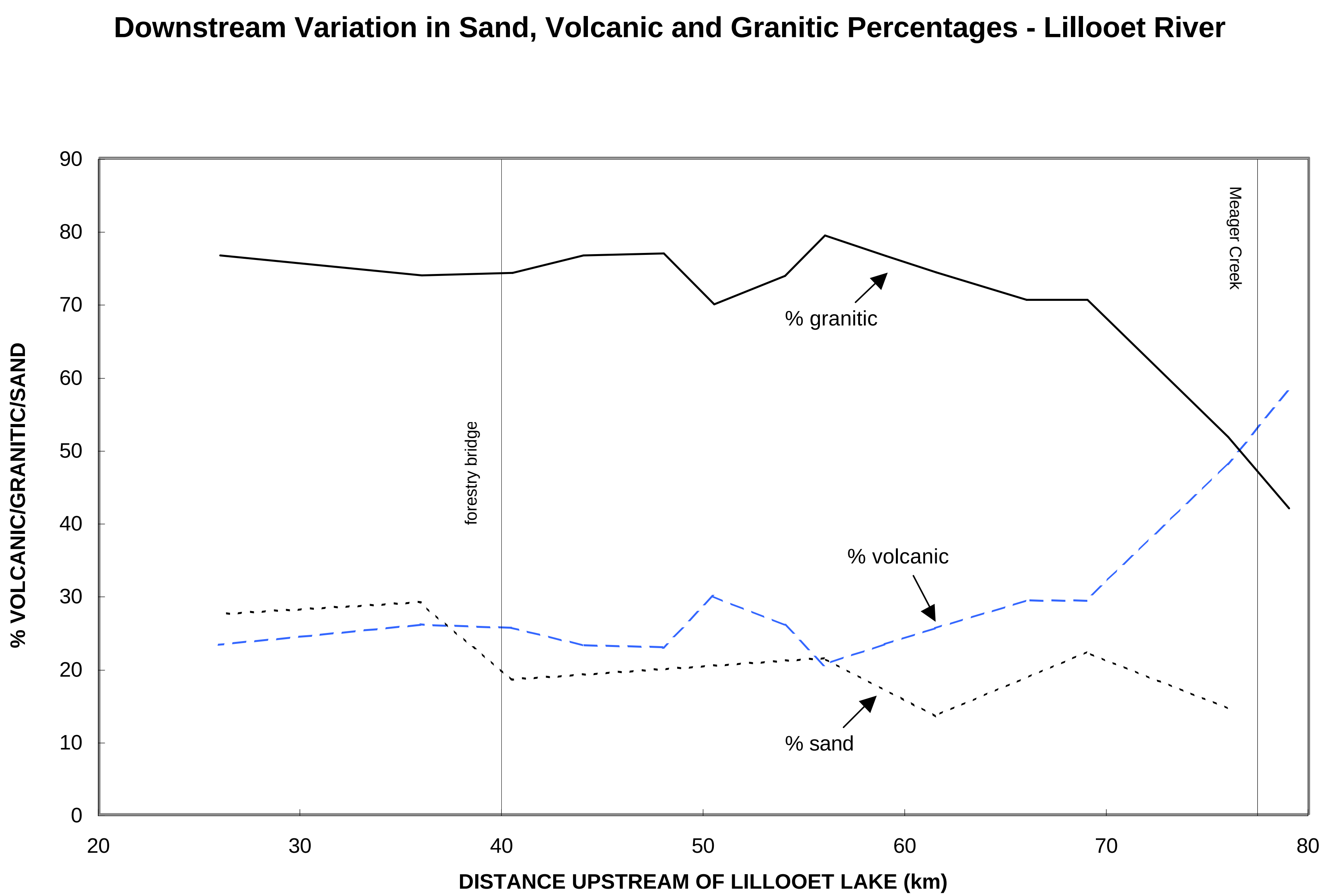
During surface sampling, individual gravel and cobble clasts were also classified according to lithology. Clasts were identified as either volcanic or plutonic (metamorphic or sedimentary rocks occurred in negligible numbers). Figure 5-5 shows the percentages of volcanic and plutonic sediment with distance upstream of the Lillooet Lake delta. Immediately downstream of Meager Creek the percentage of volcanic material is approximately 60%, but within 10 km the percentage decreases to less than 30%. Given that Meager Creek is a highly active fluvial system and volcanic bedrock is a common occurrence within its watershed, a high percentage of volcanic sediment in the mainstem is expected.

A rapid decrease in volcanic material downstream of this point can be interpreted in two ways. Because volcanics weather more rapidly than plutonic rock, the decrease could be explained by rapid abrasion of volcanic clasts into sand-sized sediment. If this was the case, the impacts of introducing large volumes of sediment from Meager Creek may be less in comparison to granitic source rock. That is, the overall bed level would increase less rapidly due to particle abrasion. Alternatively, recent debris flows may have deposited large volumes of sediment with a high percentage of volcanics. In this case, the slug of sediment may not have had time to disperse downstream, resulting in the observed trend.

Active Channel Width

Channel width should remain roughly constant over time if the magnitude of the dominant channel forming discharge remains constant, but will change if the river experiences a large flow. Increases in width reflect active bank erosion or removal of islands. Such activity is generally related to large flow events or the introduction of large volumes of sediment to the channel (i.e. unstable, sedimentation zones). Decreases reflect floodplain/island construction or vegetation of gravel bars and typically represent more stable reaches.

Figures 5-6 through 5-9 illustrate changes in active channel width downstream of the Meager Creek confluence for various time periods: 1947-1965, 1965-1977, 1977-1988, and 1988-1994. An obvious trend toward a narrower channel is apparent downstream of 43 km between 1947 and 1965. This represents the effects of channel straightening during the late 1940’s and is discussed in more detail in the following section. For the other reaches and time periods, fluctuations in active channel width appear to be random at first glance. Table 5-3 summarizes changes in active channel width as reach-averaged values.



Downstream Variation in Sand, Volcanic and Granitic Percentages - Lillooet River
% VOLCANIC/GRANITIC/SAND
DISTANCE UPSTREAM OF LILLOOET LAKE (km)
0
10
20
30
40
50
60
70
80
90
20
30
40
50
60
70
80
forestry bridge
Meager Creek
% granitic
% volcanic
% sand

Figure 5-5

Table 5-3
Changes in Active Channel Width of Lillooet River – 1947 to 1994

Reach	1947 (m)	1965 (m)	1977 (m)	1988 (m)	1994 (m)	Percent Change 1947-65	Percent Change 1965-77	Percent Change 1977-88	Percent Change 1988-94
3	173	93	89	85	87	-45.8	-4.6	-4.8	2.9
4	199	155	127	133	135	-22.2	-17.9	4.3	1.3
5	176	216	211	221	226	22.6	-2.3	4.7	2.5
6	285	291	327	327	297	2.1	12.5	0.0	-9.3
7	346	300	348	329	338	-13.2	15.9	-5.6	2.8
8	371	326	299	312	292	-12.3	-8.3	4.4	-6.3
9	344	256	237	279	297	-25.6	-7.6	18.0	6.2
10	335	311	277	286	361	-7.4	-10.9	3.5	26.2

Changes in active channel width are greatest for the 1947-1965 period with significant reductions noted for Reaches 7 through 10 (-7 to -26%). Decreases in channel width reflect vegetation of gravel bars and typically represent more stable conditions (i.e., a series of low flow years or reduced sediment inputs). A significant decrease in active channel width in upper reaches is consistent with the occurrence of the 1931 Devastation Creek debris flow (5 x 10^6 m^3) in the Meager Creek watershed. Although the entire volume of the debris flow would not have entered Lillooet River, a significant proportion was deposited into the mainstem. This is apparent from the channel mapping of 1947 where a remnant deposit of the debris flow is visible on the opposite bank of the Meager Creek confluence (Figure D-1, Appendix D).

If only 20% of the debris flow entered Lillooet River, 1 x 10^6 m^3 of sediment still represents an extremely large influx. The immediate response of the river would have been a large increase in channel width to accommodate the sediment influx. As the river adjusted to the increased load (principally by local aggradation and downstream sediment transfers) and sediment input rates returned to more "typical values", the active channel width would have decreased. The observed reductions in channel width in upper reaches for the 1947-1965 period and to a lesser extent the 1965-1977 period are consistent with this hypothesis.

After a period of relatively modest channel activity, the upper reaches appear to have been more active the past two decades with significant increases in active channel width for Reaches 9 and 10 since 1977. The rate of mass movement in the Meager Creek watershed has increased the past two decades in response to glacial retreat (Bovis and Jakob, 2000) and the observed increase in channel width for upper reaches of Lillooet River is consistent with this trend. However, it is also important to note that average channel widths in Reaches 9 and 10 were greater in 1947 than in 1994 (Table 5-3). That is, the air photograph record indicates that Lillooet River was more active in the past than it is at present.

Measurements of active channel width are also useful for determining whether there is a large slug of sediment moving downstream toward the meandering reach of the river. As shown on Table 5-3, there have been no large increases in channel width for lower reaches (Reaches 4 to 6) since 1977. A large unstable sedimentation zone migrating downstream would be expected to show up in the air photograph record as a pronounced increase in channel width.

Sediment Transport Rates

Volumetric changes were determined for Reaches 4 to 10 for the following three periods: 1965 to 1977, 1977 to 1988, and 1988 to 1994. Equation 2 was then used to transform these deposition and erosion volumes into sediment transport estimates. However, this equation requires an estimate of the sediment travel distance or step length (L_t). Using Lillooet River as an example, Church and Jones (1982) took channel width as a surrogate measure of local deposition and noted that the braided channel consisted of a sequence of sedimentation zones, connected by short reaches with less vigorous depositional activity. A similar pattern was observed in analyzing the channel width, as demonstrated by Figures 5-6 to 5-9. For each reach, there are regular peaks in channel width and the average distance between these peaks is considered to be the sediment travel distance.

Table 5-4 provided below is a summary of sediment transport estimates for the three time periods. Estimates have been provided using both the total erosion and depositional volumes. Appendix F provides a detailed summary of the calculations, including erosion and deposition volumes, reach lengths and sediment travel distances.

Table 5-4
Sediment Transport Estimates for Upper Lillooet River

Reach	**1965 to 1977**		**1977 to 1988**		**1988 to 1994**	
	Erosion transport rate (m^3/yr)	Deposition transport rate (m^3/yr)	Erosion transport rate (m^3/yr)	Deposition transport rate (m^3/yr)	Erosion transport rate (m^3/yr)	Deposition transport rate (m^3/yr)
4	-	-	-	-	-	-
5	31,300	31,500	36,000	29,700	34,700	43,200
6	29,800	36,300	34,400	33,400	91,400	93,300
7	35,000	36,300	44,900	45,600	89,500	92,500
8	30,200	36,100	39,600	37,400	82,400	87,900
9	17,000	14,900	32,000	23,400	78,400	68,800
10	-	-	-	-	-	-

Of note in the above table is the lack of sediment transport estimates for Reaches 4 and 10. Both of these reaches consist of one sedimentation zone (Figure 5-9) and it is difficult to determine an average step length for sediment transport. As a result, sediment transport rates were not estimated for either reach.

713-001\Drawings\Excel Figure\WidthB.xls

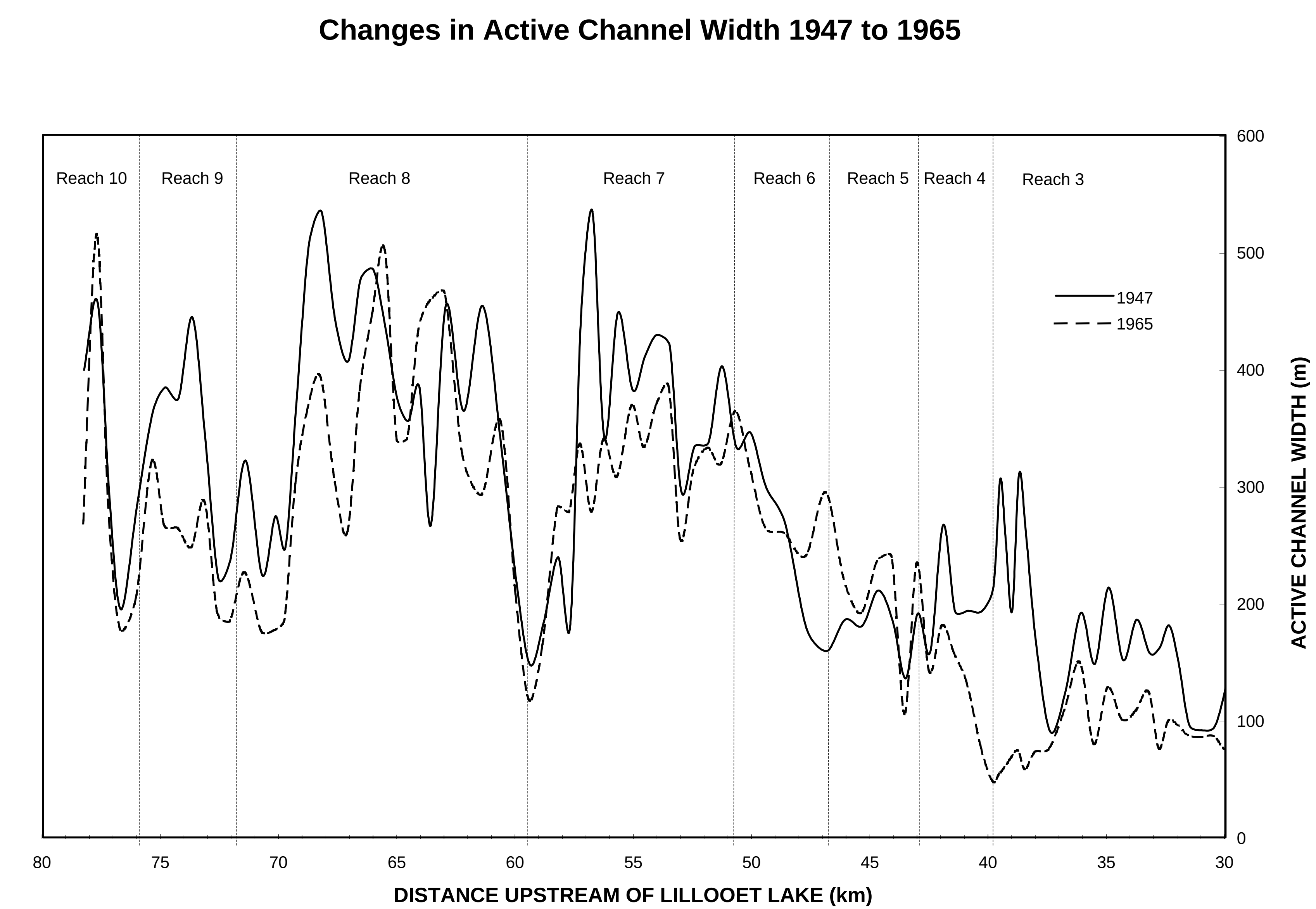


Figure 5-6

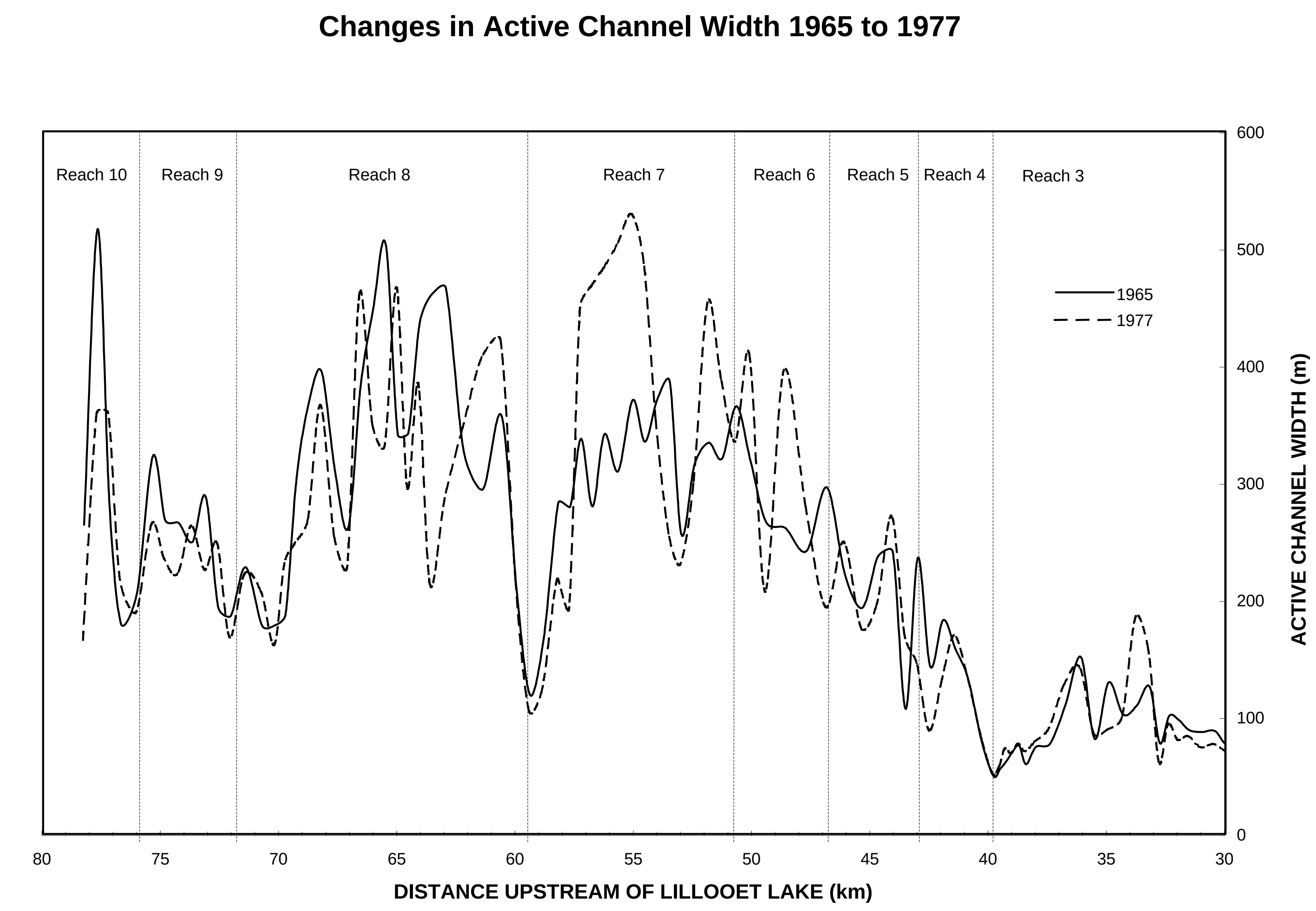

Changes in Active Channel Width 1965 to 1977
Reach 10
Reach 9
Reach 8
Reach 7
Reach 6
Reach 5
Reach 4
Reach 3
1965
1977
600
500
400
300
200
100
0
ACTIVE CHANNEL WIDTH (m)
80
75
70
65
60
55
50
45
40
35
30
DISTANCE UPSTREAM OF LILLOOET LAKE (km)

Figure 5-7

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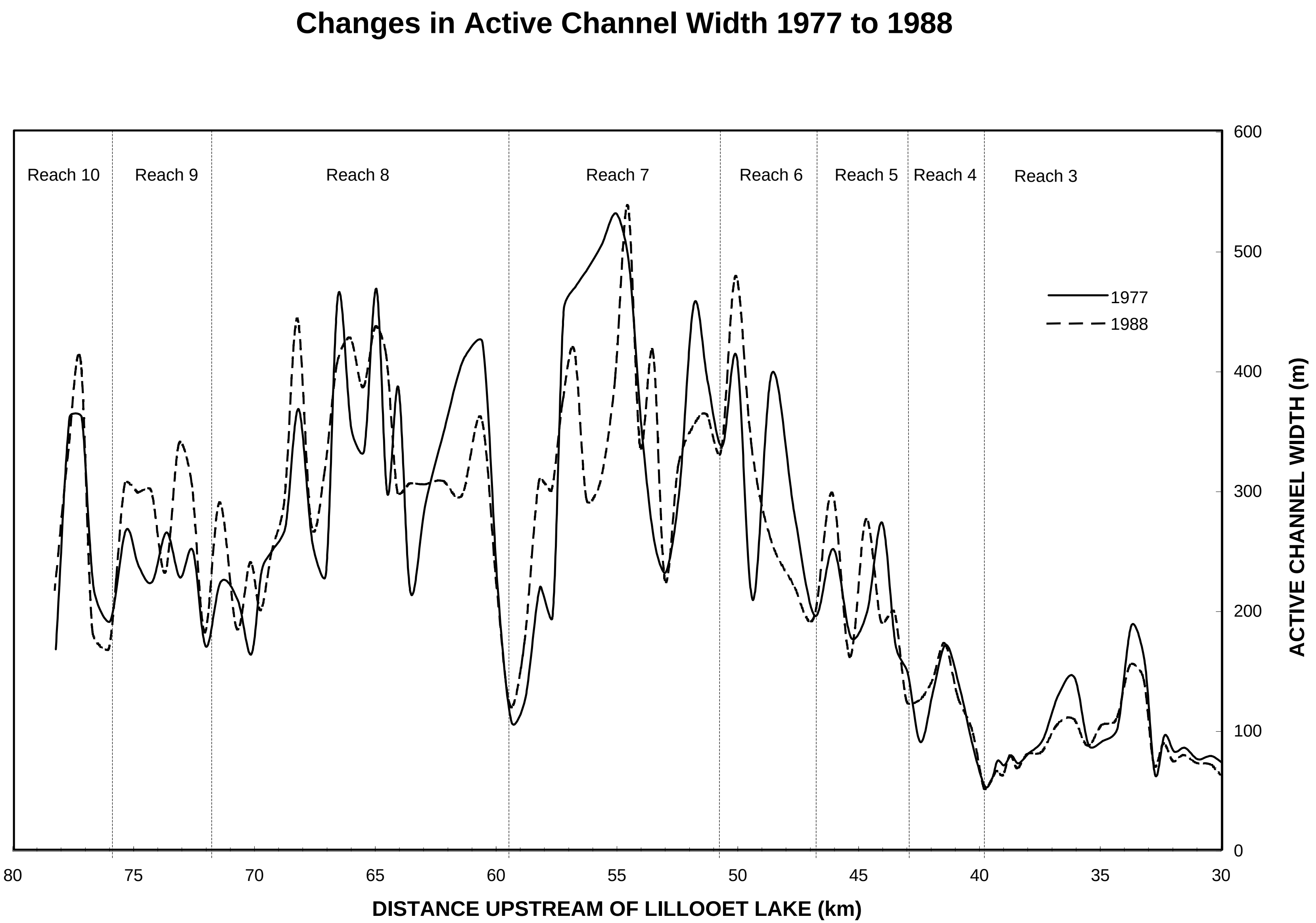

Figure 5-8

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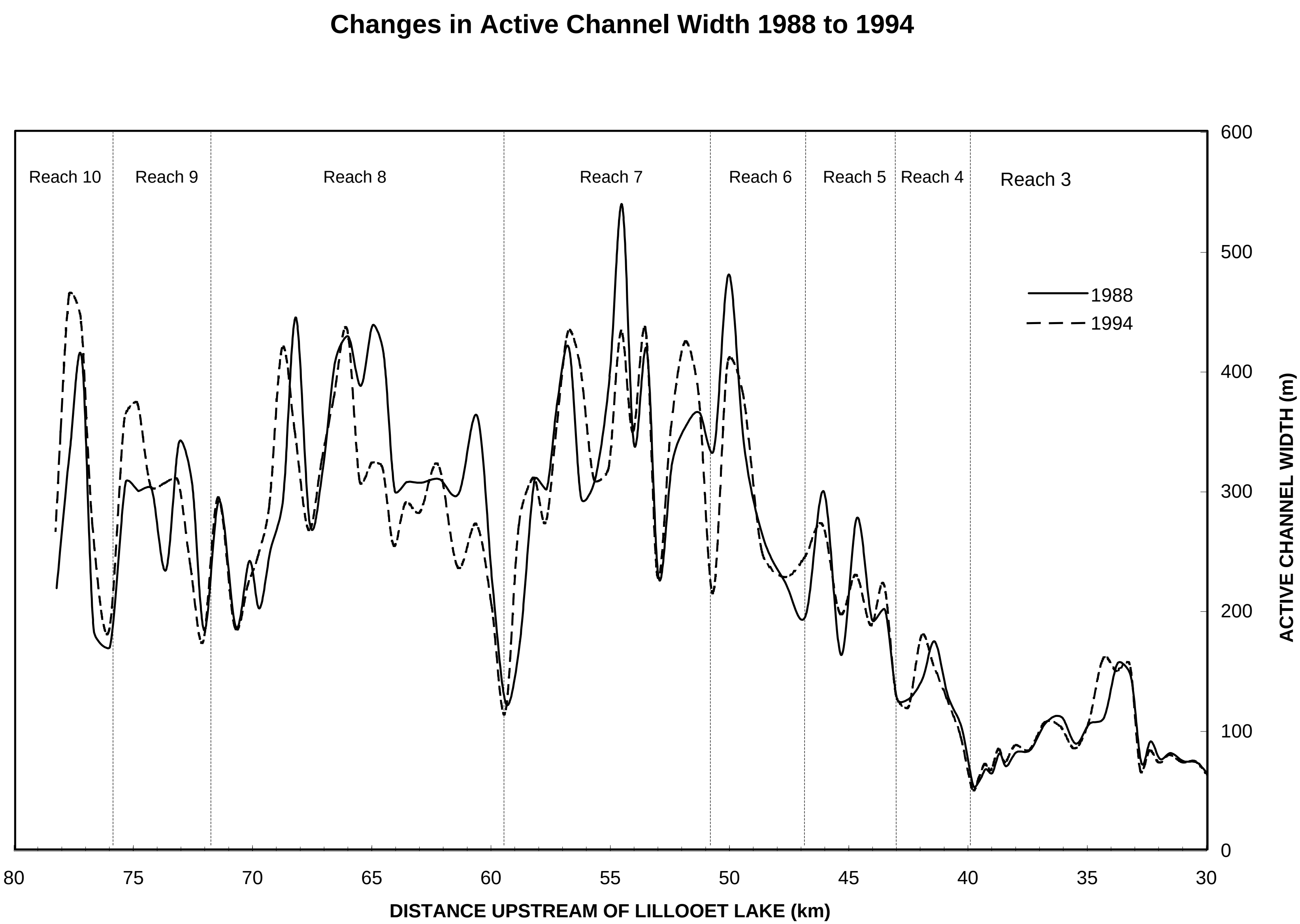


Figure 5-9

Reach 5 is of the greatest interest since it is a good measure of the amount of sediment being transported past the forestry bridge into the single-thread dyked reach. The results indicate that between 30,000 and 40,000 m^3/yr of sediment is being supplied to downstream reaches. This range represents a lower bound estimate since the morphologic approach assumes that sites of erosion and deposition within study reaches remain distinct between surveys. This assumption does not always hold since compensating scour and fill is not an unusual occurrence, especially if the period between surveys is excessively long or a number of large flood events occur between surveys. This is especially true for braided rivers where the channels are very active and continually shifting.

For Reach 5, it appears reasonable that there is minimal compensating scour and fill between the dates of channel mapping. Unlike the upper reaches, Reach 5 is transitional between meandering and braided, and is more appropriately classified as wandering (Photo 6, Appendix C). Channel changes in wandering rivers are considerably less frequent in comparison to braided reaches. Thus, consistent sediment transport estimates would be expected for all three periods despite survey periods ranging between 6 and 12 years. As shown on Table 5-4, this is the case with sediment transport estimates ranging between 30,000 and 43,000 m^3/yr. Slightly lower values were determined for the longer survey periods indicating that a shorter time period (i.e. 1988-94) is preferred for sediment transport estimates. The relatively low annual estimate of 30,000 to 40,000 m^3/yr is consistent with channel changes observed in lower reaches (see Section 5.3).

In terms of morphology, Reach 6 falls within a wandering to braided planform and as such the morphologic approach should be valid despite relatively long periods between channel mapping. For the 1988-94 period, sediment transport is estimated at 90,000 m^3/yr while for the other time periods the estimates are considerably less (approximately 35,000 m^3/yr). These values could reflect natural variations in sediment transport but the lower bound estimates are probably minimized due to compensating scour and fill over a longer time period.

Moving upstream into more active reaches, it is apparent that the long periods between channel mapping makes the morphologic approach unsuitable for estimates of sediment transport. The estimates summarized in Table 5-4 are much too low for an active braided system and represent extreme lower bound values. Higher sediment transport rates for the braided reaches were obtained for the shortest time period (1988-94), which is not unexpected since there is less chance of compensating scour and fill with short time periods. Channel mapping in successive or alternate years would provide a better estimate of sediment transport rates in the braided reaches.

CONCLUSIONS

An analysis of channel changes in the upper reaches of Lillooet River results in the following conclusions:

1. Measurements of active channel width from 1947, 1965, 1977, 1988 and 1994 appear to indicate that large sediment inputs from Meager Creek are incorporated into the floodplain by vertical aggradation. The effects of the 1931 Devastation Creek debris flow (5×10^6 m^3) on Lillooet River were pronounced but there is no indication that the associated instability has migrated downstream as far as the forestry bridge. This would indicate that a high percentage of the introduced sediment remains within the braided reach. This is consistent with one of the aggradation models proposed by Jordan and Slaymaker (1991) that postulated greater rates of aggradation for the upper reaches.

2. Despite the shortcomings of the morphologic approach in estimating sediment transport rates for the braided reaches, reasonable values have been determined for Reaches 5 and 6 where the river is in transition from a braided to meandering morphology. Results indicate that approximately 30,000 to 40,000 m^3/yr of sediment is being transported past the forestry bridge into lower reaches of the river. Based on observed channel changes in lower reaches (the focus of the next subsection), this estimate is considered reasonable.

3. There is no indication that sediment transport rates into the lower meandering reach will increase significantly in the next decade as a result of large sediment inputs from Meager Creek.

5.3 CHANNEL CHANGES AND SEDIMENT TRANSPORT IN LOWER REACHES OF LILLOOET RIVER

The previous section detailed changes in the upper reaches of Lillooet River that have occurred over the past half-century. Based on an analysis of historic air photos, it is estimated that approximately 30,000 to 40,000 m^3/yr of gravel is being transported from the upper, braided section through to lower, dyked reaches. An estimate of sediment transport into the lower reach is important because the level of protection afforded by dykes tends to be compromised over time due to the accumulation of gravel-sized sediment along the channel bed.

However, one of the limitations of the sediment transport estimate provided is that compensating scour and fill can occur between survey dates. Hence, estimates of sediment transport using the morphologic approach can be underestimated. Fortunately, there is a downstream limit to gravel transport in Lillooet River. A majority of gravel is not transported beyond the confluence with Green River, 6 to 8 km upstream of Lillooet Lake. That is, there is a point of zero transport and any gravel being transported past the forestry bridge must accumulate along the channel bed unless it is extracted from the bed. Repeated cross section surveys in the lower reach facilitate an additional *average* estimate of sediment transport, which can then be compared with results from the upper reach.

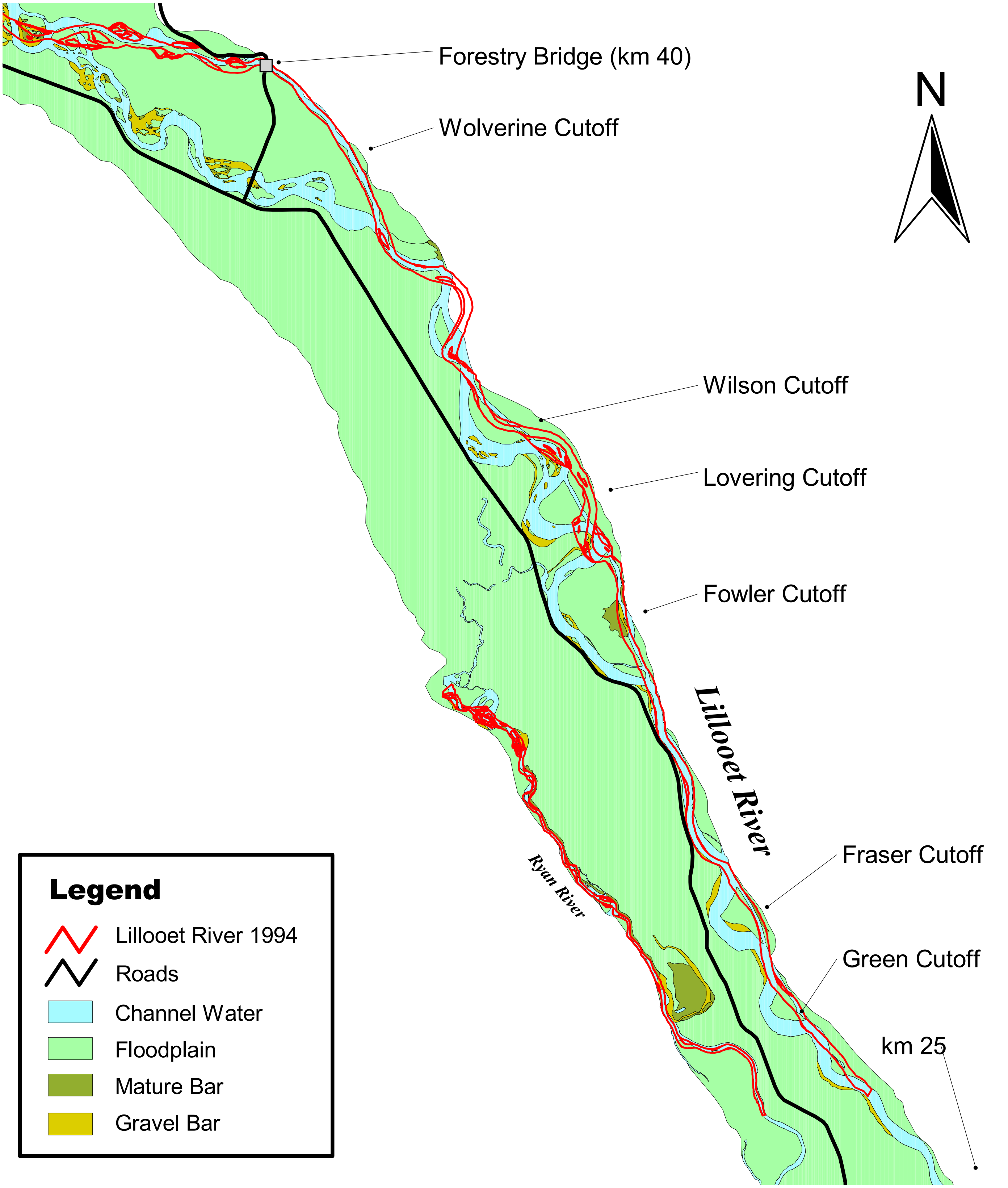
As such, this section quantifies the short and long-term morphologic responses of lower reaches of Lillooet River to natural and anthropogenic influences. This includes an analysis of channel adjustments to engineering works in the late 1940s and an estimate of sediment transport rates between 1969 and 2000.

CHANNEL ADJUSTMENTS TO 1946-1953 ENGINEERING WORKS

As described in Section 3, extreme channel modifications were carried out on Lillooet River in the 1946 to 1953 period. The modifications included straightening of the channel by cutting off meanders (Figure 3-1) and lowering the water level of Lillooet Lake by approximately 2.5 m. The latter was accomplished by dredging the lake outlet at Lillooet Narrows. Meander cutoffs were created by trenching a narrow straight ditch and then blocking the former course of the river, forcing water to follow the new alignment (see Figure 3-2). Subsequent channel widening occurred by natural erosion processes. While the former channel was bordered by natural gallery vegetation (cottonwood trees, shrubs and bushes), the new channels had little or no riparian vegetation. The lack of vegetation, namely the bank stabilizing effect of roots along the watercourse, attributed to the rapid widening of the channel.

Figure 5-10 is a channel map of Lillooet River from 25 km to 42 km based on 1947 photogrammetry. The original meanders are still visible as are the excavated cutoffs, which would have been trenched within a year or two of the date of photography. Superimposed on the figure are 1994 banklines of the river, demonstrating the extreme change in channel alignment that followed these works. By 1965 the cutoff meanders were almost completely isolated from the mainstem and had largely infilled with fine sediment. The most dramatic changes are the MacKenzie and Wolverine Cuts, the latter cut involving redirection of the main flow into a large side channel. Unlike the other meander cutoffs, the Wolverine Cut was privately constructed prior to the commencement of PFRA works in 1946.

The practice of channelizing streams to control flooding and drain wetlands for farming has been common in the past, but is becoming increasingly controversial. As noted by Keller (1976), an increase in the slope of a river by straightening will disrupt the quasi-equilibrium between streamflow, sediment concentration and channel characteristics. Channel adjustments can be described in terms of stream power by Lane's (1955) relation:



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KERR WOOD LEIDAL associates limited CONSULTING ENGINEERS

Engineering Study for Lillooet River Corridor

Project No.	Date
713-002	December 2002

600 0 600 1200 1800
Scale in Metres

1947 Channel Map of Lillooet River Between km 25 and 42 with 1994 Banklines Superimposed

Figure 5-10

$$QS \propto Q_S D_{50}$$

where Q is water discharge, S is channel slope, Q_S is the bed material load, and D_{50} is the median size of the bed material. The above equation indicates that an increase in channel slope will increase sediment transport rates and/or the size of the transported sediment must increase.

Slope adjustments are often the dominant response when cutoffs are constructed in an alluvial channel (Biedenharn, 2000). Degradation migrates upstream of the cutoff so that the slope flattens to re-establish an equilibrium slope at a lower elevation. However, the reach downstream of the cutoff aggrades due to the increased sediment supply from the degrading reach upstream. The result of these adjustments is an overall decrease in slope (from the imposed slope) as the channel attempts to absorb the imposed increase in energy conditions created by channel straightening. Lane (1947) divided the morphological response of erodible channels to cutoffs into two phases: an immediate response following the cutoff and a subsequent response that takes place gradually and over a long time period. Channel adjustments as described above have been observed in South Fork Forked Deer river in West Tennessee (Simon and Robbins, 1986) and the lower Mississippi River (Winkley, 1977; Biedenharn, 2000).

Historic Survey Data

Following the extensive channel works, there have been few attempts to monitor the resulting channel adjustments. Channel surveys from 1969, 1978, 1985 and 2000 are well documented in provincial records. Unbeknownst to many, however, Lillooet River surveys were also completed in 1913 and 1945.

Prior to the construction of the meander cutoffs, twenty-four cross sections were surveyed by the Water Development Branch of the Federal Department of Agriculture (under the auspices of the Prairie Farm Rehabilitation Administration). The cross sections were surveyed in April 1945 and extend from the delta (XS I) to the downstream end of Salmon Slough at km 38 (XS XXIV).

The cross section data were discovered in the offices of the PVDD in a large binder. The cross section data, along with water and thalweg profiles, are transcribed onto linen sheets. The cross section locations are marked on a general topographic plan of the Pemberton Valley, which has been divided into three sheets (also transcribed onto linen). Sheets 1 and 2 extend from Lillooet Lake to the old confluence with Ryan River. The topographic data on these two sheets, which includes the channel planform, is based on a 1913 survey by Cameron and Crowe. [Apart from the reference to Cameron and Crowe on the 1945 mapsheets, it is unknown who commissioned the 1913 survey.] Topographic and channel planform data on Sheet 3, which extends from the confluence with Ryan River to the downstream end of Salmon Slough, is based on a 1945 survey.

Most of the 1913 survey was also discovered on a large mapsheet that extends from Lillooet Lake to the railway crossing. The remainder of the survey up to the confluence with Ryan River was not located. The 1913 survey includes floodplain contours at 1 foot intervals, bathymetric data (cross sections spaced approximately 300 m apart) and the channel planform (including bars, islands and banklines).

Results from the 1913 survey have not been incorporated into this study, but the 1945 cross sections are very useful for comparative purposes. Table 5-5 lists the approximate location of the 1945 cross sections with respect to the contemporary cross sections and the chainage upstream of XS 0.3.

Table 5-5
Approximate Locations of 1945 Cross Sections

1945 Cross Section Identifier	Relative Location	Distance Upstream from Lillooet Lake (km)
I		
II	XS 1.1	1.45
III	XS 5	4.1
IV	~150 m u/s of XS 8	6.7
V	~ 300 m d/s of XS 11	8.5
VI	~ 250 m d/s of XS 14	10.9
VII	~ 200 m d/s of XS 18	14.2
VIII	XS 19.2 (BC Rail bridge)	15.6
IX	XS 20.1 (WSC gauge)	16.6
X	~ 400 m u/s of XS 22	18.1
XI	XS 25	19.9
XII	along old section of Lillooet River (now Ryan River)	n/a
XIII	along old section of Lillooet River (now Ryan River)	n/a
XIV	old confluence of Lillooet River and Ryan River	n/a
XV	½ way along abandoned (cutoff) Lillooet Slough	n/a
XVI	~ 500 m u/s of XS 32	25.2
XVII	surveyed on Green Cutoff (now abandoned), XS 36.1 lies adjacent	28.1
XVIII	XS 39	30.5
XIX	surveyed near downstream end of Fowler Cutoff (now abandoned), XS 41 located ~ 350 m d/s	32.2
XX	surveyed at confluence of North Ryan	32.8

1945 Cross Section Identifier	Relative Location	Distance Upstream from Lillooet Lake (km)
	River and Fowler Cutoff, XS 42 lies adjacent	
XXI	surveyed at maximum curvature of Lovering Cutoff, XS 43.1 lies adjacent	34.2
XXII	XS 45	35.7
XXIII	XS 46	36.4
XXIV	XS 48	37.6
Distance upstream from Lillooet Lake is measured from XS 0.3 (km 0).		

A direct comparison of the 1945 survey with more recent cross sections is problematic due to extreme channel changes in some reaches and the fact that the cross sections are not typically in the same location. However, as noted in the above table there are enough coincident cross sections to analyze channel adjustments.

Lower River Adjustments

For Lillooet River, channel adjustments can be separated into distinct reaches. Upstream of the delta there was significant channel degradation in response to lowering the lake level by 2.5 m. As shown on Figure 5-11, 0.5 to 3 m of degradation was observed between the delta and km 13 for the 1945-1969 period. These are probably near maximum values since experiments by Begin et al. (1981) have shown that the ultimate effect of base level lowering by a given amount is degradation by the same amount. Channel profiles provided in Figure 5-11 were obtained from a study conducted by graduate students in the Department of Civil Engineering, University of British Columbia (Cormie et al., 1970). The data is based on cross section drawings and water level profiles compiled by consulting engineers Burnett & McGugan. Due to difficulties in matching up cross sections, however, their report cautions that the profiles should be interpreted with caution.

[The data gathered by Cormie et al. appears to be different than the PFRA 1945 cross section data. The number of 1945 points shown on Figure 5-11 far exceeds the density of the 1945 PFRA soundings. This implies that additional survey data was compiled for the lower river by the aforementioned Burnett & McGugan.]

The flood benefits of lowering the lake level by 2.5 m were short-lived however. Wester (1970), in an internal Water Management Branch memorandum, noted that the delta had extended some 600 m since the lowering of the lake in 1946. Because of this extension, the water surface slope decreased and up to 2.5 m of sediment re-deposited in a reach of the lower river. As shown on Figure 5-11, there is little degradation between 3 km and 5 km and near the delta. It was concluded that any further dredging of the lower river would only have very short term benefits, and rapid degradation would renew the cycle of aggradation and channel surface slope reduction.

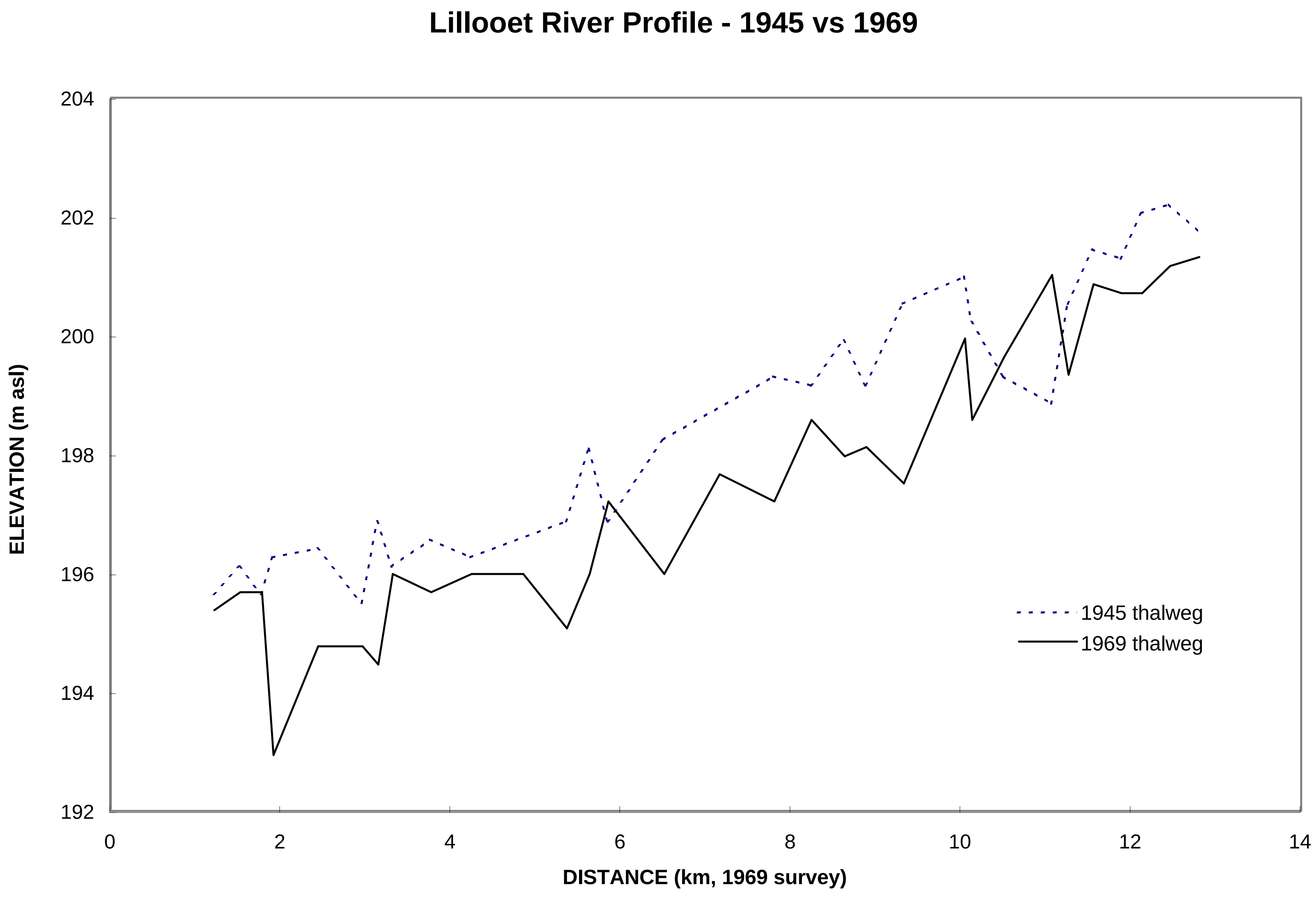

Lillooet River Profile - 1945 vs 1969
ELEVATION (m asl)
204
202
200
198
196
194
192
0
2
4
6
8
10
12
14
DISTANCE (km, 1969 survey)
1945 thalweg
1969 thalweg

Figure 5-11

This is not a surprising result when once considers that upstream gravel deposition exerts a considerable influence on downstream deposition patterns in the sand reach. The downstream extent of gravel deposition currently occurs in the general vicinity of the Green River confluence (8 km). Downstream of this point the channel slope is insufficient to transport significant quantities of gravel. When the lake level was lowered by 2.5 m, rapid degradation of the sand bed occurred upstream of the lake due to local steepening of the slope. Over time the abrupt break in slope migrated upstream, flattening along the way (the rate of degradation reaches a peak relatively quickly but then declines slowly over time and also decreases with distance from the outlet). In the case of Lillooet River, the headward degradation would have eventually encountered the more resistant gravel deposits. Because the gravel deposits were near their downstream threshold of transport, the local increase in slope would have been insufficient to induce significant degradation (unlike sand the eroded gravel could not be flushed into the lake).

The gravel deposits also influence sand deposition downstream. Because the base level of the gravel deposits can not be changed without extensive removal, dredging of the lower sand reach would have only short-term benefits. Sand would accumulate in the excavated zone because the trenching would not induce upstream degradation in the gravel reach. That is, the gravel deposition acts as a base level and the river returns to equilibrium by infilling of the excavated downstream area.

Adjustments to Meander Cutoffs

While channel adjustments for the 13 km stretch upstream of Lillooet Lake are relatively well documented, there have been no published accounts of channel adjustments in the vicinity of the meander cut-offs. A report by Sutek Services Ltd. and Kellerhals Engineering Services Ltd. (1989) provides some details. The report addresses gravel supply and removal in fisheries streams, with Lillooet River being one of the case studies. The authors state that “It has generally been assumed by the Water Management Branch that straightening and the consequently increased slope would produce profile adjustment and degradation upstream of the engineered reach. This is contradicted by survey evidence. The Water Management Branch attributes the lack of degradation to resistant materials underlying the river bed but it could also be due to cross sectional adjustments to engineering modifications.”

However, the recently uncovered documentation at the offices of the PVDD shows the opposite is true - cross sectional changes were extreme upstream of the confluence of Ryan and Miller Creeks following channel straightening.

Figure 5-12 compares selected 1945 cross sections with more recent surveys at four locations between MacKenzie Cut (km 20) and the downstream end of Salmon Slough (km 38). At each of these locations, the 1945 data is compared to survey data from either 1969, 1985, 2000 or a combination of these three dates. [Survey data from 2000 is limited for upper reaches and as such is not available for all four plots.] The attached

Upstream Response of Lillooet River to Meander Cutoffs

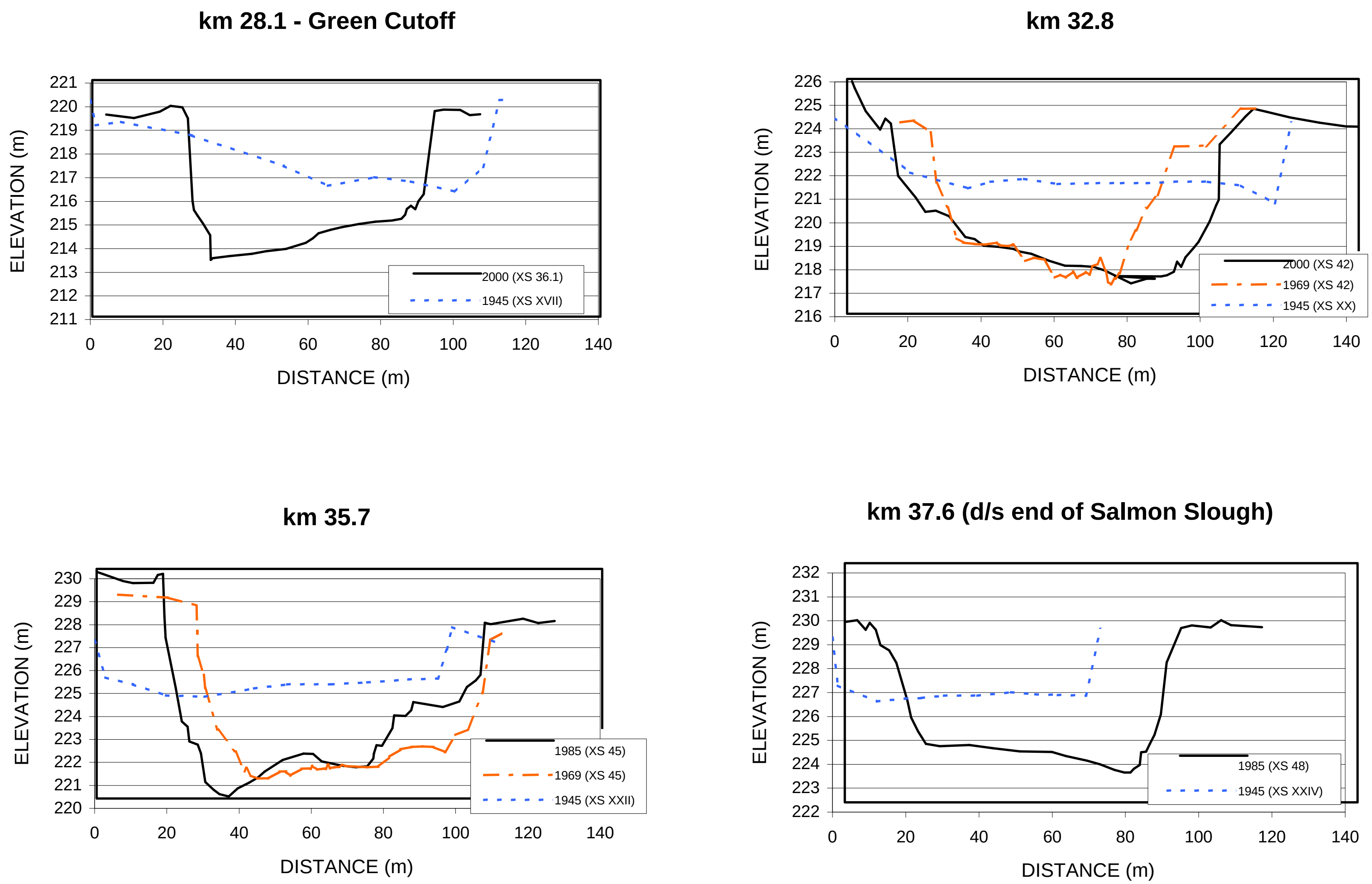


Figure 5-12

figure clearly shows that 3 to 4 m of degradation has occurred as a result of the channel straightening. A majority of the degradation would have occurred rapidly as indicated by the 1969 data in two of the plots.

Significant reductions in active channel width have also occurred in the vicinity of the cutoffs. Figure 5-13 shows changes in active channel width in Reaches 3 and 4 for the 1947-1994 period. In Reach 3 below the forestry bridge (km 27 to 40), the average channel width has decreased from 173 m to 87 m while the average width of Reach 4 has decreased from 199 m to 135 m (Table 5-3). A majority of this adjustment occurred in the 1947-1965 period (Figure 5-6).

Miller Creek to Green River Adjustments

The observed pattern of channel degradation upstream of the meander cutoffs is consistent with the expected response. That is, degradation migrates upstream of the cutoff (in this case upstream of MacKenzie Cut at km 20) as the slope flattens to re-establish an equilibrium slope. For the overall slope to decrease (from the imposed slope), the reach downstream of the cutoffs should aggrade due to the increased sediment supply from the degrading reach upstream.

However, degradation is also observed between the downstream end of the cutoffs and the Highway 99 bridge (km 12). Up to 2.5 m of degradation is noted for two sections where comparisons are possible between the recent monumented cross sections and the 1945 data (Figure 5-14). The observed degradation is in response to the lake being lowered at the time of the meander cutoffs.

If degradation has occurred downstream of the cutoffs, where has all the increased sediment supply from the upstream degradation been deposited?

The answer appears to lie in the gravel-sand transition, which presently occurs downstream of the Green River confluence (km 6 to 8). Yet, annotations on the 1945 mapsheets indicate the presence of sand bars immediately downstream of the WSC gauge at km 16. This implies that the engineering works increased the channel gradient sufficiently that gravel could be transported further downstream and that the gravel-sand transition has migrated downstream about 8 km in the past forty-five years. In this case, sand has been evacuated from lower reaches and been replaced by the increased gravel supply from upstream.

Degradation has still occurred downstream of the meander cutoffs because the channel gradient had to adjust not only to channel straightening but the lowering of the lake level by 2.5 m. Without the lake lowering, the observed downstream migration of the gravel-sand transition would still have occurred but the degradation downstream of the cutoffs would have been replaced by aggradation.

Changes in Active Channel Width in Reaches 3 and 4 - 1947 to 1994

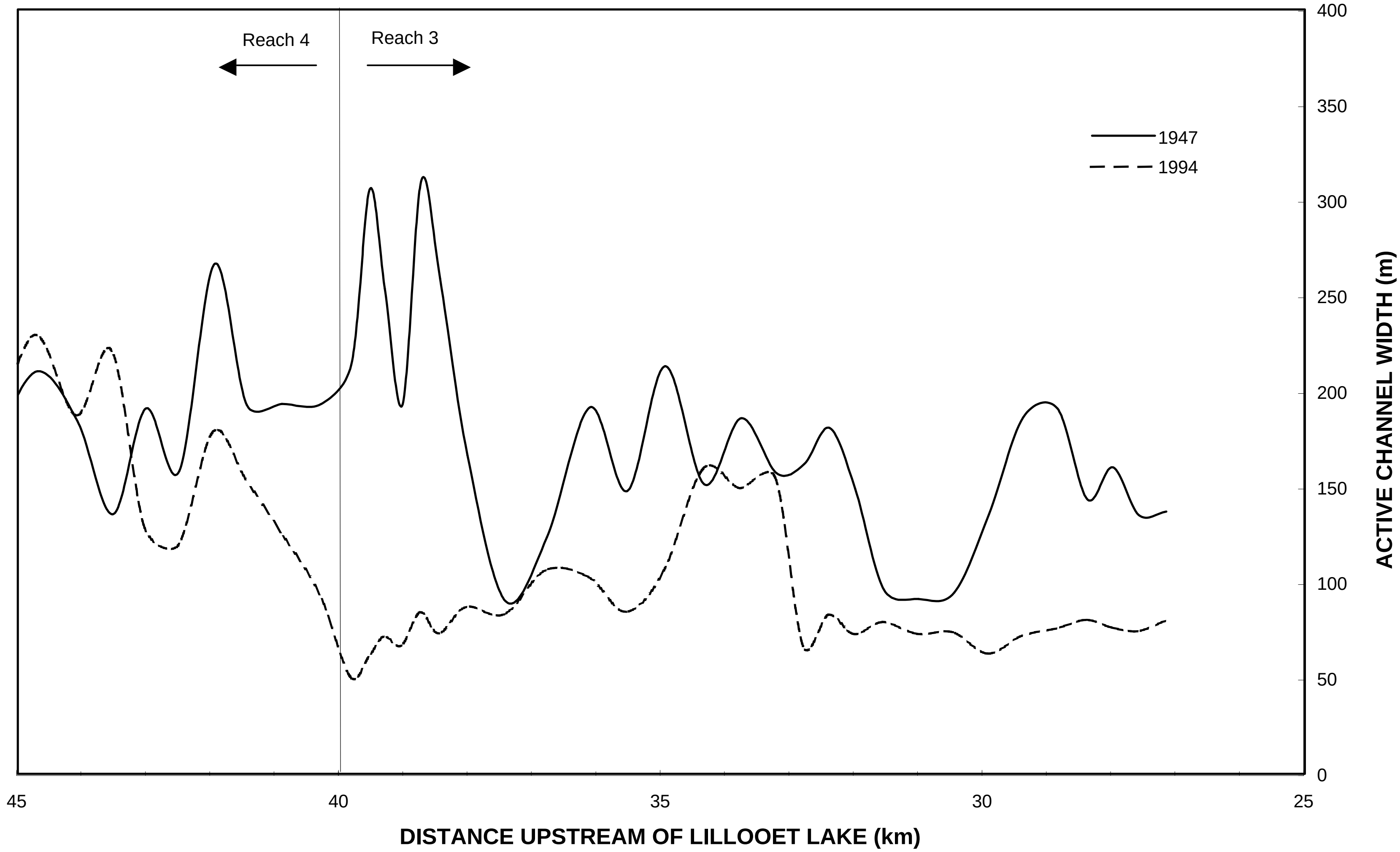
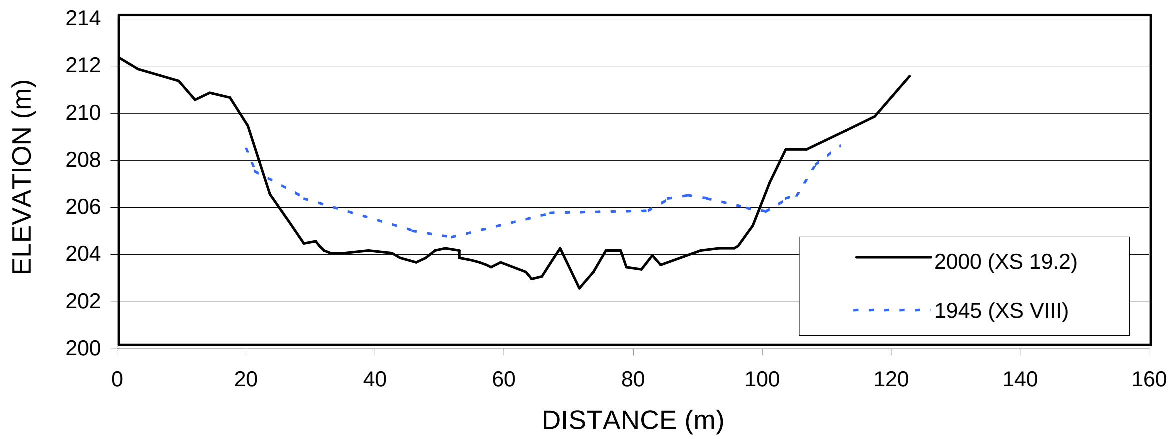


Figure 5-13

Downstream Response of Lillooet River to Meander Cutoffs

km 15.6 - BC Rail Bridge



km 19.9

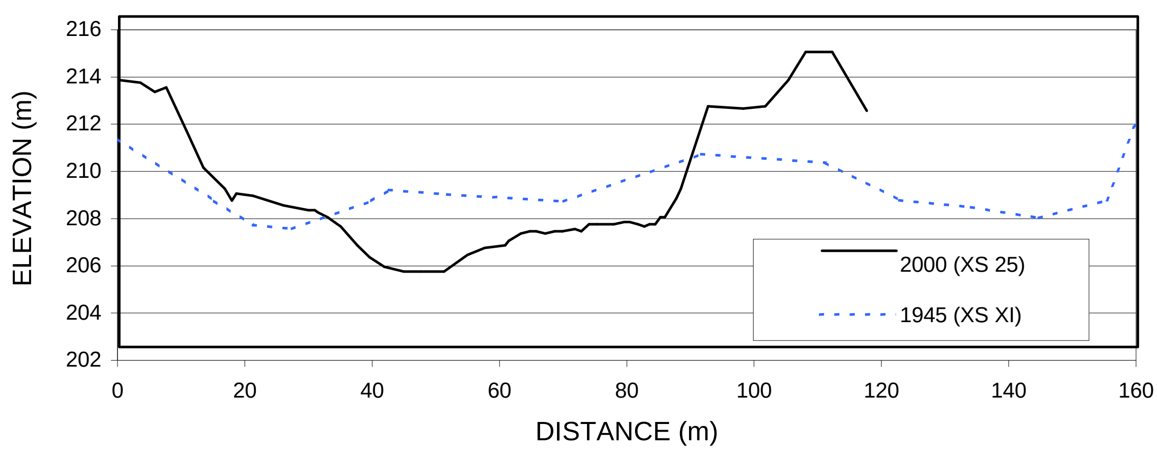


Figure 5-14

Table 5-6. Cross-sectional changes along lower reaches of Lillooet River - 1969 to 2000

Cross-section	Chainage	WIDTH (m)					AREA (m^2)					AVERAGE DEPTH (m)					MAXIMUM DEPTH (m)				
	(m)	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000
0.3	0	-	-	712	-	714	-	-	2011	-	1843	-	-	2.8	-	2.6	-	-	5.8	-	5.6
1	926	201	276	301	-	279	670	1117	976	-	947	3.3	4.0	3.2	-	3.4	4.6	5.0	6.6	-	7.4
2	1,895	157	184	172	-	-	696	788	738	-	-	4.4	4.3	4.3	-	-	5.9	6.0	6.1	-	-
3	2,590	118	124	124	-	126	588	647	619	-	624	5.0	5.2	5.0		5.0	8.5	9.5	8.5	-	8.1
4	3,310	126	133	153	-	-	552	630	622	-	-	4.4	4.7	4.1	-	-	6.1	7.0	6.3	-	-
5	4,063	148	148	173	-	184	637	698	711		702	4.3	4.7	4.1		3.8	5.6	6.6	6.4	-	5.2
6	4,929	151	153	154	152	154	641	693	681	653	679	4.3	4.5	4.4	4.3	4.4	6.2	6.4	6.3	5.9	6.2
7	5,685	125	126	126	126	126	554	610	598	591	602	4.4	4.8	4.7	4.7	4.8	5.4	5.6	5.7	5.7	5.8
8	6,520	205	234	249	236	-	708	847	788	758	-	3.5	3.6	3.2	3.2	-	5.8	5.5	5.4	5.6	-
9	7,420	100	100	116	115	-	526	540	575	592	-	5.3	5.4	5.0	5.1	-	7.0	7.7	7.8	8.0	-
9.1	7,599	-	-	201	198	198	-	-	793	755	753	-	-	3.9	3.8	3.8	-	-	7.6	8.5	7.6
9.2	7,786	-	-	104	117	126	-	-	469	490	528	-	-	4.5	4.2	4.2	-	-	5.8	5.4	5.4
10	8,005	150	183	181	182	182	416	544	536	511	554	2.8	3.0	3.0	2.8	3.0	7.2	5.3	6.0	5.5	5.6
11	8,840	77	75	75	76	83	302	335	327	334	372	3.9	4.5	4.4	4.4	4.5	5.5	6.5	6.6	5.4	5.3
12	9,520	94	91	104	105	103	360	406	437	416	422	3.8	4.5	4.2	4.0	4.1	4.7	5.0	6.0	5.8	5.8
13	10,313	81	84	86	92	93	317	381	386	389	394	3.9	4.5	4.5	4.2	4.2	5.6	5.6	5.8	5.1	5.2
14	11,204	84	84	90	92	92	330	370	368	390	418	4.0	4.4	4.1	4.2	4.5	5.6	6.3	5.9	5.5	5.9
14.1	11,398	-	-	88	97	97	-	-	363	399	394	-	-	4.1	4.1	4.0	-	-	7.9	5.9	6.0
14.2	11,513	-	-	157	147	146	-	-	542	533	551	-	-	3.4	3.6	3.8	-	-	4.5	5.1	6.7
15	12,014	111	113	102	102	102	330	372	367	366	386	3.0	3.3	3.6	3.6	3.8	5.2	5.0	4.8	5.0	5.2
15.2	12,247	-	-	116	118	121	-	-	561	536	537	-	-	4.8	4.5	4.4	-	-	9.8	8.9	8.5
15.4	12,373	-	-	123	112	113	-	-	413	415	390	-	-	3.3	3.7	3.5	-	-	9.2	8.8	8.4
16	12,758	106	106	106	105	105	437	421	456	470	467	4.1	4.0	4.3	4.5	4.5	5.0	5.0	5.5	5.5	8.4
17	13,636	120	121	128	-	127	424	462	509	-	517	3.5	3.8	4.0	-	4.1	6.4	7.7	8.4	-	7.8
18	14,466	143	143	141	-	142	501	586	568	-	681	3.5	4.1	4.0	-	4.8	7.4	7.5	8.2	-	8.7
19	15,226	140	143	138	-	112	383	389	396	-	410	2.7	2.7	2.9	-	3.7	6.2	6.7	6.6	-	6.3
19.1	15,496	-	-	111	-	110	-	-	422	-	430	-	-	3.8	-	3.9	-	-	6.5	-	7.7
19.2	15,575	-	-	130	-	125	-	-	734	-	714	-	-	5.6	-	5.7	-	-	12.2	-	9.5
19.4	15,717	-	-	200	-	202	-	-	669	-	666	-	-	3.4	-	3.3	-	-	6.3	-	5.2
20	16,195	121	122	146	-	129	369	459	475	-	476	3.1	3.8	3.2	-	3.7	5.0	5.7	6.3	-	5.2
21	16,938	124	126	122	-	120	354	426	413	-	458	2.9	3.4	3.4	-	3.8	3.8	4.7	5.2	-	5.0
22	17,655	104	109	102	-	150	356	478	410	-	545	3.4	4.4	4.0	-	3.6	5.0	5.2	5.4	-	5.8
23	18,490	112	111	105	-	-	308	354	355	-	-	2.8	3.2	3.4	-	-	4.1	5.1	5.0	-	-
24	19,353	111	112	109	-	106	336	388	404	-	406	3.0	3.5	3.7	-	3.8	5.4	5.3	5.8	-	5.6
25	19,869	97	97	95	-	93	380	428	424	-	418	3.9	4.4	4.5	-	4.5	5.6	6.1	6.5	-	7.2
26	20,207	77	89	87	-	88	271	375	354	-	357	3.5	4.2	4.1	-	4.0	5.6	6.4	6.1	-	5.9
26.1	20,491	-	-	71	-	68	-	-	287	-	300	-	-	4.0	-	4.4	-	-	5.7	-	5.4
27	20,983	69	71	68	-	75	238	283	304	-	330	3.5	4.0	4.5	-	4.4	5.0	5.3	5.6	-	5.1
28	21,819	49	53	65	-	78	193	241	274	-	300	3.9	4.6	4.2	-	3.8	4.7	6.2	5.6	-	5.5
28.1	22,223	-	-	62	-	75			278	-	308			4.5	-	4.1			5.5	-	5.3
29	22,468	59	67	65	-	75	244	286	292	-	295	4.1	4.3	4.5	-	3.9	5.2	5.9	5.8	-	5.6
30	23,182	54	60	60	-	118	238	266	290	-	351	4.4	4.4	4.8	-	3.0	5.8	6.0	5.8	-	5.7
31	23,995	53	57	65	-	81	237	289	303	-	351	4.5	5.1	4.7	-	4.3	5.5	6.2	6.5	-	6.0
32	24,754	96	114	106	-	118	349	545	524	-	516	3.7	4.8	5.0	-	4.4	7.8	6.8	7.0	-	6.7
33	25,555	125	127	126	-	-	537	558	558	-	-	4.3	4.4	4.4	-	-	7.2	8.9	8.9	-	-
34	26,281	94	94	94	-	94	446	490	453	-	457	4.8	5.2	4.8	-	4.9	6.4	7.1	7.1	-	7.4
35	27,133	102	102	105	-	102	350	399	396	-	426	3.4	3.9	3.8	-	4.2	5.5	6.9	7.5	-	6.9
36	27,909	104	103	109	-	105	352	394	387	-	403	3.4	3.8	3.5	-	3.8	6.6	6.9	6.8	-	6.9
36.1	28,093	-	-	68	-	69	-	-	326	-	341	-	-	4.8	-	4.9	-	-	6.1	-	6.3
37	28,753	81	83	96	-	93	157	194	439	-	436	1.9	2.3	4.6	-	4.7	3.3	3.6	6.5	-	6.6
38	29,651	68	68	72	-	-	247	282	302	-	-	3.6	4.1	4.2	-	-	5.3	5.8	5.2	-	-
39	30,531	76	76	76	-	-	242	292	305	-	-	3.2	3.8	4.0	-	-	4.3	5.5	4.8	-	-
40	31,291	72	71	68	-	-	221	255	262	-	-	3.1	3.6	3.9	-	-	5.1	6.1	5.8	-	-
41	31,891	92	90	91	-	-	407	465	483	-	-	4.4	5.2	5.3	-	-	5.9	6.7	7.0	-	-
42	32,812	87	91	97	-	102	359	388	453	-	498	4.1	4.3	4.7	-	4.9	7.0	7.6	6.7	-	7.1
42.1	33,191	-	-	135	-	133	-	-	682	-	745	-	-	5.1	-	5.6	-	-	7.3	-	8.1

Cross-section	Chainage	WIDTH (m)					AREA (m^2)					AVERAGE DEPTH (m)					MAXIMUM DEPTH (m)				
	(m)	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000	1969	1978	1985	1993	2000
42.2	33,462	-	-	288	-	289	-	-	764	-	695	-	-	2.6	-	2.4	-	-	5.0	-	6.3
43	33,815	191	234	?	-	-	372	617	?	-	-	1.9	2.6	?	-	-	4.6	4.7	?	-	-
44	35,080	77	84	92	-	-	340	396	452	-	-	4.4	4.7	4.9	-	-	6.2	6.8	6.4	-	-
45	35,660	81	80	88	-	-	385	433	425	-	-	4.8	5.4	4.8	-	-	6.0	7.5	72.0	-	-
46	36,400	211	229	272	-	-	754	826	822	-	-	3.6	3.6	3.0	-	-	5.8	5.7	6.0	-	-
47	37,030	?	120	154	-	-	?	574	675	-	-	?	4.8	4.4	-	-	?	6.2	6.6	-	-
48	37,640	?	84	83	-	-	?	390	375	-	-	?	4.6	4.5	-	-	?	6.2	6.0	-	-
49	38,270	?	68	66	-	-	?	367	346	-	-	?	5.4	5.2	-	-	?	7.3	6.9	-	-
50	38,930	?	85	85	-	-	?	360	360	-	-	?	4.2	4.2	-	-	?	5.3	5.5	-	-
51	39,610	?	51	52	-	-	?	241	245	-	-	?	4.7	4.7	-	-	?	6.5	6.5	-	-
51.1	39,907	-	-	66	-	66	-	-	621	-	577	-	-	9.5	-	8.7	-	-	13.2	-	12.2
51.2	39,916	-	-	68	-	69	-	-	641	-	615	-	-	9.4	-	9.0	-	-	13.2	-	12.5
52	40,265	179	162	83	-	164	410	392	284	-	461	2.3	2.4	3.4	-	2.8	5.9	7.5	5.6	-	5.6
53	41,306	169	144	182	-	184	266	297	560	-	594	1.6	2.1	3.1	-	3.2	3.7	4.3	4.8	-	5.5
54	42,246	78	77	128	-	129	252	292	406	-	371	3.2	3.8	3.2	-	2.9	4.4	5.6	6.2	-	5.9
55	42,503	66	114	151	-	151	290	471	449	-	421	4.4	4.1	3.0	-	2.8	5.2	5.5	5.8	-	4.2
56	43,213	144	200	178	-	?	486	634	627	-	?	3	3	4	-	?	5.8	5.4	6.2	-	?

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Cross-Sectional Changes in Channel Width Along Lower Reaches of Lillooet River - 1969 to 2000

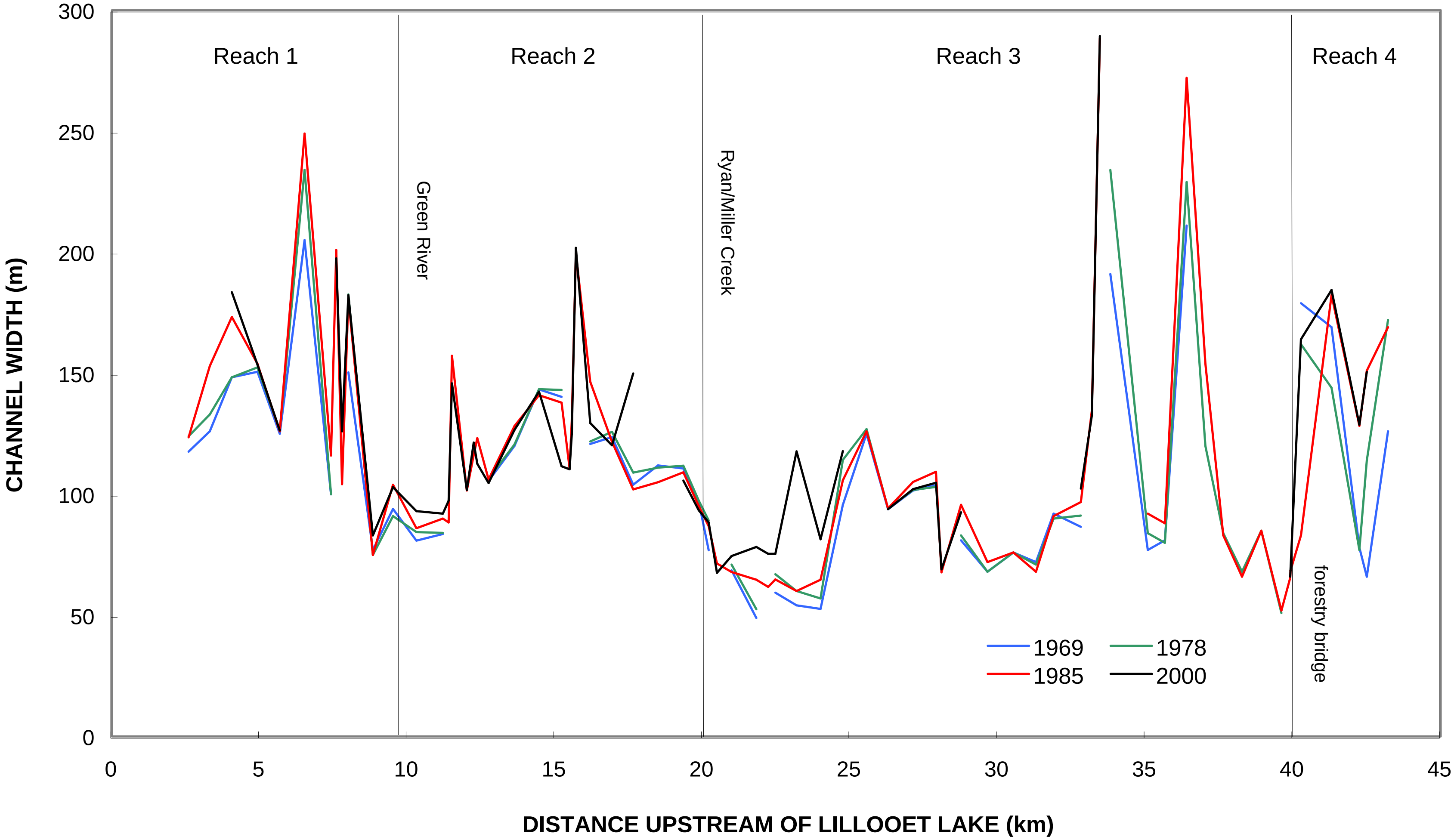

Figure 5-15

It is also possible that the some of the observed degradation between km 12 and 20 is a result of undocumented gravel removals. This is particularly true prior to 1980 when gravel removals were not systematically recorded.

CROSS SECTIONAL CHANGES

Morphologic changes along the lower reaches were quantitatively assessed for the past three decades by a comparison of cross section survey data from 1969, 1978, 1985, 1993 and 2000. These data, even though section data for some years are missing, constitute one of the most complete data sets in British Columbia. The variables measured for this analysis were width (w), maximum depth (d), and area (A). All variables were measured assuming bankfull conditions to allow meaningful comparisons between different survey years. Average depth (*d*) was calculated by dividing channel area by width.

When interpreting the results anthropogenic channel alterations have to be taken into account. This includes placement of riprap that can decrease channel width over the short-term, and gravel mining that usually increases channel depth. These channel alterations not only cause deviations from the natural channel development at the site they took place, but can influence channel development up and downstream.

Table 5-6 provides a summary of channel changes between 1969 and 2000 for Reaches 1 through 4. Blanks in the data set indicate that cross section measurements were not taken for that particular year.

Figure 5-15 shows changes in channel width along lower reaches for the four main surveys – 1969, 1978, 1985 and 2000. For the most part, there are few changes in channel width between surveys for a majority of the cross sections. A noticeable exception is the MacKenzie Cut upstream of km 20 where changes are variable ranging from minor decreases since 1985 to a 50 m increase at km 23.2 (XS 30). Apart from an old private agricultural dyke, the river is largely unconfined by artificial structures or bedrock through this section, and is flowing through a former peat bog.

However, the observed changes in channel width from the cross sectional data yield a false impression of relative stability. Because the cross sections are spaced relatively far apart (approximately 800 m), significant bank erosion can occur between cross sections and not be accounted for. A better measure of changes in channel width is to superimpose banklines for various dates. This exercise was completed for the upper braided section and km 27 to km 40 of Reach 3 in the previous section (Figures 5-6 to 5-9). Subsection 5.4 provides a more detailed account of bank erosion rates for lower reaches since 1970.

A better measure of channel changes through time is a comparison of channel area, as shown on Figure 5-16 and Table 5-6. Since 1978, there have been relatively minor changes in channel area for both Reaches 2 and 3 (although there are significant sections upstream of the confluence with Ryan River and Miller Creek where 2000 surveys were

713-001\Drawings\Excel Figure\x-sections.xls

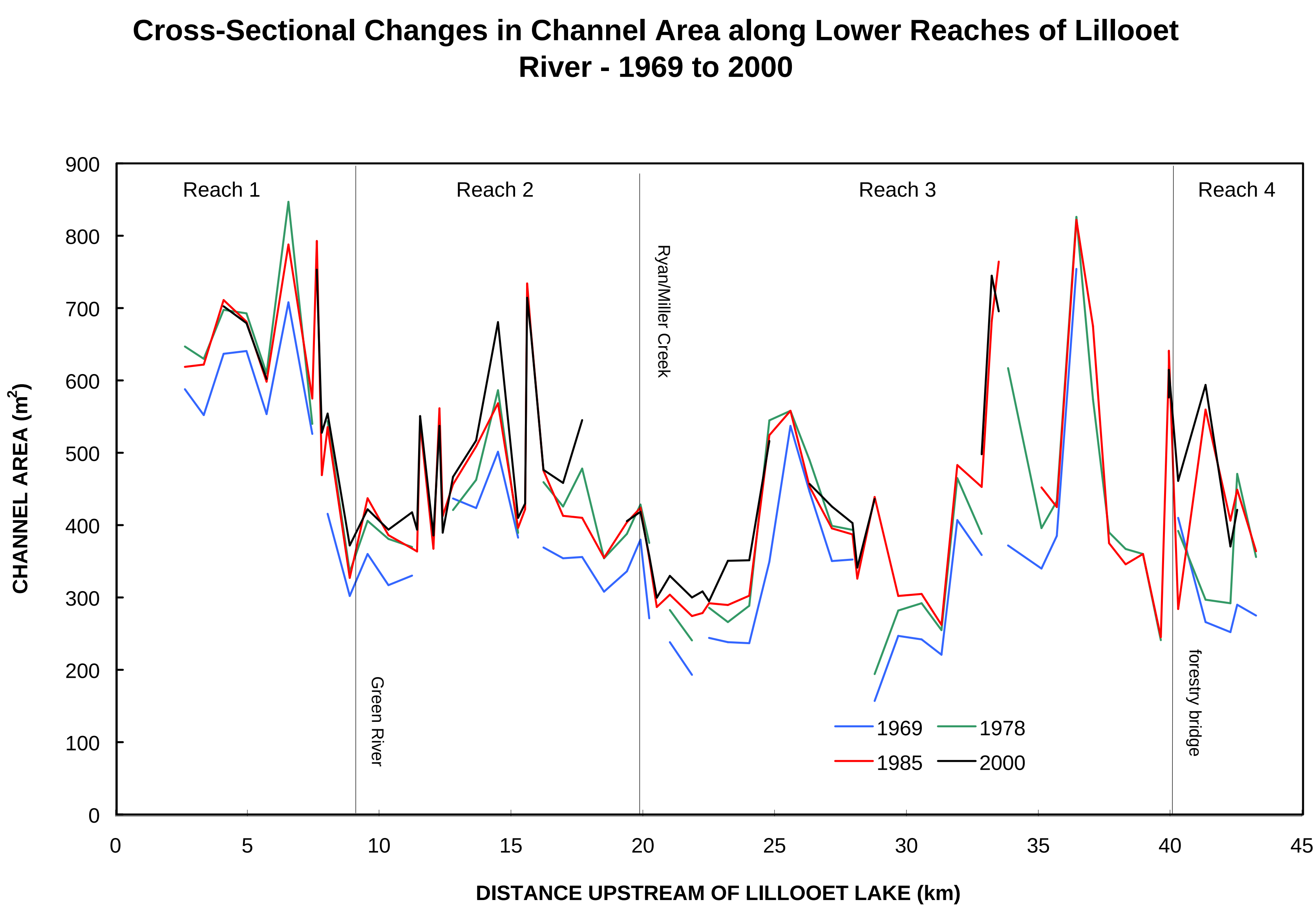

Figure 5-16

not completed at the monumented cross sections). Between 1969 and 1978, however, there were significant increases in channel area - particularly in Reach 2 in the vicinity of Pemberton. This may be the result of large, undocumented gravel removals during this period that exceeded input rates. As before, however, caution must be exercised in interpreting these results since the cross sections are on average about 800 m apart. The implication of these results is discussed in more detail in the following section.

SEDIMENT TRANSPORT

Sutek Services Ltd. and Kellerhals Engineering Services Ltd (1989) have previously estimated gravel transport rates in Lillooet River (herein referenced as Kellerhals, 1989). As noted in this section, their report addresses gravel supply and removal in fisheries streams, with Lillooet River being one of the case studies. Several methods were used to estimate gravel transport rates for Lillooet River and are described in more detail below.

1. *Suspended Sediment Approach*

   It is commonly observed that bedload is a small proportion of the suspended load, with the proportion dependant on basin size and the physiographic setting of the basin. For British Columbia, Kellerhals stated that gravel load as a percentage of suspended load in glacierized basins is typically 5 to 12%. For Lillooet River, Kellerhals suggested that the bedload could amount to as much as 8% of the suspended load, providing an estimated annual load of 48,000 m^3.

2. *Bedload Yield Estimates*

   Another approach is through comparison with bedload measurements collected in other streams. Based on a review of available data (which the author stated were probably too few), Kellerhals determined average bedload yield for different gravel river types. While local gravel sources such as outwash terraces or erodible banks will cause significant differences between basins in similar physiographic regions, this method is useful as a first approximation. Based on gravel yield rates of 10 to 20 $m^3/km^2/yr$ for braided rivers, bedload was estimated for Lillooet River between 22,000 and 44,000 m^3/yr.

3. *Channel Profile Shifting*

   If average progradation rates for a delta are known and an accurate river profile is available, the material deposited in a gravel accumulation zone may be estimated by shifting the profile by a distance equivalent to the delta progradation over a long period and measuring the accumulation as the area between the two profiles within the gravel accumulation zone. The volume of accumulation requires an estimate of the channel width. For Lillooet River, Kellerhals constructed a profile by using a 1985 river survey completed by the Water Management Branch and contour crossings taken from NTS 1:50,000 maps. The delta progradation from 1858 to 1969 of 1,140 m (see Section 5.5) was used to shift the profile and approximate the river profile as of 1858. An annual gravel supply of 23,000 m^3/yr was calculated from the cross sectional area between the two profiles, an estimated channel width of 100 m, and the 110 year period between the two profiles.

4. *Lateral Erosion*

Kellerhals also used an approach similar to that used in this report for the braided reach (Section 5.2). The basis of the approach is the work of Neill (1971). Neill (1971) postulated that a lower bound estimate of bed load could be determined by assuming that all of the material eroded out of the floodplain at the outside of one bend is deposited on the inside (point bar) of the next downstream bend. Kellerhals used an extension of this approach for application to the braided reach of Lillooet River. That is, an estimate of gravel transport can be determined by using the average volume eroded or deposited per year in a given reach and the average distance between bank erosion sites and the next downstream deposition site. Kellerhals created a map of eroded and deposited areas between km 46 and km 52 using air photographs taken on July 15, 1979 and August 4, 1986. Erosional volumes were calculated from the erosional areas by assuming a 3 m depth of gravel in the eroded gravel bars and a 4 m depth in the eroding floodplain. For the 5 km reach, the spacing between major deposition zones appeared to be about 1 km and Kellerhals estimated the gravel load at 30,000 m^3/yr.

5. *Replicate Surveys*

The final approach used by Kellerhals is a method that will be expanded upon later in this section. In brief, repeated cross section surveys can be used to determine changes in channel area between survey dates. Volumes are then calculated by extrapolating between sections. Using this method for cross sections situated between km 14 and 44, Kellerhals estimated gravel transport at 40,000 m^3/yr.

Kellerhals concluded that the contemporary annual gravel transport rate in Lillooet River was likely between 30,000 and 40,000 m^3/yr.

While the results of Kellerhals (1989) are extremely useful for estimating gravel transport rates, further analysis of the cross sectional records is required, as Kellerhals' investigation was cursory. The use of cross sectional changes to estimate gravel transport rates has been successfully applied to a number of rivers including Vedder River in British Columbia (Martin and Church, 1995). Successive surveys are overlaid to obtain the net change in cross section area. Net volumetric changes are then calculated between sections on the assumption that the change in area at a cross section is representative of the distance between it and the half-distance to each adjacent cross section. Once the volumetric change between survey dates is determined, and if the bed-material transport rate is known at one place along the channel, calculations can be extended upstream or downstream using the sediment budget approach.

This approach appears to be well-suited for Lillooet River given the repeated cross sections and point of zero gravel transport near the Green River confluence. The analysis of cross sectional changes for Lillooet River extends from km 11 (XS 14) to km 44 (XS56), with km 11 being near the distal end of gravel deposition. Unfortunately, the cross sectional areas listed in Table 5-5 are unsuitable for calculating areal changes between survey dates. The primary interest is the gravel area of the cross section only and as such, the area contributed by overbank deposits of fine sediment is not considered. In general, the overbank deposits are 2.5 to 3.5 m thick in the irregularly meandering reach of the river.

However, even if the tabulated areas considered gravel only, there is no obvious trend toward reduced channel area at the cross sections. Because significant amounts of gravel are not being transported past the confluence with Green River, gravel transported past the forestry bridge should be accumulating along the channel bed in Reaches 2 and 3. As such, a majority of cross sections should have reduced cross sectional areas over time. While such a trend is not apparent, there are several factors that explain the lack of observed aggradation:

1. An analysis of morphologic changes for the upper reaches indicated that approximately 40,000 m^3/yr of sediment is being transported past the forestry bridge into Reaches 2 and 3. Given that the channel is on average 110 m wide and the lower gravel reach has an approximate length of 30 km, ten years of sediment transport represents approximately 12 cm of aggradation. Although it is unrealistic to expect that the aggradation would be evenly distributed along the channel, this calculation does illustrate that sediment input rates are modest and therefore relatively difficult to identify with repeated surveys.

2. Sediment in irregularly wandering gravel-bed rivers tends to accumulate in sedimentation zones that are separated by long stable reaches. This is the case with Lillooet River where long stretches exhibit few changes due to extensive bank protection or semi-confinement by natural topographic features (see Sheets 3 to 11, Appendix B). Sediment tends to be transported through these stable reaches and accumulate where the channel is laterally unconfined.

   As previously noted, the cross sections are spaced approximately 800 m apart. Given that the average channel width is only 110 m, it is unrealistic to expect that the cross sections would intersect all sedimentation zones. Sedimentation zones along lower reaches of the river are presently located up and downstream of the confluence with Pemberton Creek (at approximately km 11), immediately upstream of the BCR bridge (km 16, XS 19.4), upstream of XS 22 (km 18), upstream of XS 32 (km 25), between XS 34 and 35 (km 26.5), between XS 42 and 44 (km 34), and between XS 46 and 48 (km 37). In almost all of these cases, cross sections do not intersect these sedimentation zones or are not representative of the aggradation. As a result, the cross sectional data are poorly suited for sediment transport estimates.

3. A final complicating factor in analyzing channel changes for the lower reaches is gravel removal. Table 5-7 is a list of known gravel removal activities along Lillooet River and its tributaries. The totals presented in the table represent minimum volumes as gravel removals have not all been well documented.

Table 5-7
Documented Gravel Removals on Lillooet River and Tributaries

Creek	Date	Removal (m^3)	Notes
Ryan	1980 – 1987	98,000	79,000 removed from upper river and 21,000 near the highway crossing
Miller	1980 – 1987	108,000	upstream of the highway bridge
	August 1998	2,500	
	March 1999	5,255	
	October 2000	2,680	
	March 2001	450	
Pemberton	1980 – 1987	27,500	near the highway crossing
	1991	500	used by MELP
	1998	800	
	2000	900	
Lillooet			
km 48	1980 – 1987	20,000	
km 41	1980 – 1987	31,000	
km 18 – 21	1980 – 1987	134,000	
km 18	1992 - 1993	30,000	gravel removed by Rush Contracting (contact Joe Miller) for construction of Pemberton high school
km 16	1980 – 1987	10,000	
km 14	1980 – 1987	9,000	
N.B. Sources are "Assessing Gravel Supply and removal in fisheries streams – Sutek Services Ltd. and Kellerhals Engineering Services Ltd., March 1989" for the period 1980 to 1987 and PVDD for removals since 1990 (except where noted).			

An assessment of channel changes should incorporate these gravel removals as this activity can give the appearance of degradation or no change at a given cross section. For the 1980-1987 period, gravel removals from Lillooet River averaged about 30,000 m^3/yr. Because this value represents a lower bound estimate of the amount of gravel entering lower reaches, it is not surprising that cross sectional changes are poorly suited for estimating gravel transport rates.

While the total amount of gravel removed from Lillooet River over the last thirty years is not well documented, this is particularly so prior to 1980. Large volumes may have been removed from the river (in the vicinity of Pemberton) in this period resulting in the observed pattern of increased channel area between 1969 and 1978. Further study of historical records is needed to determine whether there is a link between gravel removals and the observed changes in cross sectional area.

Also of interest are the gravel volumes that have been removed from Ryan, Miller and Pemberton Creeks. These tributaries are potential sources of gravel for the mainstem of Lillooet River but it appears that these creeks are being managed such that gravel does not accumulate along their lower reaches. As a result, gravel inputs from these tributaries are probably not significant and do not have to be accounted for in an analysis of cross sectional changes.

CONCLUSIONS

Several conclusions can be made regarding sediment transport in the lower reaches of Lillooet River:

1. The contemporary annual gravel transport rate in Lillooet River is approximately 40,000 m^3/yr.

2. The current spacing of the cross sections (approximately 800 m) is inadequate to quantify aggradation between surveys. Gravel deposition below the forestry bridge tends to occur in well defined sedimentation zones. These zones are separated by long stable reaches (generally riprapped) that exhibit few channel changes and act as effective conduits for downstream gravel transport. In many cases, the existing monumented cross sections do not intersect these sedimentation zones.

3. Sedimentation zones are currently located up and downstream of the confluence with Pemberton Creek (approximately 11 km), immediately upstream of the BCR bridge (16 km, XS 19.4), upstream of XS 22 (18 km), upstream of XS 32 (km 25), between XS 34 and 35 (26.5 km), between XS 42 and 44 (34 km), and between XS 46 and 48 (37 km).

4. Because sedimentation tends to be localized, the potential for reduced channel conveyance during flooding is also localized. The implication is that flood management can concentrate on several points along the river rather than along its entire length. However, difficulties in quantifying the amount of aggradation (and hence the flood risk) will be encountered due to inadequate spacing of the existing cross sections.

5. Bedload transport rates below the forestry bridge are modest (approximately 40,000 m^3/yr), which represents an overall bed level increase of 0.12 m over a ten year period. As such, there is not a concern of significant aggradation along the channel bed that would require immediate attention. Aggradation should occur over a number of years allowing sufficient time for appropriate flood measures to be implemented.

5.4 BANK EROSION

Bank erosion is of fundamental importance for management of land adjacent to Lillooet River and its tributaries. Unless protected with riprap, the river banks usually consist of weak and easily erodible floodplain deposits. Two different data sets were used to analyze bank erosion. The figures in Appendix B, which show the location of river cross sections and engineering works, also show banklines taken from 1971 and 1986 floodplain maps superimposed on a 1999 orthophoto of the lower reach. These overlay maps were used to quantify changes in stream alignment since 1971 and predict potential problem sites for future bank erosion. In addition, repeated cross section surveys allowed comparison of channel geometry adjustments over time. The following description of bank erosion begins at cross section XS 1 and follows the river upstream. Changes in the lake delta are discussed separately in Section 5.5.

LILLOOET RIVER

In the period 1971 to 1979, the river made a significant move to the north between XS 1 and XS 2 (Photo 7, Appendix C). Up to 125 m of Mount Currie Band land was eroded along an 800 m reach of channel in this area, corresponding to an average erosion rate of 16 m/yr. Without riprap placement in 1979, with funding provided by ARDSA, bank erosion would likely have continued. Erosion appears to have been initiated by deposition of sediment on the right bank (Sheet 1, Appendix B).

From XS 3 to XS 4, the river has shifted further to the north by 5 to 20 m over a 600 m length of channel. Most of this erosion appears to have occurred between 1971 and 1981, ending when riprap was placed along the north bank. Bank erosion has also occurred on the south bank at XS 4 where there has been up to 100 m of erosion since 1971 over a distance of about 300 m. This erosion can be attributed to two geomorphic features. First, a large bar developed immediately upstream of XS 4 on the left bank. This would have resulted in the river shifting to the south and eroding the right bank. A notable feature at this location, however, is a debris flow fan from a small unnamed tributary. Material deposited onto the fan is relatively competent relative to the fine-grained floodplain deposits. Therefore, the fan restricted further erosion of the south bank and shifted the attack of the river to the north side.

In the vicinity of XS 5, there was up to 15 m of erosion on the left bank between 1978 and 1983. Erosion occurred over a distance of approximately 500 m, with riprap placed in a 180 m section upstream of the cross section in 1983. Little change has occurred between XS 5 and XS 7 as the river follows a straight alignment, although 400 m of riprap was placed downstream of XS 7 on the left bank in 1983.

Downstream of XS 8, 25 to 40 m of erosion occurred on the left bank between 1971 and 1978 for an approximate distance of 300 m. The emplacement of 550 m of riprap in 1980 stopped further erosion. Sedimentation and island formation on the right bank was responsible for directing flows towards the left-hand side. Erosion of the left bank also occurred downstream of XS 9 for 300 m – up to 15 m between 1978 and 1980 – before riprapping of the bank in 1980.

At XS 10, localized erosion (15 to 30 m over a distance of 150 m) of the left bank occurred between 1971 and 1978. The placement of bank protection over a distance of 515 m in 1980 stopped further erosion but a potential future trend is for the mainstem channel to occupy a small side channel on the left-hand side immediately downstream of XS 10 (Sheet 3, Appendix B). At this location, the mainstem channel of Lillooet River meanders around a large island and confluences with Green River (Photo 8, Appendix C). Given the curvature of the bend, a potential scenario is for the channel to straighten over time.

Between XS 10 and XS 16, there have been some minor placement and repairs of bank protection but few changes in channel geometry since 1971 (Photo 8, Appendix C). 10 to 20 m of erosion did occur on the left bank between XS 12 and 13 (about 700 m distance) in the 1971 to 1978 period, but the floodplain is well vegetated and not currently developed at this location. The overall stability of this reach can probably be attributed to extensive bank protection, particularly on the right dyked bank. If future aggradation were to occur in this reach (for example a gravel bar is currently aggrading on the right bank between XS 13 and XS 14), bank erosion would be more likely.

At XS 17 to XS 18, the river meanders around a relatively tight bend as shown in Appendix B (Sheet 5). Prior to 1970 almost 700 m of the left bank was protected with riprap due to ongoing bank erosion and the proximity of the BC Rail line. The riprap was upgraded in 1977. Since then there have been localized erosion problems. At XS 17, 15 m of erosion occurred between 1978 and 1984, prompting riprap repair over a 100 m section in 1984. XS 18 appears to have been the upstream extent of the original riprap and up to 35 m of erosion occurred above this point between 1969 and 1978 over a distance of about 200 m. A further 15 m of erosion prompted BC Rail to construct 200 m of bank protection in 1984 to protect the railway.

Upstream of XS 18, there have been few changes in channel alignment up to the BC Rail bridge. However, 1,055 m of riprap was placed along the right bank in 1980 to discourage bank erosion. Through this section the river takes a sharp bend and is a potential problem site for future erosion (Photo 10, Appendix C). The most problematic area is in the vicinity of XS 19.4 where 30 to 50 m of erosion (1971 to 1985) occurred on the left bank over a 150 m length of channel. The erosion resulted in channel widening and as noted on Sheet 5, a large gravel bar has accumulated immediately upstream of the bridge (Photo 11, Appendix C). Deposition at this location could result in a tendency for the channel to straighten itself (and thereby attack the left bridge abutment) or force more of the flow toward the right bank and right abutment. Channel stability will likely be an ongoing problem at this location, particularly during the next large flood.

Between XS 20 and XS 22 there have been few changes in channel alignment since 1971, which is probably due to existing riprap (1,000 m of bank protection in 1980 on the right bank) and a relatively straight channel alignment. Minor riprap repairs were completed for short sections in the 1980s.

Upstream of XS 22 for about 200 m, 30 to 40 m of bank erosion has occurred on the left bank since 1985. As noted on Sheet 6, the erosion is in response to lateral bar growth on the right bank. The left bank is a potential site of future erosion. Continued erosion of the left bank in a downstream direction is possible. Further upstream, up to the Miller Creek confluence (XS 25 – Photo 12, Appendix C), there have been few changes in alignment since the early 1970s. Again, extensive riprapping (over 1,700 m) of the right bank in the early 1980's has discouraged any bank erosion. Riprap placed on the right bank between Miller Creek and Ryan River prior to 1970 is oversteepened and required some repair in 1983 (Photo 13, Appendix C).

From XS 26 to XS 32, Lillooet River flows along the MacKenzie cut – the longest meander cutoff constructed in the late 1940's (Figures 3-1 and 3-2). Since the early 1970's, there has been a tendency for this stretch of river to widen along both banks (Photo 14, Appendix C). Along the right bank there has been 5 to 20 m of erosion, with slightly lower values along the left bank (0 to 10 m). To discourage further erosion between XS 31 and XS 32, approximately 1 km of bank protection was constructed along the right bank in the early 1980s. This section of the river flows through peat deposits that are easily eroded.

Upstream of XS 32, 30 to 40 m of the left bank was eroded between 1969 and 1978. Approximately 300 m of bank was impacted and was probably initiated by gravel bar deposition on the right bank (Sheet 7, Appendix B). Further upstream 1,300 m of riprap on the right bank and rocky ground on the left bank have effectively held the river in its present alignment.

Gravel bar deposition on the right bank has also resulted in left bank erosion between XS 34 and XS 35. Approximately 60 m of bank has eroded over a distance of 300 m, with much of the erosion occurring since 1985 (Sheet 7 and 8, Appendix B). Gravel deposition in the eroded bank area is visible on the 1999 orthophoto, perhaps indicating that the river will start to erode the gravel bar along the right bank and threaten the existing bank protection and threaten the existing riprap (Photo 15, Appendix C). Upstream of this point to XS 42, changes in channel alignment have been minimal due to high, rocky ground along the left bank and riprapped sections along the right bank.

Channel changes between XS 42 and XS 44 have been extreme over the past three decades and are coincident with a zone of active sedimentation (Sheet 9, Appendix B). In the vicinity of XS 42.2, 50 to 150 m of the right bank eroded between 1971 and 1985 over an approximate distance of 300 m. A compensating zone of deposition established on the opposite bank in the same time frame. At XS 43, 100 m of the left bank has eroded since 1971 but an equal area of deposition has built up on the opposite bank. A little further upstream 100 to 150 m of the left bank has eroded for a distance approaching 700 m. A majority of this erosion occurred after 1985. Compensating deposition has occurred along the opposite bank in the form of a large point bar. Portions of the point bar are in the initial stages of vegetative development. The observed instability is the result of gravel accumulation in the mainstem channel, which has resulted in flow diversions and large-scale channel changes. This sedimentation zone is likely to work its way downstream of XS 42, causing additional lateral instability.

Between XS 44 and XS 45, 5 to 10 m of erosion occurred on the left bank between 1978 and 1985. However, construction of 560 m of bank protection in 1981 has ensured that the right bank position has remained fixed. Further upstream, the 1971 to 1985 period saw erosion of the right bank between XS 45 and 46 (50 to 100 m over 300 m distance) and at XS 47 (75 m over 250 m of channel). Bank erosion between the former cross sections was curtailed by riprap extension and repair in the early 1980s (Sheet 10, Appendix B). Some of this riprap is currently oversteepened and sloughing (Photo 16, Appendix C). Upstream of XS 47 towards the forest bridge (XS 51.1), there have been few changes in channel alignment since the early 1970s.

In the vicinity of the forestry bridge, riprap was placed on the right bank in the early 1980s to prevent erosion (Photos 17 and 18). Upstream of the bridge, the riprap is ravelling and appears to have been dumped rather than placed during construction.

SUMMARY

Table 5-8 provides a summary of observed bank erosion for the 1971 to 2000 period. The time frame of the observed erosion is approximate as it is constrained by the dates of the mapped bankline positions and cross section surveys.

Table 5-8
Summary of Lateral Channel Changes of Lillooet River (XS 1 to XS 51.1) – 1971 to 2000

Location	Description	Time Frame	Channel Length (m)	Erosion (m)	Erosion (m/yr)
XS1 – XS2	Erosion of left bank with bank protection constructed in 1971.	1971 - 1979	800	125	16
XS3 – XS4	Erosion of left bank until riprap placement in 1981.	1971 - 1981	600	5 - 20	0.1 – 2
XS4	Erosion of right bank immediately downstream of debris flow fan.	1971 - 2000	300	100	3
XS5	Erosion of left bank with some bank protection in 1983.	1978 - 1983	500	15	3
XS5 – XS7	No significant changes.				
XS8	Left bank erosion d/s of XS 8 due to island and bar development along right bank.	1971 - 1978	300	25 - 40	3.5 – 5.5
XS9	Left bank erosion d/s of XS 9 before riprapping of bank in 1980.	1978 - 1980	300	15	8

Location	Description	Time Frame	Channel Length (m)	Erosion (m)	Erosion (m/yr)
XS10	Localized erosion of left bank until riprap placement in 1980.	1971 - 1978	150	15 - 30	2 – 4
XS12 – XS13	Erosion of undeveloped left bank.	1971 - 1978	700	10 - 20	1.5 – 3
XS13 – XS16	No significant changes.				
XS17	Left bank erosion of existing riprap – repaired in 1984.	1978 - 1984	150	15	2.5
XS18	Erosion of left bank upstream of existing riprap. Additional riprap placed by BC Rail to protect line.	1969 - 1978	200	35	4
XS19.4	Left bank erosion immediately u/s of BC Rail bridge.	1971 - 1985	150	30 - 50	2 – 3.5
XS20 – XS22	No significant changes but 1000 m of bank protection on right bank.				
XS22	Left bank erosion u/s of XS22 in response to lateral bar growth on opposite bank.	1985 - 2000	200	30 - 40	2 – 3
XS22 – XS25	Few changes in alignment – in part due to 1700 m of right bank protection built in early 1980s.				
XS26 – XS32	Length of MacKenzie cut which has widened along both banks since 1970.	1970 - 2000	4500	5 -20	0.2 – 0.7
XS32	Left bank erosion u/s of XS32.	1969 - 1978	300	30 - 40	3 – 4.5
XS34 – XS35	Gravel deposition on right bank resulted in left bank erosion.	1985 - 2000	300	60	4
XS35 – XS42	Minimal changes in alignment due to right bank riprap and hard high ground on left side.				
XS42.2	Right bank erosion with equal deposition on opposite bank.	1971 - 1985	300	50 - 150	3.5 – 11
XS43	Left bank erosion with compensating right bank deposition.	1971 - 2000	300	100	3
XS43 – XS44	Left bank erosion and right bank deposition.	1985 - 2000	700	100 - 150	6.5 – 10
XS44 – XS45	Minor left bank erosion – right bank is riprapped through this section.	1978 - 1985	550	5 - 10	0.5 – 1.5
XS45 – XS46	Right bank erosion.	1971 – 1985	300	50 - 100	3.5 – 7
XS47	Right bank erosion.	1971 – 1985	250	75	5
XS47– XS51.1	No significant alignment changes.				

A review of the summary table emphasizes two items of note. First, most of the observed erosion since 1970 occurred in the 1970s and early 1980s. Significant riprap placement and repair was conducted in the early 1980s, which probably explains the reduced observations of bank erosion after 1985. Where bank erosion has occurred since 1985, it has generally been confined to sections without riprap. Second, there appears to be a higher frequency of significant left bank (looking downstream) erosion. This probably reflects a greater emphasis on right bank protection since a majority of landowners in the valley are situated on the much broader right floodplain.

Although the incidence of bank erosion has reduced in the 1990s, several problem areas have been identified. These include (but are not restricted to) gravel deposition upstream of the BC Railway bridge (XS 19.4), the banks in the vicinity of XS 34, and sections downstream of XS 42 due to an upstream sedimentation zone. More detailed surveys would be required at these locations to further delineate the erosion hazard.

GREEN RIVER

Unlike Lillooet River, cross section survey data for Green River is restricted to 1985 and 2000 only. However, bankline positions from 1971 are mapped on the figures in Appendix B. As noted in the Section 5.2, the alignment of Green River was radically altered after the flood engineering works in the late 1940s. This section addresses the last three decades only.

Green River joins Lillooet River upstream of XS 9.1 and little change has taken place at the confluence since 1971. Between XS 2 and XS 3 (about 300 m distance), Green River has eroded its left bank by 50 to 75 m with much of the erosion occurring since 1985. This erosion could cause a threat to the Pemberton airport runway if it continues at its present rate. Erosion of the left bank has been accompanied by deposition on the inside of the meander (Sheet 3, Appendix B). A trend towards a tighter meander bend is emphasized by surveys of XS 4, where there has been approximately 10 m of right bank erosion since 1985.

The reach between XS 4 and XS 6 has changed dramatically over the past three decades. The area that is now covered by part of the Pemberton Valley golf course was part of an island of Green River in 1971 (Sheet 3, Appendix B). The side channel that separated the island from the left bank was cut off by 1985. It is not known whether infilling of the back channel was due to natural channel processes (i.e. gravel build-up at the back channel inflow) or a result of anthropogenic modifications.

Between XS 6 and XS 9 (Green River bridge), the planform of Green River has also undergone fundamental change. A side channel of Lillooet River used to flow southward and into Green River some 50 m downstream of Pemberton Creek. As shown by the banklines of 1971 on Sheet 4, flow in the side channel was split into three channels with the main flow meeting Green River at XS 7. The most easterly of these side channels formed a large island, which is now part of the Pemberton Valley golf course. After the

October 1984 flood the side channel was apparently cut off as a flood control measure and as a result, there has been considerable simplifying of the channel.

Further simplification of the channel has occurred further upstream between XS 12 and XS 15. In 1971, a side channel used to flow around an island on the left bank. The side channel was cut off by 1985, although the cause of channel realignment is not known. A portion of the Big Sky golf course now occupies part or the former side channel (Photo 19, Appendix C). Following abandonment of the side channel, considerable bank erosion (50 to 100 m over a 300 m distance) has occurred downstream of XS 13 since 1985 (Sheet 4). The bank erosion appears to be related to downstream meander migration, a natural tendency for this low gradient, gravel-bedded stream.

Major changes have also taken place upstream of XS 16 where the river has widened and become more sinuous (Sheet 4). For the most part, Green River flows against the colluvial and bedrock slopes of the bluffs separating it from Pemberton Creek. A back channel re-enters Green River immediately upstream of XS 16. Unlike the mainstem, the back channel has decreased in width and gravel bars visible in 1971 are now largely vegetated.

Increased channel width and sinuosity in the mainstem appears to be related to sedimentation in the area below the Nairn Falls rapids. Here, the river has changed drastically from single-thread to braided indicating a large influx of sediment to the reach. Downstream transfer of this sediment is a likely source of the observed channel changes in the study area. Over the next decade, this sedimentation could result in major changes in channel geometry, including reoccupation of its former channel to the northwest of Big Sky golf course (Sheet 4, Appendix B).

BIRKENHEAD RIVER

The Lower reaches of Birkenhead River have witnessed significantly less changes than either Lillooet River or Green River since 1971 (Photo 20, Appendix C). The only change to be observed between XS 0.3 (Highway 99 bridge) and XS 5.3 (bridge to Xit'lolaw village) is plant growth on previously bare gravel bars. There are no notable changes in width over this reach.

The same is true for the reach between XS 5.3 and XS 11, with few changes in channel alignment since 1971. Only between XS 7.1 and XS 8 has the river built out a meander, making it more sinuous. Further bank erosion was curtailed by the placement of bank protection (approximately 550 m) in the early 1980s.

More significant changes have occurred between XS 11 and XS 18 (Sheets 2 and 3, Appendix B). Several gravel bars that were unvegetated in 1971 are now covered with shrub vegetation. Furthermore, two back channels that exit the river at XS 12 had become significantly less active by 1999. The overall trend in this section is toward channel simplification, which may be related to a number of factors including upstream engineering efforts, a decrease in sediment supply or reduced flows.

RYAN RIVER AND MILLER CREEK

Few changes in channel alignment have occurred in the lower reaches of either Ryan River or Miller Creek in the past few decades (see Sheet 6, Appendix B). Miller Creek, in particular, has almost no room for lateral movement due to extensive riprapping and dyking of its banks.

5.5 DELTA FORMATION

Following the engineering works in the late 1940's, there was a rapid increase in the rate of delta advance in Lillooet Lake. Using old planimetric surveys, Gilbert (1973) determined that the average delta advance rate from 1858 to 1948 was 7 to 8 m/yr. For the 1948-1953 and 1953-1969 periods, the delta advanced rapidly with rates of 30 and 21 m/yr respectively. Based on a 1986 air photograph, Jordan and Slaymaker (1991) calculated the average rate of advance from 1969 to 1986 to be 14 m/yr.

Recent air photographs (August 3, 1999) have been used to update the mean annual advance of the delta. For the 1986-1999 period, the delta advanced at a rate of 16 m/yr (Photo 21, Appendix C).

Jordan and Slaymaker (1991) determined that approximately 2 x 10^6 m^3/yr of additional sediment, or about 40 x 10^6 m^3 over the 1948-1969 period was needed to explain the increased rate of delta advance. The authors concluded that the steadily declining rate of delta advance was consistent with a period of rapid downcutting and erosion following the lake level lowering and channel straightening, and a more gradual adjustment of the river in recent years. However, the authors also noted that an alternative source might be increased sediment yield during rapid glacier retreat from the 1920s through the 1960s, or a pulse of sediment from the 1931 Meager Creek debris flow. If these sediment inputs were delayed by several decades in the upper part of the valley, then they could have reached the delta after 1948. While a firm conclusion was not reached, Jordan and Slaymaker (1991) suggested that engineering works were largely responsible for increased sedimentation after 1948.

Table 5-9 and Figure 5-17 summarize delta changes since 1858. Included in the table are estimates of deposition volume between survey dates. Volumes are calculated by using the length of delta advance, an average delta width of 1,200 m and assuming that the delta geometry has remained constant over the time frame studied. Based on a 1987 survey by MELP, the delta front is situated approximately 100 m below the lake surface.

For a century prior to the major engineering works, an average of 900,000 m^3 of sand-sized and finer sediment was deposited onto the delta each year. Two to three times this annual volume was deposited in the following 21-year period – a total of approximately

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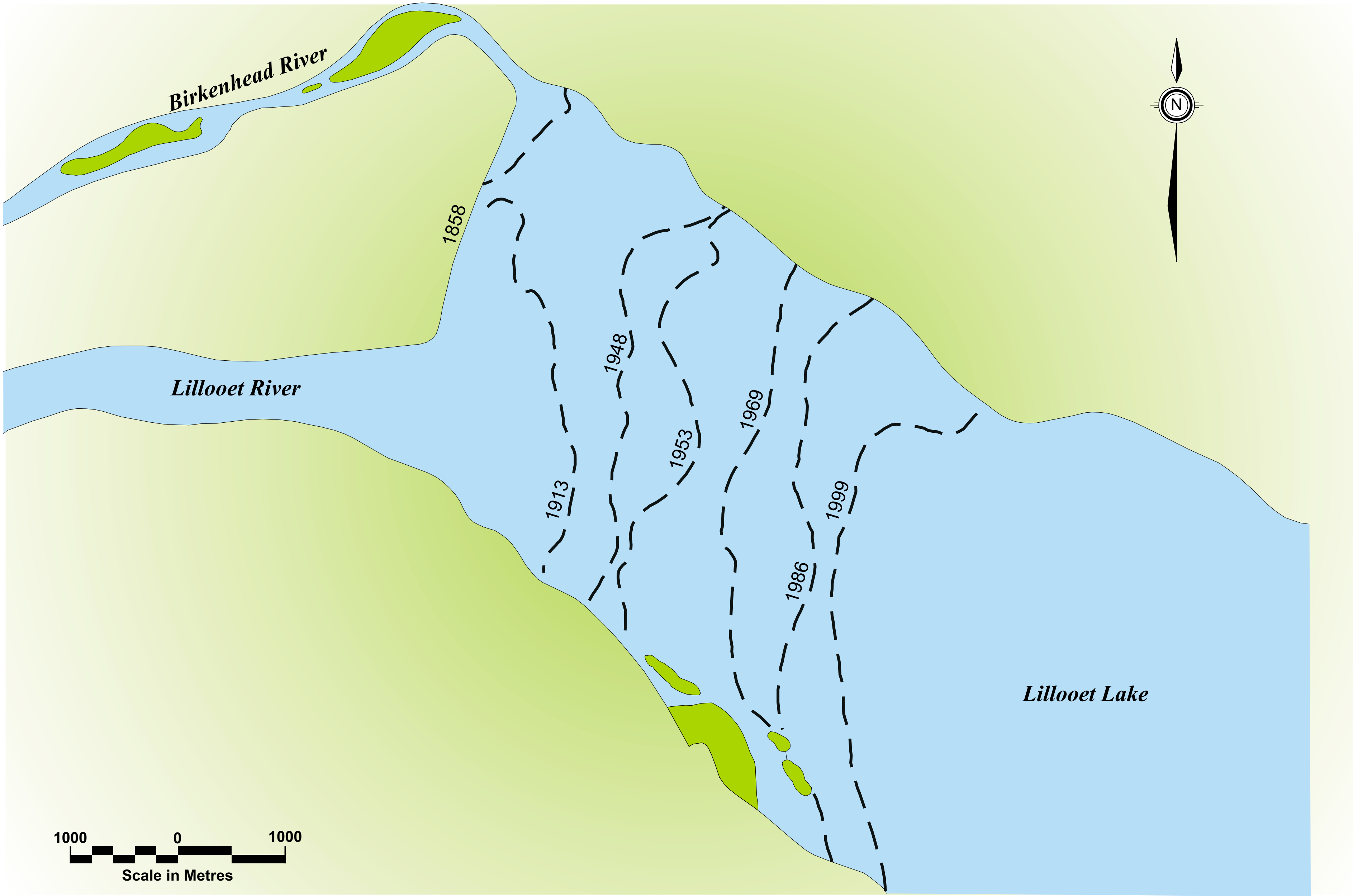

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Rate of Advance of Lillooet Delta Since 1858

Figure 5-17

57 x 10^6 m^3 of fine-grained sediment. Using the pre-1948 average volume of 0.9 x 10^6 m^3/yr, approximately 38 x 10^6 m^3 of additional sediment was deposited following the engineering works - a similar total to that calculated by Jordan and Slaymaker (1991).

Table 5-9
Changes in Lillooet Lake Delta Advance Rates

Period	**Natural Events and Engineering Works**	**Time** (yrs)	**Delta Advance** (m)	**Total Deposition** (x10^6 m^3)	**Average Deposition** (x10^6 m^3/yr)	**Mean Annual Advance** (m/yr)
1858-1913	No known events	55	374	44.9	0.8	7
1913-1948	Meander cutoffs, dyking, 1931 Devastation Creek debris flow, 1940 large flood	35	284	34.1	1.0	8
1948-1953	Dredging at Tenas Narrows, multiple debris flows and large landslides at Mount Meager	5	140	16.8	3.4	30
1953-1969	No known events	16	338	40.6	2.5	21
1969-1986	10^6 m^3 debris flow at Capricorn Creek	17	233	28.0	1.6	14
1986-1999	Several 10^5 m^3 debris flows in Meager Creek tributaries, 1.2 x10^6 m^3 debris flow at Capricorn Creek	13	213	25.6	2.0	16

The current rate of delta advance (16 m/yr) is approximately double the rate observed prior to the major engineering works. Because the river would have long since adjusted to the lake level lowering and channel straightening, the implication is that other factors are responsible for the "elevated" contemporary advance rate. A plausible explanation is that the delta is responding to increased sediment yield due to periods of rapid glacial retreat in the last century. Alternatively, the constriction of the river due to dyking and bank protection has restricted overbank flows from the floodplain. Hence, suspended material that would normally be deposited on the floodplain during floods is being transported through to the lake more effectively. This explanation is less likely given that high suspended sediment concentrations occur during low flow conditions also. Another possible explanation or factor is the increasing tend in maximum daily discharge of Lillooet River over the past 20 years (discussed in Section 6).

It should be noted that although the delta is advancing rapidly, sedimentation in the sand reach is dependent on gravel accumulations further upstream. If gravel did not accumulate upstream of km 8, the gradient of the river is sufficient to transport sand and finer sediment to the delta, and the delta, although advancing, would retain a relatively constant gradient. In other words, the channel bed in the sand reach would not aggrade significantly. Without gravel removals, however, the long-term trend of the river is for aggradation upstream of km 8. As the channel bed rises in the gravel reach due to deposition, the channel bed in the sand reach is also expected to rise accordingly.

Section 6

Climate and Hydrology

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6. CLIMATE AND HYDROLOGY

This section provides an analysis of historical climate data (snow and rainfall, Lillooet River discharge) for the Pemberton Valley, a listing of the most significant floods in recorded history, a review of previous hydrology studies and peak flow estimates, and definition of boundary conditions (flood hydrographs and lake levels) to be used for flood modelling in the Mike 11 model.

6.1 BACKGROUND AND HISTORICAL ANALYSIS

As noted previously, climate and hydrology are important factors in determining the planform of Lillooet River, and climatic characteristics vary notably across the watershed. The most pronounced difference is found between the humid upper watershed and the relatively dry valley bottom near Pemberton. Accordingly, mean annual precipitation in the watershed ranges between approximately 3,000 mm at higher elevations in the northwest to only 1,000 mm at Pemberton Airport. There is a pronounced fall and winter maximum of precipitation. The highest rainfall intensity and the highest peak discharge tend to occur from October through January.

There are two dominant hydro-climatic factors that influence the runoff regime of Lillooet River. One factor is snowmelt, which usually culminates in mid to late July when record temperatures are measured in Pemberton valley. Consequently, snowmelt floods are most frequent in July. The other factor is intense rainfall during fall storms on the western flank of the Coast Mountains, where Pacific cyclones cause prolonged, orographically enhanced precipitation. This is often exacerbated by rapid rises in freezing level associated with warm fronts from the central Pacific, often referred to as the "Pineapple Express". This scenario, where rain falls on autumn snow, usually occurs in October and November, before the snowpack is of sufficient thickness to absorb much rainfall before releasing it to the underlying ground.

Therefore, the type of precipitation as well as the associated freezing level are important. Waylen and Woo (1983) concluded that in southwestern B.C., 95% of spring floods are generated by snowmelt and over 95% of the winter floods are caused by rain or rain-on-snow. The October 1984 flood is an excellent example of an intense rain-on-snow event. This flood had the second highest peak instantaneous discharge on record, only exceeded by the August 1991 flood. This event was triggered by very intense rainfall associated with a Pacific frontal system, which caused extensive flooding in southwestern B.C.

Figure 6-1 shows the daily average discharge of Lillooet River near Pemberton for the 1914-1995 period (WSC gauge 08MG005). Lillooet River responds primarily to snowmelt as indicated by an inverse relation between mean monthly rainfall and mean monthly discharge. The watershed is heavily glaciated, and peak runoff is delayed until midsummer when most of the previous year's snow has melted. In 1948, below average

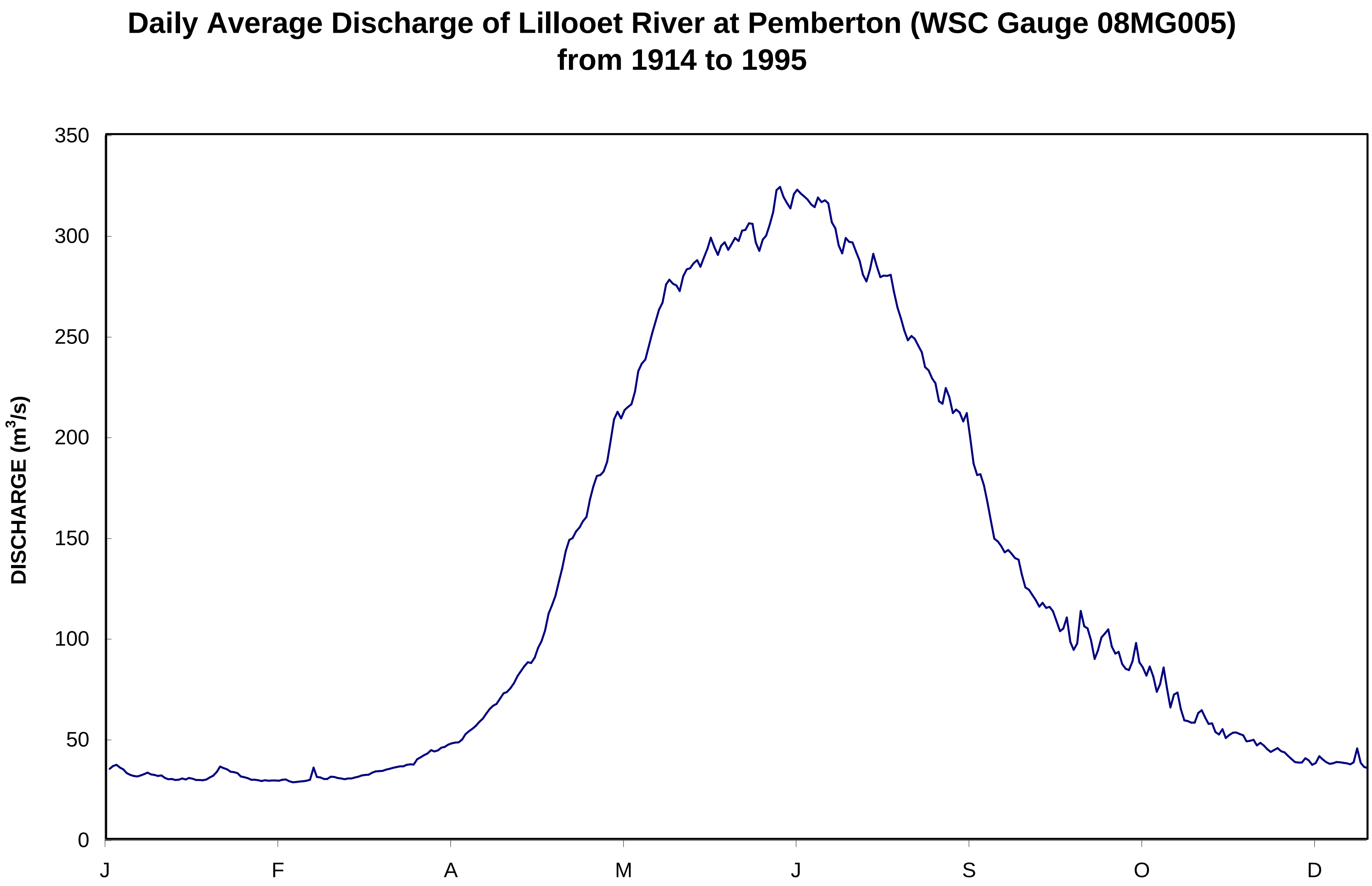
Daily Average Discharge of Lillooet River at Pemberton (WSC Gauge 08MG005)
from 1914 to 1995
DISCHARGE (m3/s)
350
300
250
200
150
100
50
0
J
F
A
M
J
S
O
D

Figure 6-1

temperatures and heavy snowpacks delayed snowmelt until May, when higher temperatures caused a sudden increase in runoff. 1948 was also the second largest flood recorded on the Fraser River.

All climate stations, river gauges and snow courses in the Lillooet River watershed with long term meteorological data, apart from Tenquille Lake, are located in the valley bottom, and do not represent the varied climatological conditions that determine the amount and seasonal variability of inflow to Lillooet Lake. Table 6-1 presents the periods of record of the hydrometric stations in the region. This is significant since hydroclimatic events that control the hydrology and sedimentology of Lillooet River are largely controlled by processes in the alpine and subalpine regions of the basins.

In the following paragraphs, snow, rainfall and Lillooet River discharge are analyzed to detect any long-term changes that may be indicative of future changes, and would thus have to be considered in any river mitigation schemes. Cumulative departure from the mean plots were used for the analysis, because they are convenient in displaying long-term changes. To generate these plots, the mean of each data series is determined and then subtracted from each individual value. The values, which can be positive or negative, are then summed. The cumulative value for each year is then plotted against time. A descending plot signifies persistently below average values; an ascending plot signifies persistently above average values; a horizontal plot indicates values persistently near average.

These analyses are precursory, and do not allow specific conclusions such as changes in night versus day temperatures, or precipitation and temperature changes for specific months, which may be more relevant to the runoff regime of Lillooet River. However, they provide an overview of prevailing conditions and are therefore helpful in assessing potential future changes.

SNOW WATER EQUIVALENT

At Tenquille Lake, a snow course operated by MWLAP has provided continuous data since 1953. The cumulative departure plot for snow water equivalent (measured on the first of March, April, May and June) shows a gradual increase in snowpack for all four months until approximately 1976. This increase is followed by a continuous decrease in snowpack until 1990 for March, April and May snowpacks. An anomaly is the June snowpack that has been subject to a delayed melt throughout the 1980s, suggesting persistently cooler springs. In the past 10 years, there is no clear trend discernible apart from very high snowpacks in 1999 that were measured throughout coastal British Columbia.

PRECIPITATION

Precipitation has been measured since 1913 at Pemberton Meadows and Pemberton airport. Mean annual precipitation in the Pemberton Valley shows a general decrease from 1915 to 1945, followed by random fluctuations until 1970 (Figure 6-2). Since that

Table 6-1
Summary of Hydrometric Stations

Station Number	Station Name	Drainage Area (km^2)	Period of Record	Type of Gauge (1)	Type of Data (2)	Type of Flow
08MG003	Green River Near Pemberton	855	1913 - 1951	R	C	Natural
08MG004	Green River Near Rainbow	195	1913 - 1948	M	C	Natural
08MG005	Lillooet River Near Pemberton	2160	1914 - Present	R	C	Natural
08MG006	Rutherford Creek Near Pemberton	179	1914 - 1948	M	C	Natural
08MG007	Soo River Near Pemberton	283	1914 - 1948	M	C	Natural
08MG008	Birkenhead River At Mount Currie	596	1945 - 1971	M	C	Natural
08MG019	Place Creek Near Birken	7.25	1969 - 1989	R	S	Natural
08MG021	Twentyone Mile Creek At 670 M Contour	28.2	1972 - 1985	R	S	Natural
08MG025	Pemberton Creek Near Pemberton	31.9	1987 - Present	R	C	Regulated

Notes:
(1) R is Recording, M is Manual
(2) C is Continuous, S is Seasonal

P:\0700-0799\713-002\Report\[TablesSection6_Dec09.xls]Table 6-1

time, annual precipitation has been consistently above average. It should be noted, however, that a number of years have incomplete precipitation records due to missing records from several months.

TEMPERATURE

A cumulative departure plot of mean annual temperature shows several fluctuations until the mid 1940s (Figure 6-3). From the mid 1940s until about 1970, mean annual temperature was below average. However, the past 23 years have shown an increase in mean daily temperature, which culminated in 1998 when the temperature in the Pemberton Valley exceeded 40 degrees Celsius for several days in late July. As with the precipitation record, however, these findings should be used with caution as a number of years have incomplete temperature records.

LILLOOET RIVER DISCHARGE

Daily discharge on Lillooet River has been gauged by the Water Survey of Canada (WSC) near Pemberton (station 08MG005) since 1914. The cumulative departure graph shows a steady decline in annual maximum daily discharge until approximately 1964 (Figure 6-4). A period of random discharge fluctuations followed until 1980. In the last 20 years, annual maximum daily discharge of Lillooet River has increased, however, with significant fluctuations from year to year.

SUMMARY OF CLIMATE TRENDS

In summary, snowpack has decreased over the past three decades but no trend is discernible at the present time. There is an increasing trend in mean daily temperature, mean annual precipitation, and maximum daily discharge of Lillooet River over the past 20 years or so. However, the precipitation and temperature trends are less reliable since a number of years have incomplete records (as noted by the data gaps in Figures 6-2 and 6-3.

It must be pointed out that there is only one reliable long-term rain gauge in Pemberton valley. Rainfall data from this gauge are by no means representative of the entire Lillooet River watershed, but rather reflect a minimum as frequently observed in wide valley bottoms. This effect is due to diverging air masses towards the centre of the valley, and the associated cloud dissipation. Adjacent hill sides receive significantly higher values of precipitation due to orographic uplift and the associated condensation. Therefore, a firm conclusion on the rainfall changes in Lillooet River basin cannot be made without adequate data from higher elevation rain gauges.

Accepting long-term predictions of climate change that claim higher winter precipitation and warmer summer temperatures, a further increase in maximum daily and maximum instantaneous discharge of Lillooet River can be expected in the next 50 to 100 years. This predicted increase is difficult to quantify without more comprehensive rainfall data, but may prove to be significant.

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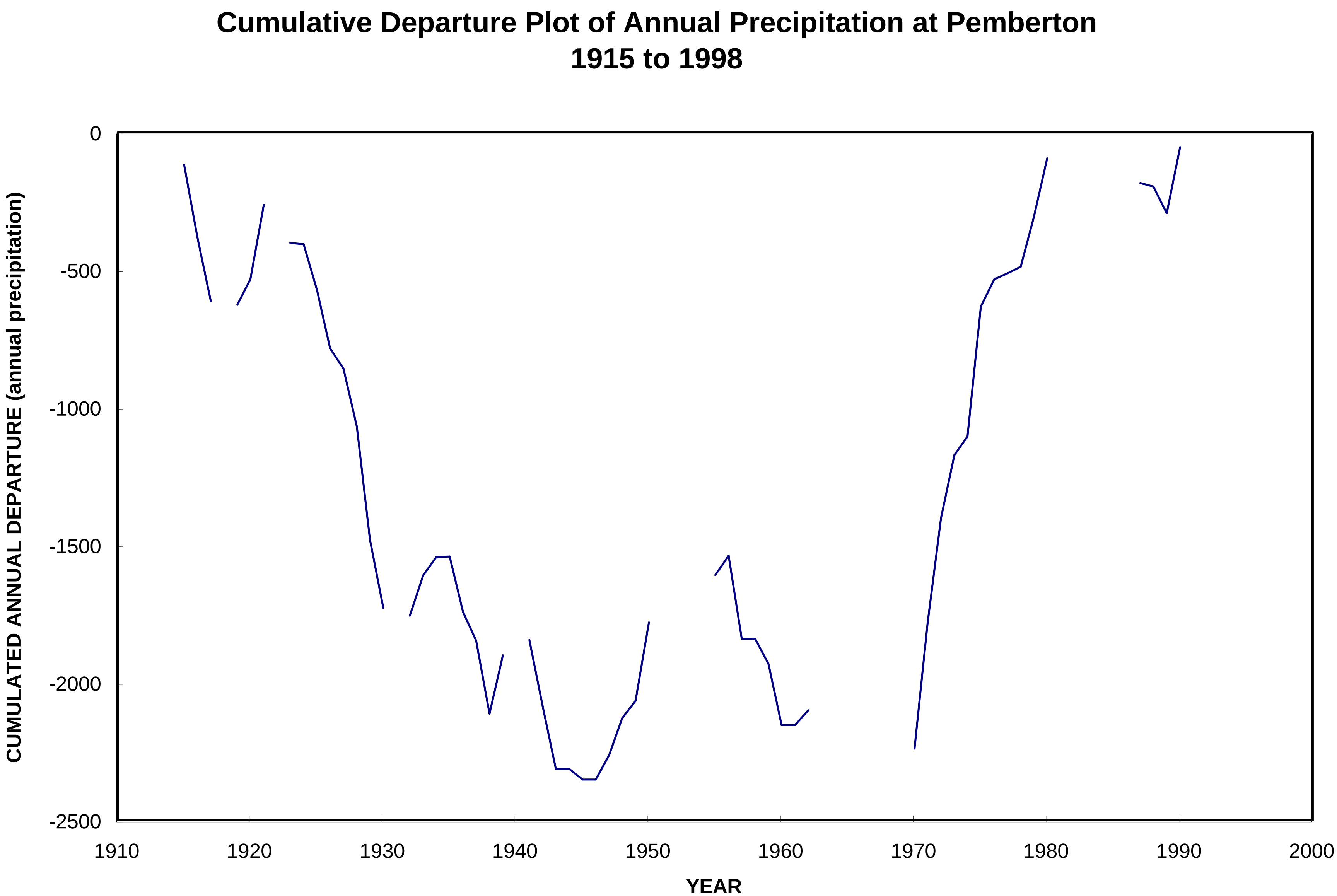

Figure 6-2

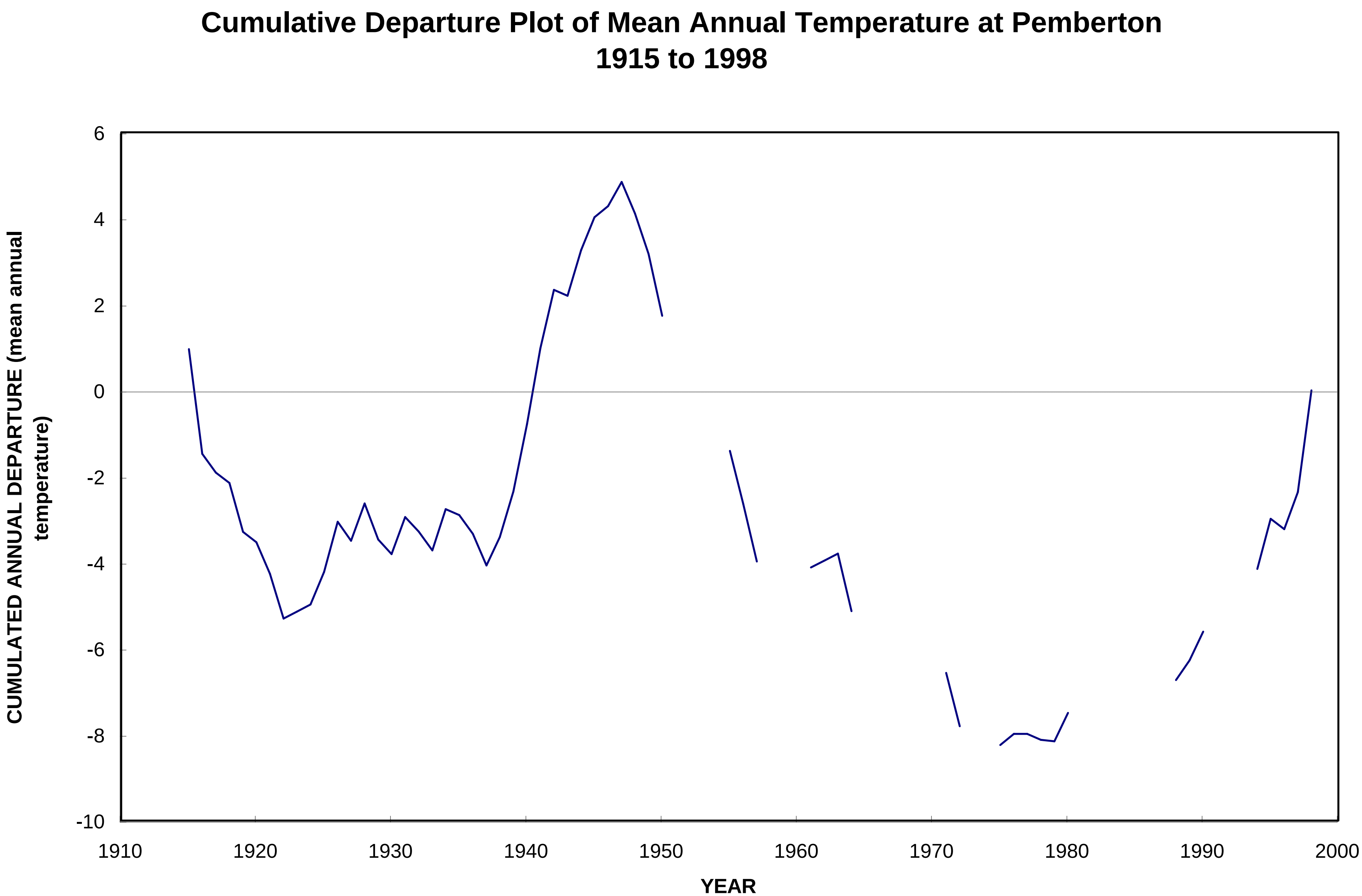
Cumulative Departure Plot of Mean Annual Temperature at Pemberton
1915 to 1998
CUMULATED ANNUAL DEPARTURE (mean annual temperature)
6
4
2
0
-2
-4
-6
-8
-10
1910
1920
1930
1940
1950
1960
1970
1980
1990
2000
YEAR

Figure 6-3

Cumulative Departure Plot of Annual Maximum Daily Flow Along Lillooet River (WSC Gauge 08MG005) - 1914 to 2000

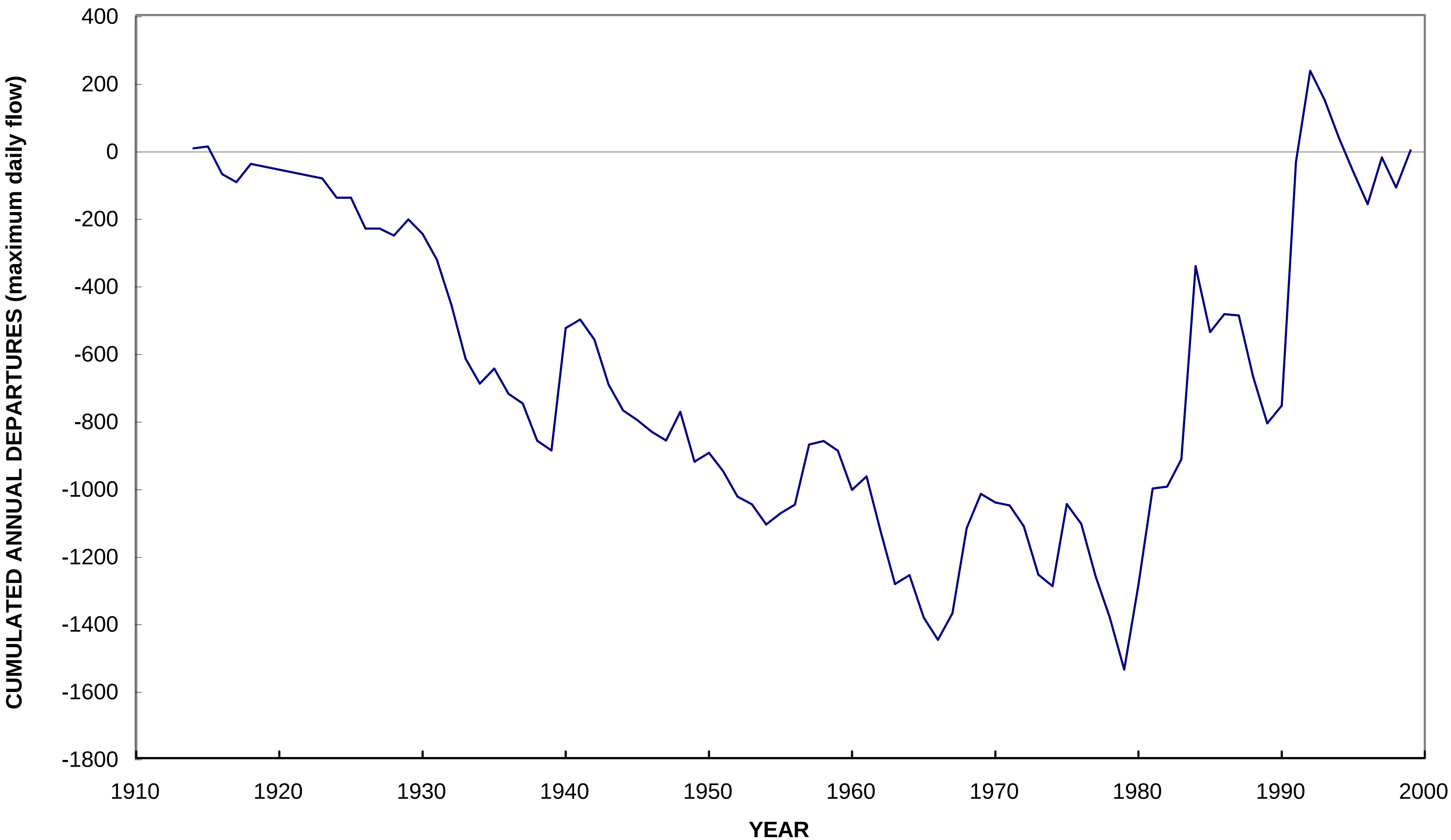

Figure 6-4

6.2 FLOOD HISTORY

The following paragraphs describe and comment on the largest floods that have occurred in the region since 1940. Table 6-2 summarises the magnitude of each event.

OCTOBER 19, 1940

By 1940, portions of the Pemberton valley were protected from low magnitude flooding by poorly constructed dykes. Generally high runoff was responsible for a relatively high ground water table and only higher lying crops had to be irrigated. In the fall of 1940, unseasonably cool weather had led to substantial snow accumulation in the surrounding mountains. On October 15, a warm front travelled north from the Pacific Ocean over the region. Temperatures began to climb rapidly and 125 mm of rain fell within 24 hours. The combination of high precipitation and a rain-on-snow event resulted in flooding of Lillooet River and its major tributaries.

Eyewitnesses to the flood described broken dykes, a flooding of the entire valley, and hundreds of livestock drowned. Some notable accounts are repeated here because they bear significance to floodplain delineation and river modelling: "…we could walk along the road [to Pemberton Meadows] and see the water flowing level with our heads….just above our farm, where the river took a big curve out and around the point, a huge cedar swept down on the flood-waters, jammed its point onto the curve of the dyke, swung around and pried an opening in the dyke…" (Dorothy Girling).

On October 19, "water covered the whole width of the valley [Lillooet River valley downstream of Pemberton] in several places and washed several buildings down the river" (from Burnett and McGugan: Report on Pemberton Valley Reclamation, Aug. 10, 1945).

The residents of Pemberton received no flood relief and little publicity.

JULY 1948

In 1946, work commenced along Lillooet River to prevent future flooding. The project, which involved dyke construction and meander cut-offs, was only half completed in July 1948 when high runoff (ranked 16th largest instantaneous flow) resulted in localized flooding. The recently completed dykes protected farms in the upper valley but much of the lower valley was flooded.

In 1950 another above-average flood threatened to inundate the parts of the valley, but the dykes held. It was noted, though, that the river level gradually dropped following the engineering works and many farmers had to purchase irrigation equipment since drying of their lands had become more of a problem than floods. By the mid-sixties, irrigation equipment was standard on most potato producing farms.

Table 6-2
Maximum Lillooet River Discharges

Date	Daily Peak (cms)	Instantaneous Peak (cms)	Comment
August 30, 1991	1260	1410	Up to this date the highest measured discharge was 795 m^3/s on August 8, 1991. This is the highest measured discharge in the period of record.
October 8, 1984	1110 (Estimated)	1310 (Estimated)	WSC noted that this event caused a shift in the rating curve. To account for “considerable ungauged flow – bypass” around dykes, the water level recorder trace was estimated to account for this.
October 19, 1940	900	1640 (Estimated, unpublished)	Unpublished value not incorporated into analysis. Value resulted in Instantaneous-to-daily peak ratio of 1.82, which is much higher than any other recorded peak.
November 1, 1981	823	897	
October 24, 1992	808	1010 (Estimated, unpublished)	Station Analysis form shows the instantaneous discharge as 1010 m3/s. There is no note indicating this was estimated – no reason has been discovered for not publishing the value.
December 27, 1980	790	993	Ice period occurred November 23 to December 13.
June 27, 1968	790 (Estimated)	827	Note on hydrograph plot: recorder inoperative June 27 to July 1; estimated from Birkenhead. June 27 values based on partial recorder chart record. Cableway rebuilt 1968.
November 5, 1975	782	858	
September 6, 1957	716	N/A	
June 18, 1997	676	851	
August 25, 1999	649	827	A shift in the hydrograph was noted after the peak.

OCTOBER, 8, 1984

On October 7, 1984 a broad frontal system moved onto the B.C. coast, focused in the Squamish area and central Vancouver Island. Extreme runoff associated with the system was attributed to the duration and north-south orientation of the front. The storm resulted in the largest flood on record for Lillooet River, and the second largest Lillooet River discharge on record. However, exact measurements of discharge were not possible because dykes were overtopped and failed in the Miller Creek area. As a result, considerable flow, estimated at 184 m^3/s, bypassed the gauging station. The adjusted daily peak flow was estimated at 1,110 m^3/s and the instantaneous peak flow at 1,310 m^3/s. During the flood, Lillooet Lake reached a then record level of 199.4 m.

Along the MacKenzie Cut, lateral erosion of this artificial reach caused the collapse of a privately constructed silt and berm dyke that had provided protection in the vicinity of what is locally known as Dr. Dill's farm. Repeated surveys have shown that this 4.4 km long reach has degraded and widened so that it no longer constitutes a flow constriction as noted by Tempest (1977).

At Ryan River, a debris flow in Nightmare Creek, a steep tributary to the river, probably caused a short-lived landslide dam. Subsequent overtopping of the landslide dam would have released a large reservoir of impounded water that overtopped the downstream dykes and resulted in flow diversion. The Pemberton Creek area was also flooded due to dyking and drainage inadequacies, and inflows from dyke failures north of the B.C. Railway embankment. At the B.C. Railway bridge, emergency protection prevented overtopping of the adjacent dykes.

Downstream from the Highway 99 bridge, the airport access road, which acts as a dyke, was overtopped by impounded water escaping to Lillooet River. The embankment of the BC railway tracks along the drainage canal near the town site of Pemberton restricted the passage of floodwaters that had overtopped the upstream dyke. This restriction caused severe flooding and property damage upstream and downstream from the railway. Floodwaters in excess of the culvert capacity ponded upstream of the railway tracks and then flowed through the Urdal Road underpass. Peak flood elevations were reached at 5:00 a.m. on October 9, 1984, almost 26 hours after the dykes had been overtopped.

The area in the vicinity of the airport is vulnerable to flooding from Green River, Lillooet River and Birkenhead River and was completely inundated during the October 1984 flood. The dyke in the vicinity of the North Arm Plug was overtopped and floodwaters inundated the area behind the dyke. Overbank flow downstream from the bridge flooded the remainder of the area.

In the vicinity of the Mount Currie I.R. No. 1, the flood overtopped a low access road that was built under the recent ARDSA program.

AUGUST 30, 1991

During late August 1991, unusually heavy rain fell in southwestern British Columbia. The Sea-to-Sky corridor received major flood damage and record floods occurred on several watercourses. In the Pemberton area, Lillooet, Green and Ryan Rivers experienced major floods. During this event, high water marks were obtained from Pemberton airport to Wolverine Creek, Lillooet Lake reached its highest level on record and the Duffey Lake Road was inundated along Lillooet Lake. Upstream of the Lillooet-Green River confluence, floodwaters inundated the airport as well as a temporary trailer camp. The August 1991 event resulted in the largest Lillooet River discharge on record, but flooding was significantly less severe than that in 1984 (due primarily to enhanced flood protection).

At MacKenzie Cut, the river eroded through a privately built agricultural berm, which resulted in flooding of pasture land. Floodwaters from Ryan River overtopped the Pemberton Meadows Road and breached the dyke in the vicinity of the rock quarry. Pemberton Meadows Road had to be breached artificially to release impounded floodwaters because the culverts were undersized.

At the forestry bridge over Lillooet River, near the upstream extent of dyking, damage was minimal. Floodwaters from Lillooet River did not reach the toe of the dyke. Downstream of the bridge minor flooding occurred as the river exceeded bankfull flow and followed a topographic depression.

On the lower Mount Currie reserves, several areas of the riverbank and access road were damaged, and approximately 1,300 lineal metres of riprap were washed out. Significant portions of the reserves flooded to an estimated depth of two metres. Many houses and access roads were flooded, creating emergency access/egress problems. Significant numbers of livestock were killed and washed downstream. Floodwaters did not recede for three weeks

The August 1991 flood demonstrates the variable susceptibility of different areas in Lillooet River valley to flood related damages. Upstream of the dyked section, flood damages were insignificant because very few people live in this area and little land is being used for agriculture. Furthermore, this section is not dyked and the river can flood inactive channels and distribute water through vegetated bar surfaces and the adjacent floodplain. Damage downstream of the forestry road bridge was related to dyke failures and the fact that floodwaters could not readily return to the river once dykes had been breached.

6.3 REVIEW OF HISTORICAL DATA

Water Survey of Canada (WSC) hydrometric survey data for 08MG005 Lillooet River at Pemberton were reviewed to determine the reliability of recorded daily and instantaneous annual maximum flows. Documents reviewed included station descriptions, stage-discharge curves and tables, annual station analyses, and daily discharge hydrograph plots. The review concentrated on high peak flow years, especially those for which annual maximums were estimated (e.g. 1940 and 1984). Mr. Kevin Dunk and Mr. Irv Neufeld, WSC hydrometric technician and area head respectively, provided valuable first hand knowledge.

A water level recorder and cableway were installed at the station in 1948-49. Prior to the cableway being installed measurements were obtained from the highway bridge with the highest measured discharge being 447 m^3/s compared to the annual maximum daily discharge of 900 m^3/s in October 1940. Gauge heights were obtained by manual (chain) gauge on the railway bridge downstream. In general, the data are of high quality and reliability.

However, there are a few annual maximum discharges whose reliability is difficult to determine. The October 20, 1940 instantaneous maximum has never been published by WSC although documents on file show that an attempt was made at the time to estimate overflow through a slough and roadway in the floodplain. There are too many unknowns to put much confidence in the estimate of 1,640 m^3/s. The ratio of instantaneous to the daily of 900 m^3/s is 1.86. Since that time the highest ratio has been 1.51 and, depending on season, usually in the order of 1.1 to 1.3. The 1940 instantaneous maximum was used in the derivation of peak Lillooet River flows for this study based on revised instantaneous-to-daily peak ratios. The effect of the 1940 instantaneous maximum on frequency analysis results is discussed in Appendix G.

The reliability of the unpublished, but provided by WSC, 1992 instantaneous maximum is not known, nor is it known why it is not published. 1996 extremes are not published, nor are any other data for 1996. Technically there is no reason for this, but because of a changeover in responsibility the data have never been approved (*pers. comm*. K. Dunk). But the documentation indicates that the maximums are below the long term mean.

6.4 PEAK FLOW ANALYSIS

Based on the above discussion, peak flow estimates were derived for the Lillooet River and five tributaries. A review was also done of the February 18, 2000 memorandum which assessed several peak flow estimates for Lillooet River at Pemberton. This review, and the subsequent peak flow analysis results are included in Appendix G. The review and estimates were completed by Mr. Donald Reksten in May 2001.

The historical annual peak flow summary for the Lillooet River WSC Gauge 08MG005 is show in Table 6-3, while the results of the analysis are presented in Table 6-4. Estimates from this table were used to generate the model hydrographs. For reference, historical Q_{200} estimates for Lillooet River and Birkenhead River are shown in Table 6-5.

Table 6-3
Annual Peak Flow Summary: WSC Gauge 08MG005 (Lillooet River Near Pemberton)

Year	Maximum Instantaneous Flow (m^3/s)	Date of Maximum Instantaneous (km^2)	Snowmelt or Rain Event (1)	Maximum Daily Flow (m^3/s)	Date of Maximum Day	Snowmelt or Rain Event (1)
1914	-	-	-	544	14-Oct	Rain on Snowmelt
1915	-	-	-	544	05-Jul	Snowmelt
1916	-	-	-	456	18-Jun	Snowmelt
1917	-	-	-	515	18-Aug	Snowmelt
1918	-	-	-	592	19-Jul	Snowmelt
1919	-	-	-	-	-	-
1920	-	-	-	-	-	-
1921	-	-	-	-	-	-
1922	-	-	-	-	-	-
1923	-	-	-	496	14-Jul	Snowmelt
1924	-	-	-	481	04-Jul	Snowmelt
1925	-	-	-	-	-	-
1926	-	-	-	447	12-Jul	Snowmelt
1927	-	-	-	-	-	-
1928	-	-	-	518	24-Jul	Snowmelt
1929	-	-	-	586	16-Oct	Rain
1930	-	-	-	496	11-Jun	Rain on Snowmelt
1931	-	-	-	462	01-Oct	Rain
1932	-	-	-	405	15-Jun	Snowmelt
1933	-	-	-	379	05-Sep	Rain on Snowmelt
1934	-	-	-	464	28-Jul	Snowmelt
1935	-	-	-	583	24-Jul	Snowmelt
1936	-	-	-	464	30-May	Snowmelt
1937	-	-	-	510	28-Oct	Rain
1938	-	-	-	428	23-Jul	Snowmelt
1939	-	-	-	510	29-Jul	Snowmelt
1940	-	-	-	900	19-Oct	Rain
1941	-	-	-	564	19-Jul	Snowmelt
1942	-	-	-	479	19-Aug	Snowmelt
1943	-	-	-	405	14-Aug	Snowmelt
1944	-	-	-	462	28-Jul	Snowmelt
1945	-	-	-	510	11-Jul	Snowmelt
1946	-	-	-	504	30-Jul	Snowmelt
1947	-	-	-	513	21-May	Snowmelt
1948	-	-	-	623	09-Jun	Snowmelt
1949	428	17-Aug	Rain on Snowmelt	391	17-Aug	Rain on Snowmelt
1950	572	17-Jun	Snowmelt	564	17-Jun	Snowmelt
1951	518	03-Jul	Snowmelt	484	03-Jul	Snowmelt
1952	501	15-Jul	Snowmelt	464	15-Jul	Snowmelt
1953	535	14-Jul	Snowmelt	515	14-Jul	Snowmelt
1954	-	-	-	479	02-Jul	Snowmelt
1955	631	17-Jul	Snowmelt	572	17-Jul	Snowmelt
1956	-	-	-	564	26-Sep	Rain
1957	-	-	-	716	06-Sep	Rain
1958	-	-	-	549	27-May	Snowmelt
1959	-	-	-	510	22-Jul	Snowmelt
1960	453	07-Jul	Snowmelt	422	07-Jul	Snowmelt
1961	609	31-Aug	Snowmelt	578	14-Jul	Snowmelt
1962	473	20-Aug	Rain on Snowmelt	374	20-Aug	Rain on Snowmelt
1963	422	22-Jul	Rain on Snowmelt	385	22-Jul	Rain on Snowmelt
1964	603	08-Jul	Snowmelt	564	08-Jul	Snowmelt
1965	462	08-Jul	Rain on Snowmelt	413	08-Jul	Rain on Snowmelt
1966	493	18-Jul	Snowmelt	473	18-Jul	Snowmelt
1967	660	23-Jun	Snowmelt	617	22-Jun	Snowmelt
1968	827	27-Jun	Rain on Snowmelt/Snowmelt	790	27-Jun	Rain on Snowmelt/Snowmelt
1969	674	13-Jun	Snowmelt	640	13-Jun	Snowmelt
1970	580	27-Jun	Rain on Snowmelt	513	27-Jun	Rain on Snowmelt
1971	558	01-Aug	Snowmelt	530	01-Aug	Snowmelt
1972	521	12-Jul	Rain on Snowmelt	476	13-Jul	Rain on Snowmelt
1973	436	23-Jun	Snowmelt	396	23-Jun	Snowmelt
1974	538	19-Jun	Snowmelt	504	19-Jun	Snowmelt
1975	858	05-Nov	Rain	782	05-Nov	Rain
1976	515	09-Jul	Snowmelt	479	08-Aug	Snowmelt
1977	422	14-Aug	Snowmelt	385	13-Aug	Snowmelt
1978	464	27-Jul	Rain on Snowmelt	416	27-Jul	Rain on Snowmelt
1979	472	03-Sep	Rain on Snowmelt	383	03-Sep	Rain on Snowmelt
1980	993	27-Dec	Rain	790	27-Dec	Rain
1981	897	01-Nov	Rain	823	01-Nov	Rain
1982	592	21-Jun	Snowmelt	544	21-Jun	Snowmelt
1983	704	12-Jul	Rain on Snowmelt	620	12-Jul	Rain on Snowmelt
1984	1310	08-Oct	Rain	1110	08-Oct	Rain
1985	380	01-Aug	Snowmelt	343	23-Jul	Snowmelt
1986	683	27-May	Rain on Snowmelt	592	26-May	Rain on Snowmelt
1987	651	12-Jun	Rain on Snowmelt	534	12-Jun	Rain on Snowmelt
1988	379	27-Jul	Snowmelt	358	27-Jul	Snowmelt
1989	432	14-Jun	Snowmelt	400	14-Jun	Snowmelt
1990	757	12-Nov	Rain	591	12-Nov	Rain
1991	1410	30-Aug	Rain on Snowmelt/Snowmelt	1260	30-Aug	Rain on Snowmelt/Snowmelt
1992	-	-	-	808	24-Oct	Rain
1993	543	14-May	Snowmelt	452	14-May	Snowmelt
1994	470	25-Jul	Snowmelt	426	24-Jul	Snowmelt
1995	-	-	-	439	26-Jul	Snowmelt
1996	-	-	-	-	-	-
1997	851	18-Jun	Rain on Snowmelt	676	18-Jun	Rain on Snowmelt
1998	486	29-Jul	Snowmelt	450	17-Jul	Snowmelt
1999	827	25-Aug	Snowmelt	649	25-Aug	Snowmelt
Number of Records	43			79		
Average	618	-	-	540	-	-
Median	543	-	-	510	-	-
Max	1410	-	-	1260	-	-

Notes:
(1) Snowmelt or Rain estimated by date of maximum flow, by examining pattern of peak, and by analyzing climate data from Pemberton Meadows, Pemberton BCFS, and Pemberton Airport Climate Stations

P:\0700-0799\713-002\Report\[TablesSection6_Dec09.xls]Table 6-3

Table 6-4
Peak Flow Estimates

Station Number		Lillooet River at Upper Forestry Bridge	Ryan River	Miller Creek at Mouth	WSC Gauge 08MG005 Lillooet River	Pemberton Creek at Mouth	WSC Gauge 08MG025 Pemberton Creek	Green River at Mouth	Birkenhead River at Mouth	Lillooet River at Lake
Drainage area	km^2	1570	419	78 (1992 MELP)	2160	51	32	868	638 (1992 MELP)	3162
DAILY ESTIMATES										
20 year return period	m^3/s	659	277	100	821	N/A	18	389	278	1043
50 year return period	m^3/s	787	348	124	968	N/A	N/A	483	354	1233
200 year return period	m^3/s	1021	503	171	1220	N/A	26	661	565	1518
INSTANTANEOUS ESTIMATES										
I/D Ratio		1.14	1.3	1.3	1.14		1.76	1.1	1.3	1.14
20 year return period	m^3/s	752	360	130	974	44	31	428	361	1190
50 year return period	m^3/s	897	452	161	1170	51	N/A	531	460	1406
200 year return period	m^3/s	1163	654	222	1520	64	46	727	735	1730

Notes:

P:\0700-0799\713-002\Report\[TablesSection6_Dec09.xls]Table 6-1

Table 6-5
Historic Design Flow Estimates

	Lillooet River (at Gauge 08MG005)		**Birkenhead River** (at Mouth)	
Year	**Instantaneous Peak Q_{200}** (m^3/s)	**Daily Peak Q_{200}** (m^3/s)	**Instantaneous Peak Q_{200}** (m^3/s)	**Daily Peak Q_{200}** (m^3/s)
2000 (for this study)	1,520	1,220	735	565
1988 (for 1991 Floodplain Mapping)	1,170	992	734	565

UPPER BOUNDARY CONDITIONS - HYDROGRAPH GENERATION

Six discharge hydrographs are required as upper boundary conditions for the hydrodynamic model. The paragraphs below explain the generation of the calibration and Q_{200} hydrograph sets.

Of the six rivers and creeks in the study reaches of Lillooet River, unregulated and detailed (hourly) hydrograph data for the 1991 flood of record are available only for the Lillooet River gauge (08MG005). For this reason, the shape of the August 30, 1991 hydrograph for Lillooet River was scaled and applied to the five tributaries. This approach, while possibly producing unrealistically shaped hydrographs for the tributaries, was considered reasonable in light of the general lack of data.

Hourly flow data for WSC Gauge 08MG005 (for 1991, and including 1981 and 1984 for comparison) is shown in Figure 6-5. The period chosen for calibration was August 1, 1991 to September 9, 1991, which encompassed the maximum historical peak discharge of 1,413 m^3/s at 19:00 H, August 30, 1991. This 40 day period included one month of antecedent flow to establish initial conditions in the model, and approximately one week of flow after the peak to allow water levels to subside.

The amount by which the 1991 hydrograph was scaled for each tributary was based on the ratio of the recorded peak flow in 1991 (1,413 m^3/s) to the estimated Q_{200} peak (1,520 m^3/s from Table 6-4), or 0.93. Each tributary peak estimate from Table 6-4 was multiplied by this factor to derive the assumed 1991 peak instantaneous discharge. The results are shown in Table 6-6.

1991 Recorded Hourly Peak Instantaneous Discharge at WSC Gauge 08MG005

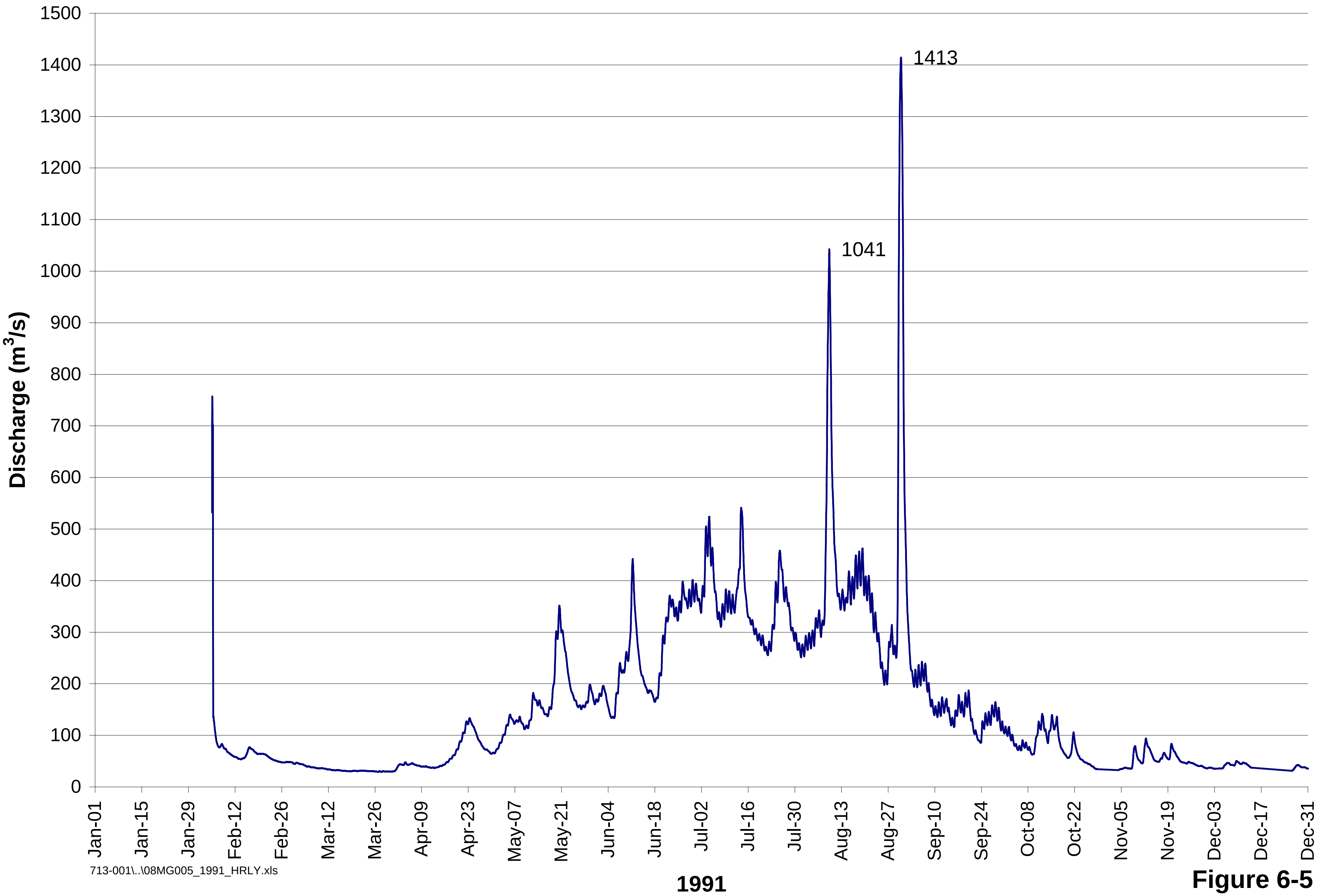

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Figure 6-5

Table 6-6
Derivation of Estimated 1991 Peak Discharge from Estimated Q_{200} Peaks

River/Creek	Estimated Q_{200} (m^3/s)	Assumed 1991 Peak (m^3/s)	Comment
Lillooet River	1,163	1,079	At forestry bridge
Lillooet River	1,520	1,413*	At gauge 08MG005
Birkenhead River	735	684	
Ryan River	654	607	
Green River	727	674	
Miller Creek	222	206	
Pemberton Creek	64	59	
* Recorded peak instantaneous discharge			

Ryan River and Miller Creek join Lillooet River above gauge 08MG005. The sum of these three 1991 peak estimates is 1,079+607+206 = 1,892 m^3/s, which is significantly higher than the recorded value of 1,413 m^3/s. Preliminary runs of the model showed that little peak attenuation resulted from flow routing, and since there was no justification for altering the peak discharge estimates or introducing losses into the model, attenuation could only be achieved by offsetting the tributary peaks.

The tributary hydrographs were advanced until the combined peak closely matched the recorded discharge at the gauge. The closest match was obtained with an advance of 28 hours, as shown in Figure 6-6. Analysis of the only overlapping set of continuous data in the system (Green River and Lillooet River: 1948 to 1951), showed that Green River peaked a minimum of five hours and a maximum of eight hours, before Lillooet River. Unfortunately, there were no events even slightly significant during the period of overlap. The obvious discrepancy in required and found peak offsets highlights the pitfalls of estimated data based on little data. This concern is noted in Section 7, and stated as a limitation.

The Q_{200} (and Q_{50}) hydrograph sets were generated by scaling the 1991 hydrograph shape in a similar manner to that described above. Each hydrograph set was exported from Microsoft Excel, and imported directly into the Mike 11 model.

LOWER BOUNDARY CONDITION - LAKE LEVEL GENERATION

The lower boundary condition of the model consists of a single lake level. A preliminary revised Q_{200} lake level was estimated based on 26 years of recorded instantaneous maximum levels at WSC Gauge 08MG020 (Table 6-7). A Q_{50} lake level was also estimated.

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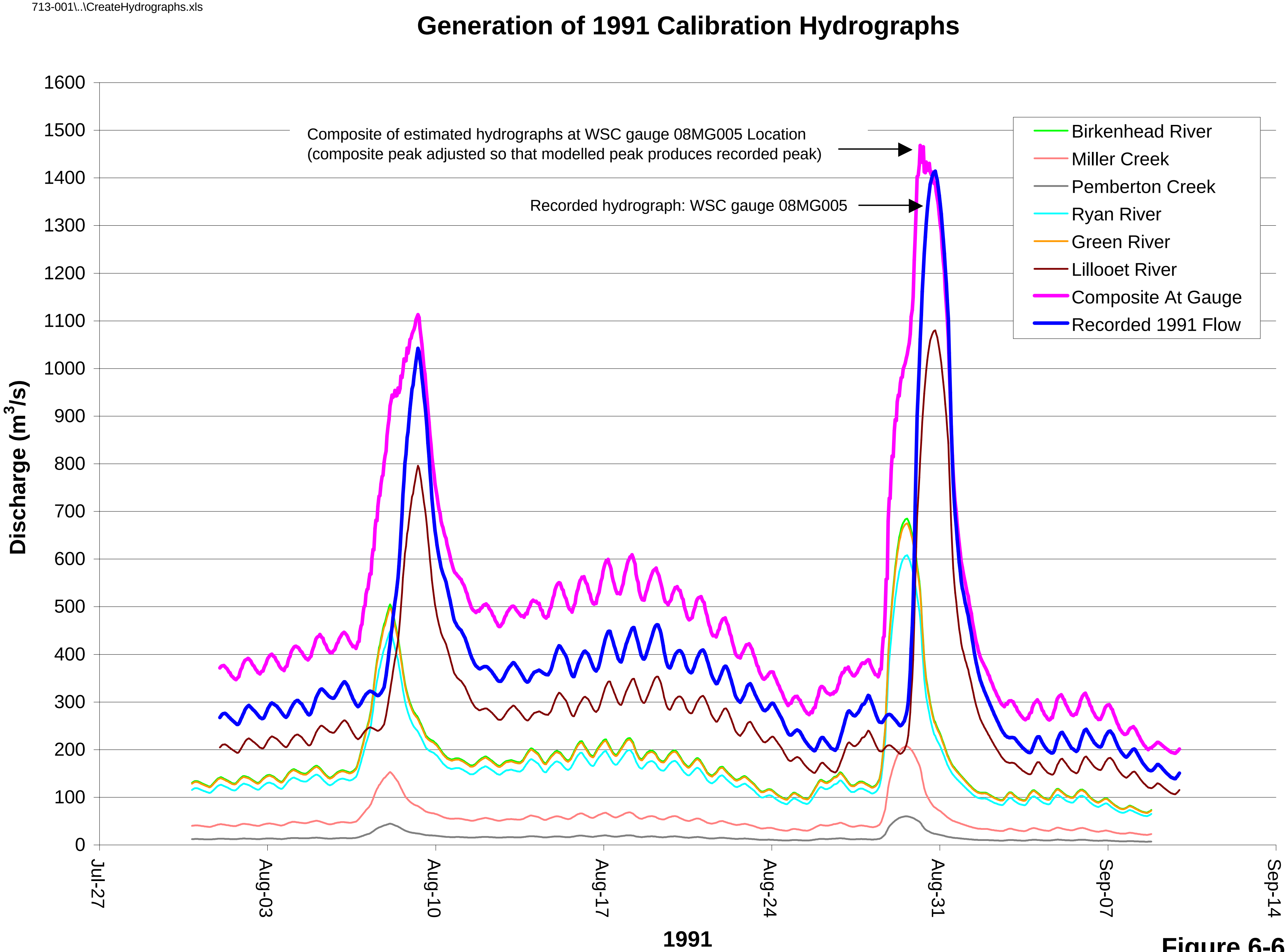

Figure 6-6

Table 6-7
Peak Historical Lillooet Lake Levels

Year	**Maximum Instantaneous Lake Level** (m Geodetic)
1972	197.93
1973	197.15
1974	198.16
1975	198.53
1976	197.77
1978	197.51
1979	196.77
1980	198.31
1981	198.25
1982	198.38
1983	198.16
1984	199.36
1985	197.04
1986	198.33
1987	197.99
1988	197.10
1989	197.46
1990	198.63
1991	**199.70***
1993	197.86
1994	197.15
1995	197.50
1997	198.68
1998	197.55
1999	198.41
2000	197.48
* Peak level for period of record	

Fitting the data to four different extreme value probability distributions produced the lake level estimates in Table 6-8:

Table 6-8
Estimated Q_{200} and Q_{50} Lillooet Lake Levels

Distribution	**Q_{200} Estimate** (m Geodetic)	**Q_{50} Estimate** (m Geodetic)
Log Pearson Type III	200.4	199.86
3-Parameter Log Normal	200.2	199.59
General Extreme Value	200.2	199.62
Non-Parametric	200.3	199.84
Average	**200.3**	**199.73**

Each of the above distributions provided an excellent fit to the data, so the average of the four was used. The lake levels used as lower boundary conditions for the model are presented in Table 6-9. These levels do not include freeboard.

Table 6-9
Lake Levels for Model Runs

Model Run	**Lake Level**
Calibration (1991 Peak)	199.70
Q_{200}	200.30
Q_{50}	199.73

The previous Q_{200} lake level estimate, used in the previous modelling exercise and subsequent floodplain mapping, was 199.73 m. This value had 1.27 m of freeboard added to produce the final design flood level for the lake of 201.0 m (Nichols, 1989).

Section 7

Flood Modelling

KERR WOOD LEIDAL
associates limited
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7. FLOOD MODELLING

This section describes the issues, development and calibration of the Mike 11 hydraulic model, and presents the results. The subsections discussing assumptions, data requirements and model limitations are fairly extensive, as it is important to recognize that computer models of natural physical systems are often gross simplifications of true physical processes. This is particularly true when one-dimensional models (such as Mike 11 and HEC-RAS) are applied to systems that are clearly not one-dimensional. Within the correct constraints, however, such models can provide very good estimations of the desired objective functions (in this case, flood level).

7.1 MODEL CHOICE: MIKE 11

The Mike 11 model (DHI Software Inc., www.dhisoftware.com) was selected for this exercise because of the model's support of unsteady flow, quasi-two-dimensional floodplain modelling capabilities, and stable resolution of diverse hydraulic conditions. The support of unsteady flow allowed the initial conditions of the model to be derived in a more realistic means. Additionally, the model may ultimately be linked to an evolving model of Harrison Lake and Harrison River, done by others.

7.2 ASSUMPTIONS AND DATA REQUIREMENTS

When applied to natural river systems, one-dimensional models rely upon several simplifying assumptions, which although in reality are never satisfied, are often 'close enough' to provide reasonable results. These assumptions are necessary to allow simplified mathematical models to be used to solve for flow velocity and water level at any given point. The simplified solution schemes are not trivial, however, and still require substantial computational gymnastics to solve. Highly sophisticated (two-dimensional) models are available, but they require much more set-up, very detailed surface data, and a relatively large amount of computational power, particularly for an area the size of the present study region.

In the case of this study, a one-dimensional model will provide the required accuracy, particularly given the limited state of existing floodplain and hydrologic data. Listed below are the assumptions and data requirements underlying the development of the Mike 11 model of the Lillooet River and tributaries.

ASSUMPTIONS

Assumptions fall into two categories: general ones relating to the validity of applying a one-dimensional model to a natural river system, and specific ones, pertaining how the model has been applied in the study area.

General

- Flow is one-dimensional: the water surface is horizontal across the entire cross section, and the water velocity is the same at all locations in the cross section.
- Channel bed slope is a relatively low gradient.
- Water pressure is hydrostatic.

Specific

- Modelling was done with Mike 11 HD module, version 2000b.
- The model represents existing conditions as of approximately November 2000 (the date of the cross section survey for the modelling and geomorphological analysis);
- 'Standard' dykes confine the flood where they exist. Where the water levels are above designated levee marks within a cross section, Mike 11 will artificially extend the cross section boundary vertically to contain the flood.
- Where formal dykes do not exist, the channel is confined by valley walls, or by linear fills (roads, rail lines). Where river and creek floodplains meet, a somewhat arbitrary but common cross section boundary has been established. Wherever possible and reasonable, such common boundaries have been aligned with those used in the 1988 HEC2 modelling exercise.
- The floodplain is included as part of the cross section where no dykes exist.
- Floodplain cross section extensions have been digitized from 1990 1:5,000 floodplain mapping with 1 m contours, and follow a similar schematization to that used in the 1988 HEC2 modelling exercise.
- All elevations are metres geodetic, and are relative to CVD28. Datums are discussed in detail in Subsection 4.1, and in a separate three-ring binder document (Kerr Wood Leidal, 2001)
- Mike 11 link channels are used to allow interaction between Lillooet River and Birkenhead River during high flood. This is explained further below in Subsection 7.4.
- The 200-year instantaneous peak lake level is assumed to occur at the same time as the instantaneous peak Lillooet River discharge. In reality, the lake level peaked 19 hours after the peak Lillooet River gauged discharge occurred.

Lake Level and Lillooet River Discharge of Record

	19:00 August 30, 1991	14:00 August 31, 1991
Lake Level	N/A	199.70 m (record)
Discharge	1413 m^3/s (record)	719 m^3/s

- Lillooet Lake level is accepted as the lower boundary condition of the model.
- A constant lake level was used for the duration of each model run to simplify the data set-up and analysis (Cunge et. al., 1980); this did not significantly effect the final results.

- Upper boundary conditions for model are hourly hydrographs, as defined in Section 6.
- Tributary hydrograph shapes are assumed to be the same shape as (but scaled differently than) the August 30, 1991 Lillooet River peak event.
- The hydrographs applied to the tributaries were staggered so that correct peak discharge was achieved at gauge 08MG005 for the calibration event. Tributary discharge peak estimates were based on Q_{200} instantaneous peak estimate ratios, as explained above. While this configuration of discharges is not likely realistic, it is considered a reasonable approach given the lack of additional data.
- Because of extreme 'spikiness' of maximum peak discharge (see graph of discharge comparison), it was determined that peak discharge would govern (provide the highest water level), and a model run of average daily discharge was not undertaken. It is also noted that in the 1988 modelling exercise, instantaneous peak discharge governed in almost all cases.
- Initial conditions are based on several days of preliminary run (derived from the hourly 1991 Lillooet River flood hydrograph), preceded by a 'hot start' configuration which provided a stable solution base for the model.
- Roughness coefficients (depicted as Manning's n values in the model) are not completely representative of true bed resistance. These factors are essentially calibration parameters, and comprise main channel and overbank (floodplain) roughness, losses at meanders, bank impingement, and other factors.
- Calibration of the model comprises matching the 1991 discharge peak at gauge 08MG005, and water levels at 1991 high water marks along the rivers and creeks.

DATA REQUIREMENTS

The following represent the most significant sources of data that comprise the model:

- Derivation of input hydrographs and lake levels was discussed in Section 6. All modelled results are derived from peak instantaneous flows.
- Survey data from 103 newly-surveyed cross sections were translated into Mike 11 RAW ASCII format for import into the model using custom data translation software.
- Additional floodplain topography was manually extracted from floodplain maps and entered directly into Mike 11.
- 1991 flood High Water Mark (HWM) documentation was obtained from MWLAP, which was combined with confluence locations, FCL elevations, and other information, and added into model for calibration and reference purposes.
- Bridge as-built drawings (or, where not available, construction drawings) were obtained, and bridge geometry was extracted. Mike 11 requires that bridges be modelled as combinations of culverts and weirs at the same river chainage. Bridge openings were scaled and translated into culverts, while bridge decks and adjacent roadways were translated into weirs.

- Dyke crest elevations were obtained from the initial cross section survey, from additional dyke surveys undertaken by others, and from ad-hoc spot elevations captured along accessible dykes by the KWL control survey crew as required.
- Field review and confirmation of relevant new information (e.g., private dyke locations) was done and incorporated into the final version of the model.
- All eight modelled bridge openings were surveyed for his project. Bridges were modelled as combinations of culverts and weirs at the same river chainage. Bridge openings were scaled and translated into culverts, while bridge decks and adjacent roadways were translated into weirs.

7.3 LIMITATIONS

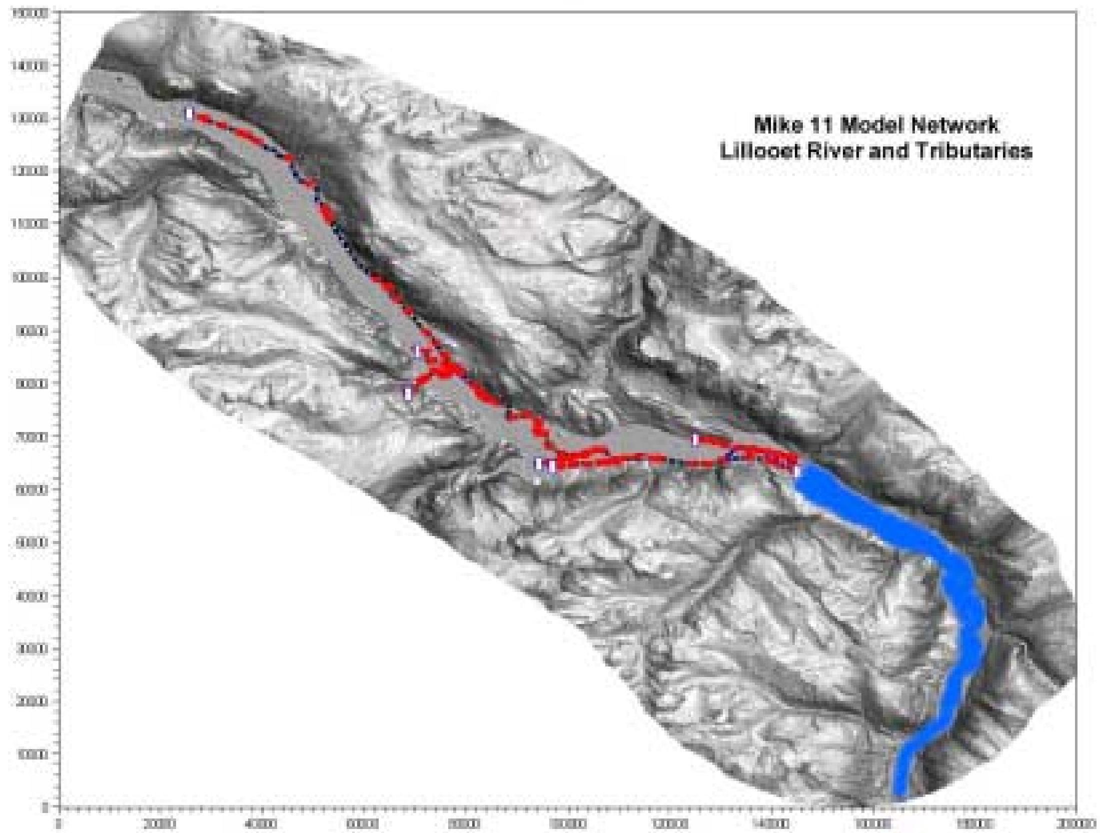
In considering the modelled results, it is important to keep in mind the limitations inherent in computer models in general, and those specific to each specific application.

- The most significant limitation is lack of hydrologic data on which discharge estimates are based. There is only one currently active and uncontrolled hydrometric station on the river system (08MG005), and one lake level gauge (08MH020).
- The very limited number (103) of cross sections over 58 km of river means that many water surface elevations are based on interpolated cross sections, which often do not reflect the true morphology of the rivers and creeks.
- The limited available floodplain data is only as good as the 1:5,000 scale, 1 m contour mapping from which it was extracted.
- The calibration event (discharge and associated HWMs) was the 1991 flood, but the model is based on 2000 cross sections. Some discrepancies in calibrated levels and surface slope are due to changes in morphology between the 1985 and 2000 surveys.
- Peak discharges of tributaries are all assumed to occur at the same time (but are staggered from Lillooet River peak). The time of concentration of each tributary cannot be easily determined due to the lack of hydrologic data.
- The Q_{200} discharge results in a modelled water level approximately 0.15 m to 0.5 m higher than the 'calibrated' 1991 water level, meaning that the calibration departure can contribute 40% or more to the error of the Q_{200} modelled level (0.03 m mean departure / 0.15 m level increase of Q_{200} over 1991 = 20%).
- While all elevations are assumed to be to the same datum, it is apparent that there are slight discrepancies between some new and previously-surveyed elevations. Also, the use of different survey methods and reference benchmarks have resulted in slight and varying vertical offsets between various data points (Kerr Wood Leidal, 2001).

7.4 PROCEDURE

The model schematization derived for Lillooet River and five tributaries sought primarily to address the adequacy of existing dyking. Additionally, the model was to identify areas where additional flooding can be anticipated, and to produce revised Q_{200} design flood levels. The Mike 11 model of Lillooet River and five tributaries is represented in Figure 7-1.

Figure 7-1
Mike 11 Model Network of Lillooet River and Tributaries



The modelling exercise proceeded in two phases:

- the Phase I (preliminary) model was derived for existing conditions as of November 2000, and used existing bridge information (from as-constructed and design drawings), and finalized cross section and input hydrograph data. This model was calibrated to the 1991 high water marks (discussed further below). The Phase I model results were presented in the Phase I Draft Report.

- the Phase II model incorporated new bridge survey data, three additional creek cross sections (Miller Creek), selected GPS spot elevations along dykes, roads, swales, floodplain, etc. – all of which were obtained as part of this exercise. The Phase II model also linked together the Birkenhead River and Lillooet River in Area 8 to more realistically model the interaction between the two during high flood. Calibration was checked, and parameters adjusted slightly to obtain twice the calibration precision as the Phase I model. Again, the model represented the conditions as they existed in November 2000.

Nothing significant has occurred within the modelled area between the initial survey and the time of writing that would impact results. The Phase II (final) model results are presented in this report.

MODEL INPUTS

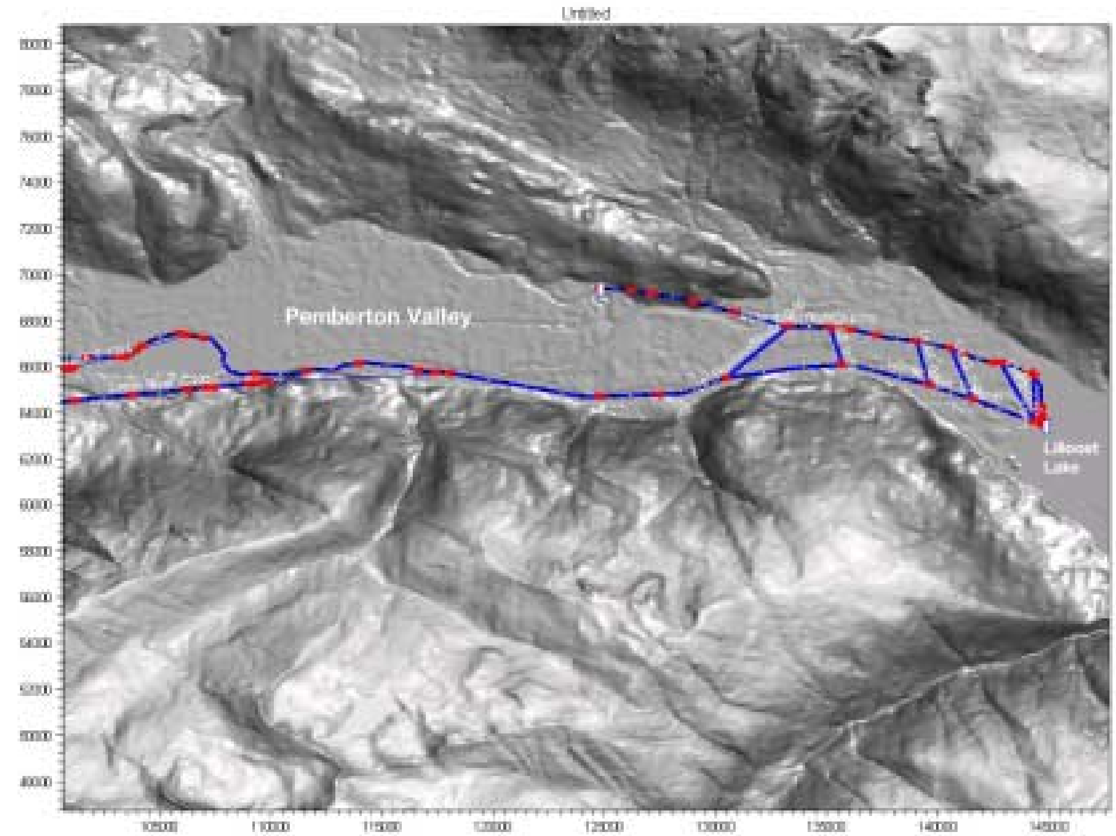
The 103 river cross sections obtained for this study were assessed for reasonableness by comparing with previous surveys (where possible). The survey data were then translated into a format suitable for importing into Mike 11. Once imported, many cross sections were modified by adding manually digitized floodplain data on one or both sides of the cross section (where standard dykes do not exist) to allow for overbank flow. The same approach was used in the previous HEC2 modelling exercises in 1988.

Note that Mike 11 treats such extended cross sections incorporating floodplain differently than the HEC2 model. HEC2 will confine flow to the main river channel until a certain (top of bank) water surface elevation is achieved, after which water is allowed to flow in the floodplain. The floodplain is usually at an elevation below the top of bank. Mike 11, however, has no similar facility, and inundates the floodplain part of the extended cross section as soon as the water level rises above the floodplain elevation. The model can therefore have flow passing over the floodplain before the water has reached the top of bank elevation. For the purposes of this modelling exercise, this is of little consequence, since almost all of the sections extended to include floodplain become fully inundated in the Q_{200} peak instantaneous flood.

Lillooet River and Birkenhead River channels were joined with Mike 11 link channels in six locations (Figure 7-2). These link channels allow flow from either channel to cross into the other should the flood exceed preset levels. In this way, the interaction between the two rivers can be modelled more realistically during high flood conditions. In total, the link channels allow flow to potentially pass over 3.1 km of Highway 99 between Seymour Slough and the Highway bridge across Birkenhead River. This allows the majority of the floodplain between the two rivers below Seymour Slough to be inundated and connected during high flood.

The approximate extents of the modelled floodplain areas are indicated on the figures in Appendix B using a white dashed line.

Figure 7-2
Link Channels Joining Birkenhead River and Lillooet River



The August 1991 and estimated peak instantaneous Q_{200} hourly discharge hydrographs (discussed in Section 6) were imported into Mike 11, and set as upper boundary conditions to each river and creek. The lower boundary condition of the model was set to a constant lake level, with the lake level selected based on the model run (calibration, Q_{50}, or Q_{200})

CALIBRATION

A series of 32 recorded 1991 high water marks (HWMs) were entered into the model as calibration targets. The HWMs were distributed as follows:

- 25 on Lillooet River;
- 3 on Ryan River;
- 2 on Green River;
- 1 on Pemberton Creek; and
- 1 on Miller Creek.

Unfortunately, no 1991 HWMs were identified on Birkenhead River, so the calibration of Birkenhead River levels was not possible with confidence. One HWM at the very top of the model (above the forestry bridge on Lillooet River) was discarded, as it is unclear whether flows were confined to the river at that point.

Using the 1991 hydrograph set, channel roughness coefficients were adjusted in all channels to alter the water level and water surface slope to obtain a good match between water level and 1991 HWMs. In some cases, culvert roughness and expansion/contraction loss coefficients were adjusted at bridges (bridges are modelled as composite structures of culverts and weirs in version 2000b of Mike 11).

The mean absolute departure of the modelled water levels from the 1991 high water marks on Lillooet River was 0.03 m, with a standard deviation of 0.03 m. This is considered to be an excellent fit. The cumulative mass error (the amount of water added or subtracted by the solution schemes in the course of solving the equation systems) was negligible. Using a 30-minute time step and computational grid spacing of 500 m in each model branch, the Courant numbers (Cunge et. al., 1980) for all points fell within a in a reasonable range (mean 16.6, maximum 27.5, minimum 7, standard deviation 4.8), indicating a good combination of grid spacing and time step. The calibration departure is shown in Figure 7-3.

VERIFICATION

The only other set of recorded HWMs which could provide verification targets were those from the 1984 flood. Sufficient changes (dyking, etc.) had occurred in the valley since that time to render these elevations meaningless for an 'existing conditions' model. No verification data sets were therefore available.

However, the estimated Q_{200} instantaneous peak discharge at the Lillooet River gauge was only 8% larger than the recorded peak instantaneous flow in 1991:

1991 recorded peak instantaneous flow:	1,413 m^3/s
2000 estimated Q_{200} peak instantaneous flow:	1,520 m^3/s (8% larger)

The closeness of these two flow values allowed a reasonable degree of confidence in the results for the Q_{200} model run.

Q_{200} AND Q_{50} RUNS

Once matched to the 1991 HWMs, the calibrated model was run with the Q_{200} (the design flood) and Q_{50} instantaneous peak hydrograph sets and lake levels. The resulting water surface elevations were extracted from the model, and became the design flood levels. Existing dyke elevations (where available) were compared to modelled and interpolated water surface elevations. Water surface elevations can be extracted from the model at intermediate model-generated grid points. These result values were extracted and used, where possible, to more accurately determine dyke adequacy.

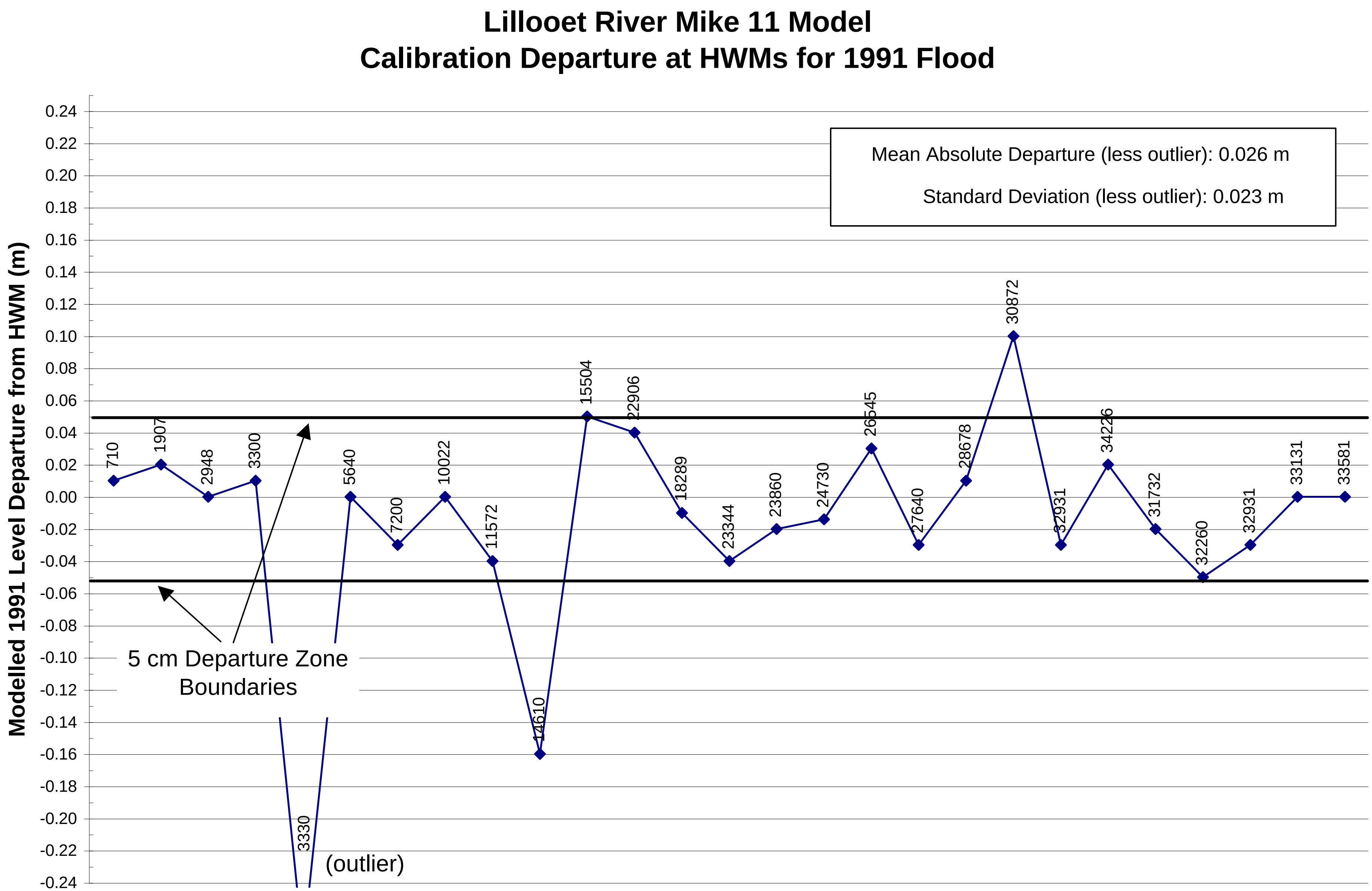


Figure 7-3

(Data Labels are Lillooet River Model Chainage Downstream from XS-56)

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The complete modelled level profiles of Lillooet River and tributaries are shown in Figures 7-4 through 7-9.

A review of the completed Phase I (preliminary) model was done by Jesper Kjelds of the Danish Hydraulic Institute (DHI), and is included as Appendix H.

POTENTIAL DYKING IMPACTS – ADDITIONAL MODEL RUNS

Cross section modifications were made to simulate setback dykes in several 'what-if' scenarios. The model results indicated the expected water level increases for upstream, adjacent and downstream properties. This approach was useful for addressing property-owners' concerns about potential impacts to their properties due to dyking by others. The configurations and results of these runs are presented below.

7.5 RESULTS

Model runs were conducted for three sets of conditions. The first results were produced for existing conditions – the base model. As a planning exercise, two additional dyking scenarios were modelled by modifying the base model. These are discussed below.

BASE MODEL - EXISTING CONDITIONS

Resulting modelled water levels in each river and creek are shown on the River Profile Figures, 7-4 through 7-9. The levels shown are the raw numbers produced by the model (design flood levels), and do not include any allowance for freeboard.

The design flood levels are indicated on the map figures in Appendix B for Lillooet River and Birkenhead River. Levels have been interpolated and noted in 0.5 m increments. Note that the width of the flood level indicators on the figures denotes neither flooding extent nor cross section extent, but merely that the level is assumed constant across the modelled cross section.

The modelled maximum instantaneous peak Q_{200} (design flood levels) and Q_{50} levels are presented in Appendix I. The resulting design flood levels are compared to previous modelled levels and FCLs in Appendix J.

It is noteworthy that only one bridge structure presented flow obstructions: the log bridge on Miller Creek at cross section 6. Table 7-1 presents the modelled bridge lower chord elevations, the corresponding peak flood elevations, and the freeboard.

Table 7-1
Modelled Q_{200} Bridge Clearances

River/Creek	Model Chainage	Description	Bridge Low Chord Elevation	Instantaneous Peak Q_{200} Level	Free-board
Birkenhead River	4350	XS-5.2: Xit'olacw Road	201.70	201.43	0.27
Birkenhead River	8200	XS-0.4: Highway 99	201.70	200.32	1.38
Lillooet River	31050	XS-15.2: Highway 99	208.74	207.39	1.35
Lillooet River	27635	XS-19.2: BC Rail	211.30	210.68	0.62
Lillooet River	3315	XS-52.1: Forestry Road	234.95	232.05	2.90
Pemberton Creek	980	XS-1: Airport Road	207.50	206.68	0.82
Miller Creek	**260**	**XS-6: Log bridge**	**218.51**	**218.70**	**-0.19**
Miller Creek	850	XS-4 : Pemberton Meadows Road	215.42	214.04	1.38

DYKE SCENARIO 1 – TYING TO HIGH GROUND IN AREAS 7/8

Dyke scenario 1 (DS1) is represented in Figure 7-10. This modelled dyke alignment required the low dyke in I.R. Nos. 1 and 8 to be raised, and then extended across the highway, tying into high ground in Area 7. The total length of dyke modelled is approximately 1,900 m.

The impact on the peak instantaneous Q_{200} levels relative to the existing conditions results is shown in the table below.

Table 7-2
Modelled Maximum Instantaneous Peak Q_{200} Levels for DS1

X-Sec. ID	Chainage	Existing Conditions: Level	DS1: Level	Level Difference (m)
LI17	29609	208.76	208.79	0.03
LI16	30486	208.11	208.15	0.04
LI15_4	30872	207.66	207.71	0.05
15.3 Est.	30970	207.44	207.51	0.07
LI15_2	31130	207.34	207.41	0.07
LI15	31231	207.57	207.64	0.07
LI14_2	31732	207.02	207.05	0.03
LI14_1	31846	206.67	206.69	0.02
LI14	32040	206.35	206.38	0.03
LI13	32931	206.08	206.15	0.07
LI12	33581	205.80	205.91	0.11

X-Sec. ID	Chainage	Existing Conditions: Level	DS1: Level	Level Difference (m)
LI11	34256	205.27	205.47	0.20
LI10	35001	204.57	204.62	0.05
LI9.2	35220	204.44	204.43	-0.01
LI9.1	35407	204.35	204.33	-0.02
LI07	37322	203.26	203.29	0.03
LI06	38078	202.91	202.96	0.05
LI05	38943	202.39	202.48	0.09
LI03	40417	201.51	201.57	0.06

DYKE SCENARIO 2 – EXTENDING THE DYKE THROUGH AREA 7

Dyke scenario 2 (DS2) is also represented in Figure 7-10. This modelled dyke alignment required the low dyke in I.R. Nos 1 and 8 to be raised, and then extended through Area 7, on the same alignment as that recommended in the report: *Pemberton Valley Flood Protection 1985 Study*, and identified in drawing 85-13-9, sheet 4 of 5. The total length of dyke modelled is approximately 5,500 m.

The impact on the peak instantaneous Q_{200} levels relative to the existing conditions results is shown in the table below.

Table 7-3
Modelled Maximum Instantaneous Peak Q_{200} Levels for DS2

X-Sec. ID	Chainage	Existing Conditions: Level	DS2: Level	Level Difference (m)
LI21	26275	210.97	211.00	0.03
LI20	27600	210.95	210.97	0.02
LI19_5	27228	211.09	211.11	0.02
LI19_4	27528	211.05	211.07	0.02
Interpolated	27600	210.79	210.82	0.03
LI19_2	27670	210.58	210.62	0.04
LI19_1	27748	210.60	210.64	0.04
LI19	28018	210.45	210.49	0.04
LI18	28778	209.59	209.66	0.07
LI17	29609	208.76	208.89	0.13
LI16	30486	208.11	208.32	0.21
LI15_4	30872	207.66	207.95	0.29
15.3 Est.	30970	207.44	207.76	0.32
LI15_2	31130	207.34	207.69	0.35
LI15	31231	207.57	207.85	0.28

X-Sec. ID	Chainage	Existing Conditions: Level	DS2: Level	Level Difference (m)
LI14_2	31732	207.02	207.21	0.19
LI14_1	31846	206.67	206.79	0.12
LI14	32040	206.35	206.35	0.0
LI13	32931	206.08	206.13	0.05
LI12	33581	205.80	205.91	0.11
LI11	34256	205.27	205.47	0.20
LI10	35001	204.57	204.62	0.05
LI9.2	35220	204.44	204.44	0.0
LI9.1	35407	204.35	204.33	-0.02
LI07	37322	203.26	203.30	0.04
LI06	38078	202.91	202.96	0.05
LI05	38943	202.39	202.48	0.09
LI03	40417	201.51	201.57	0.06

The implications of the modelling results are discussed in Section 8.

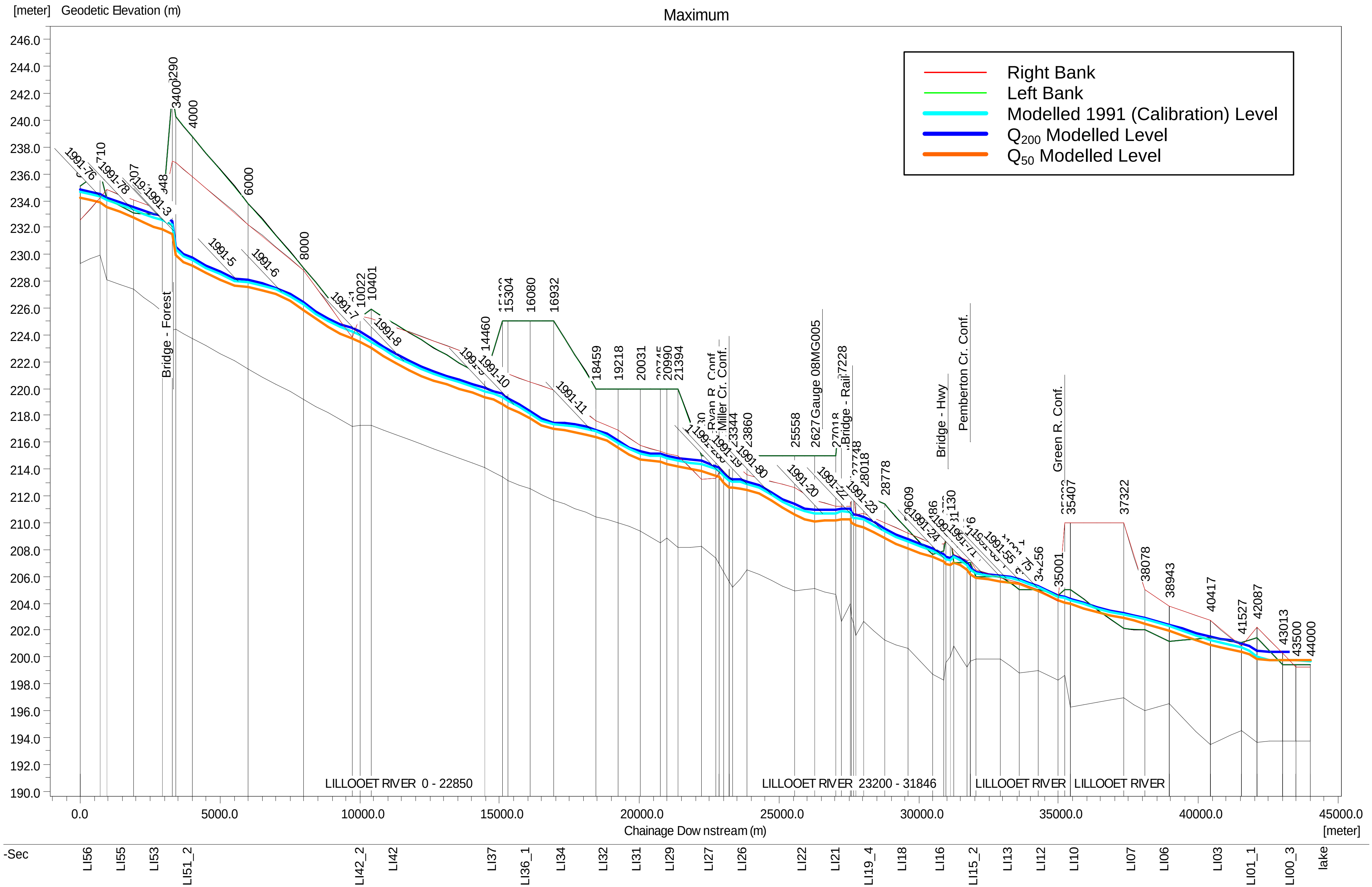
DYKING SCENARIO COST ESTIMATES

Class ‘D’ cost estimates for the two dyking scenarios discussed above are included in Appendix K. Table 7-4 summarizes these costs (refer to the appendix for a detailed breakdown of the costs):

Table 7-4
Summary of Class ‘D’ Cost Estimates for Dyking Scenarios

	Item	Capital	Engineering	Total Cost
1.	Scenario 1 Lowest Cost Option (**with** flood box & pumps)	$6,187,000	$640,000	**$6,827,000**
2.	Scenario 2 Lowest Cost Option (**with** flood box & pumps)	$8,087,000	$795,000	**$8,882,000**
1a.	Scenario 1 Lowest Cost Option (**without** pumps)	$5,687,000	$565,000	**$6,252,000**
2a.	Scenario 2 Lowest Cost Option (**without** pumps)	$7,587,000	$720,000	**$8,307,000**
	Approximate Flood Box Premium	$200,000	$30,000	**$230,000**
	Approximate Pump Station Premium	$500,000	$75,000	**$575,000**
Notes:				
1. Does not include GST.				

Lillooet River Modelled Flood Profile – Complete Profile



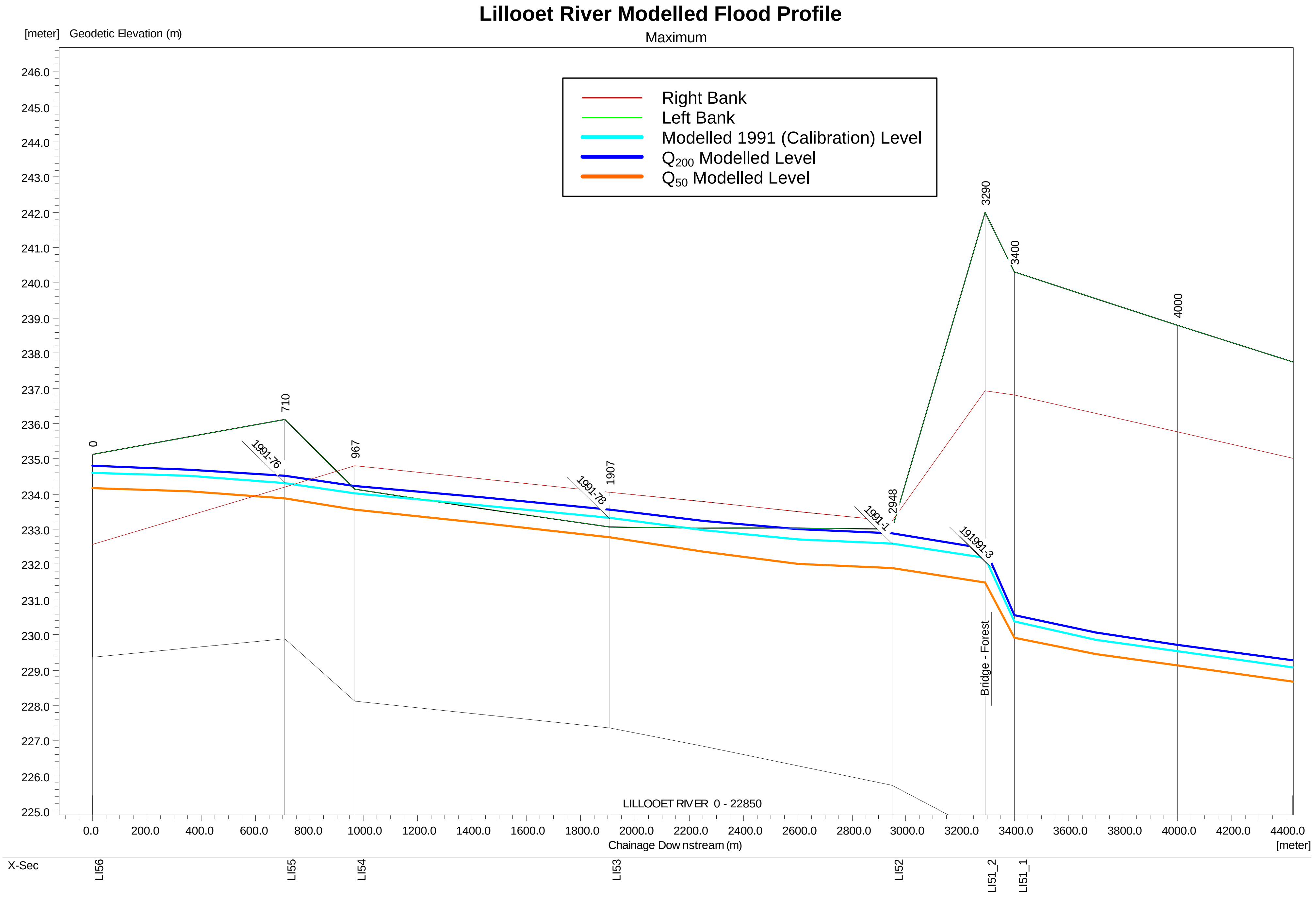


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Figure 7-4b

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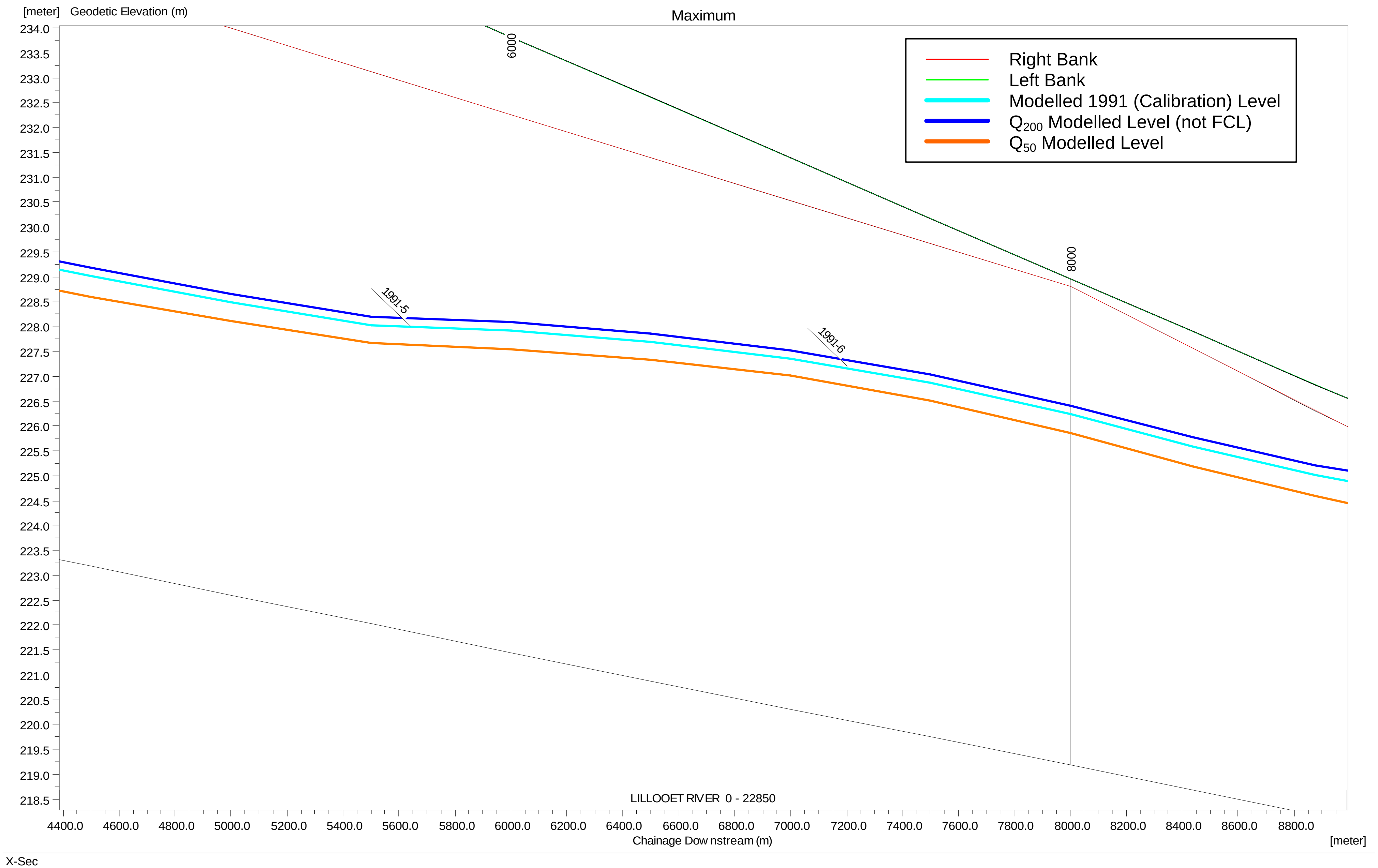
Lillooet River Modelled Flood Profile



Lillooet River Modelled Flood Profile

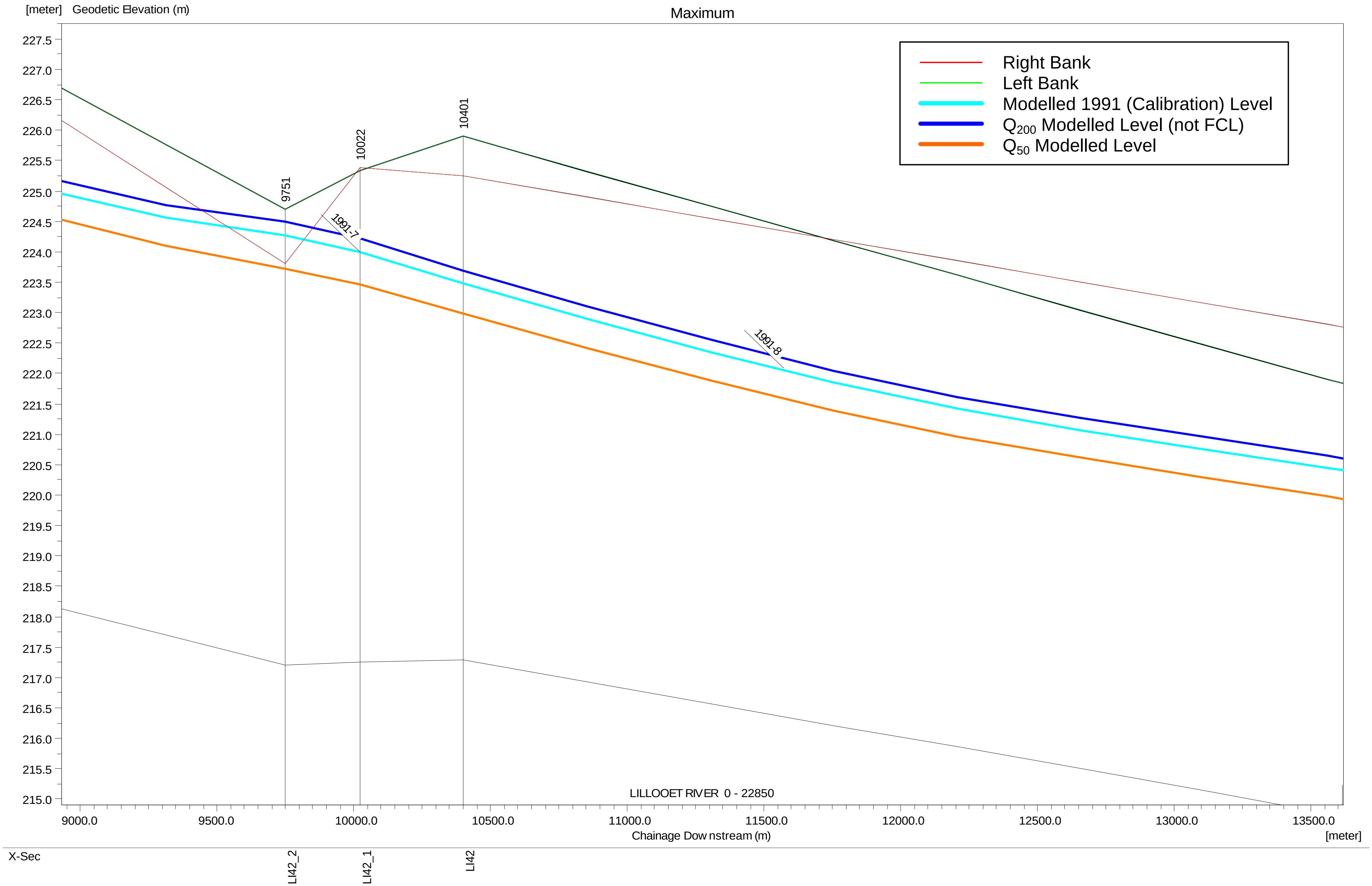


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Figure 7-4d

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Lillooet River Modelled Flood Profile

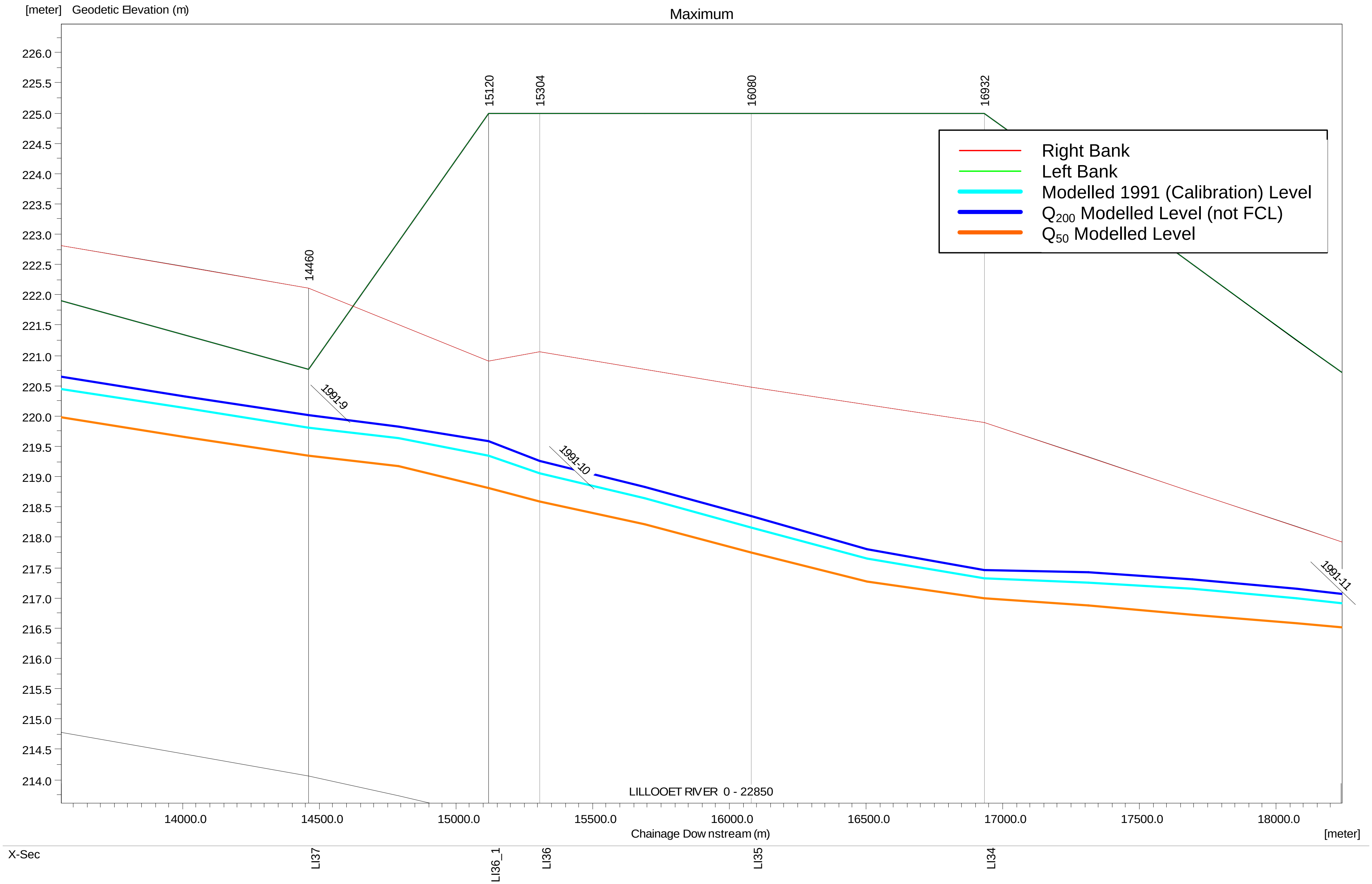

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Figure 7-4e

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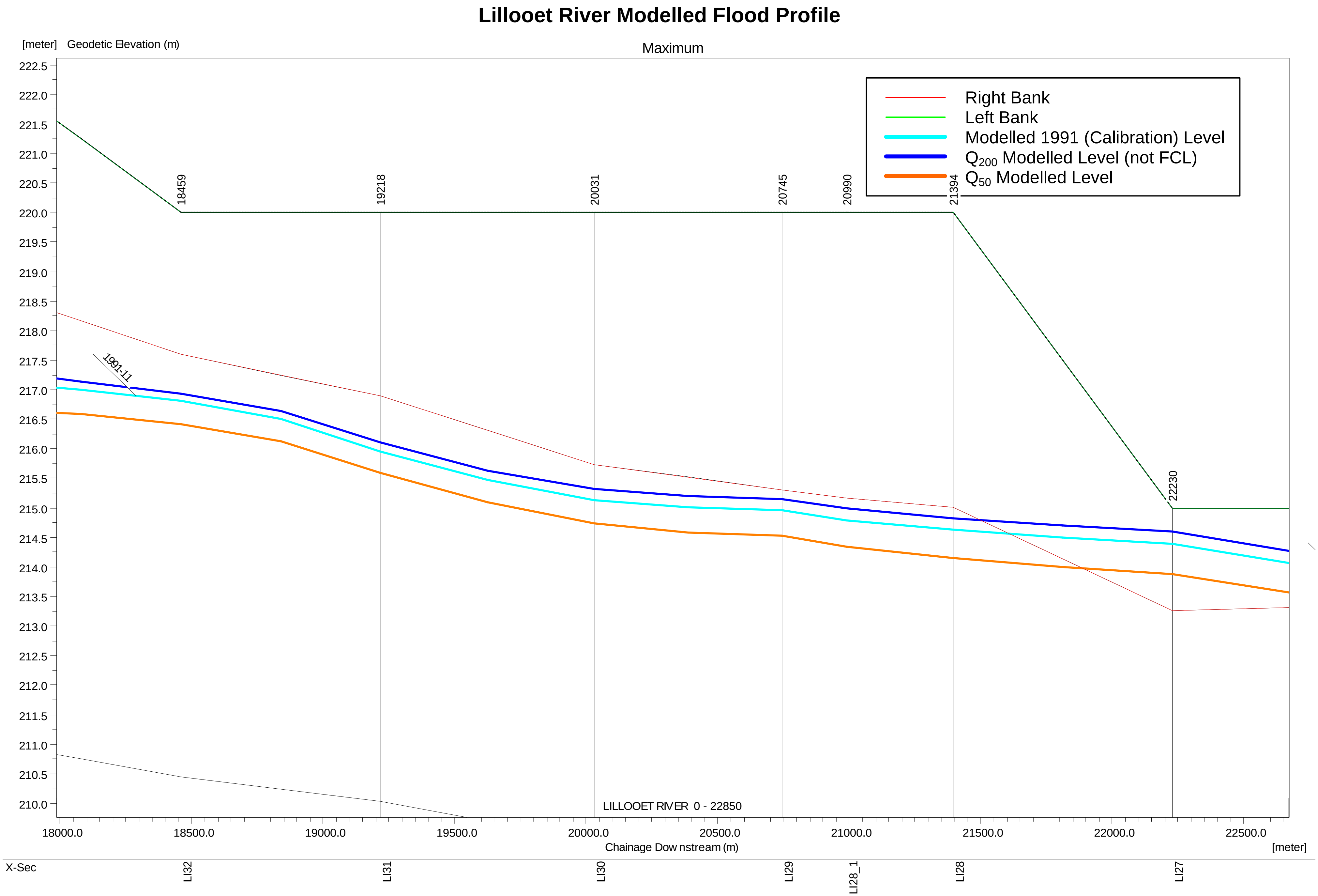

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Figure 7-4f

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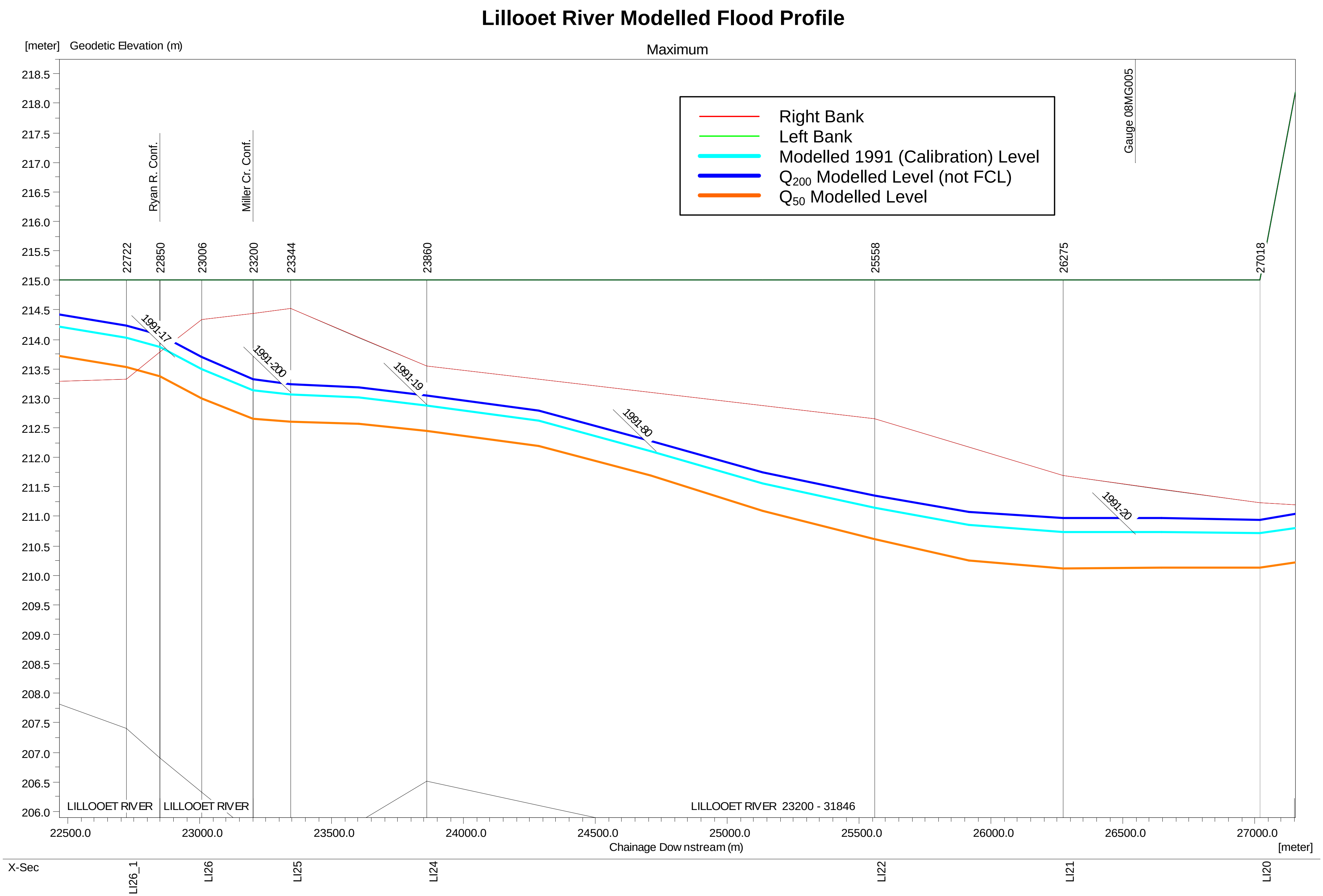

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Figure 7-4g

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Lillooet River Modelled Flood Profile

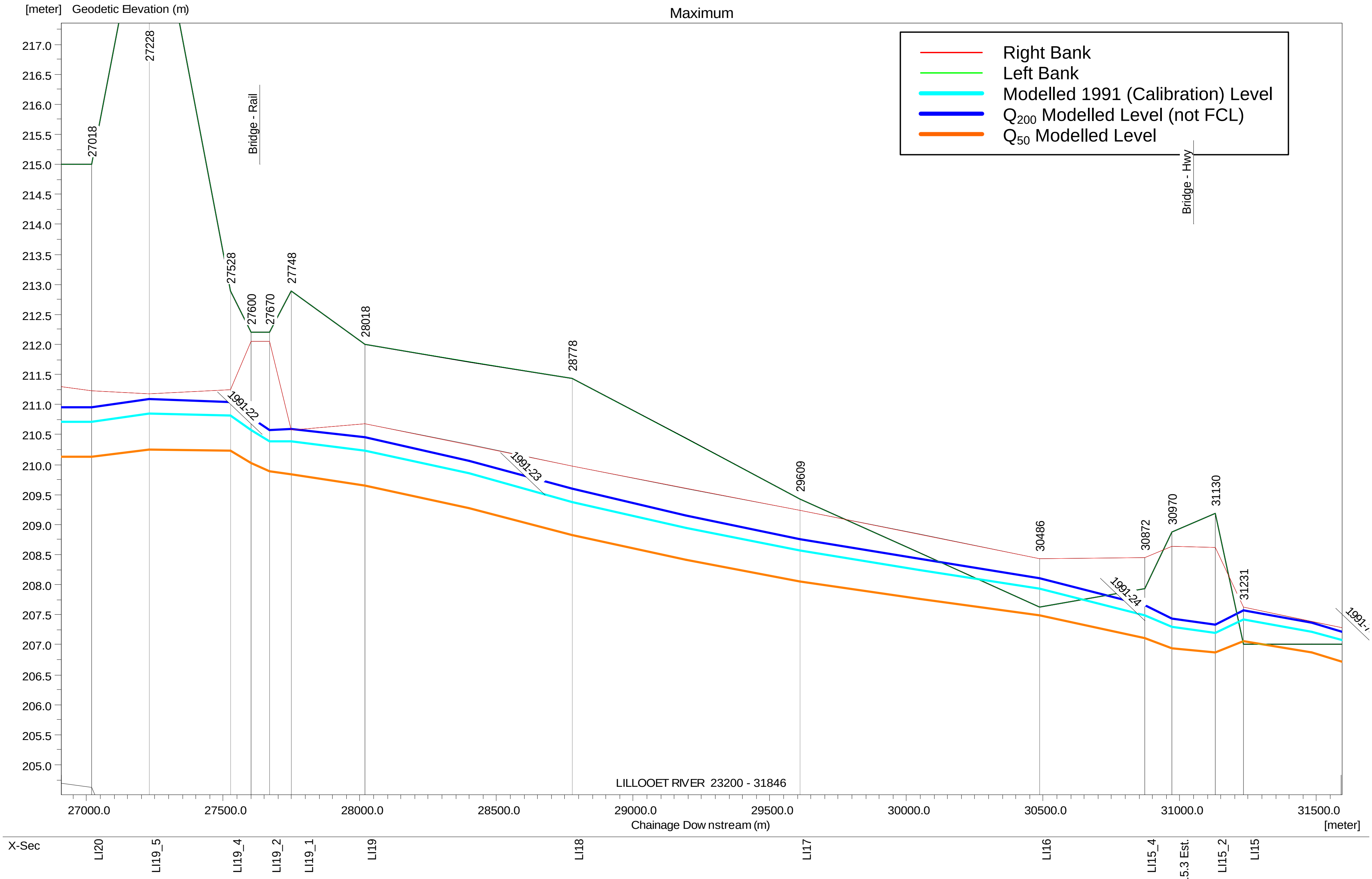

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Figure 7-4h

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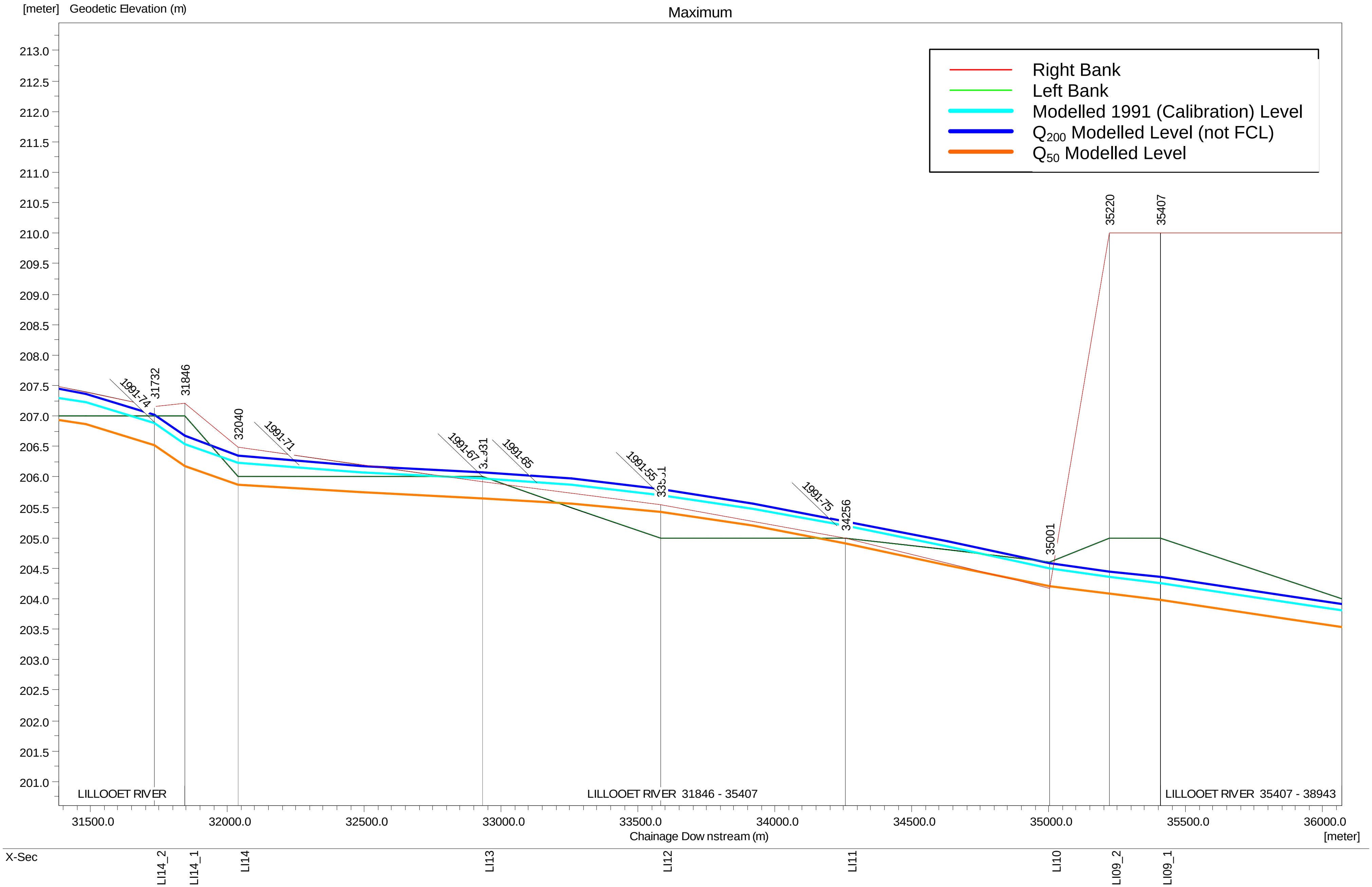
Lillooet River Modelled Flood Profile



Lillooet River Modelled Flood Profile

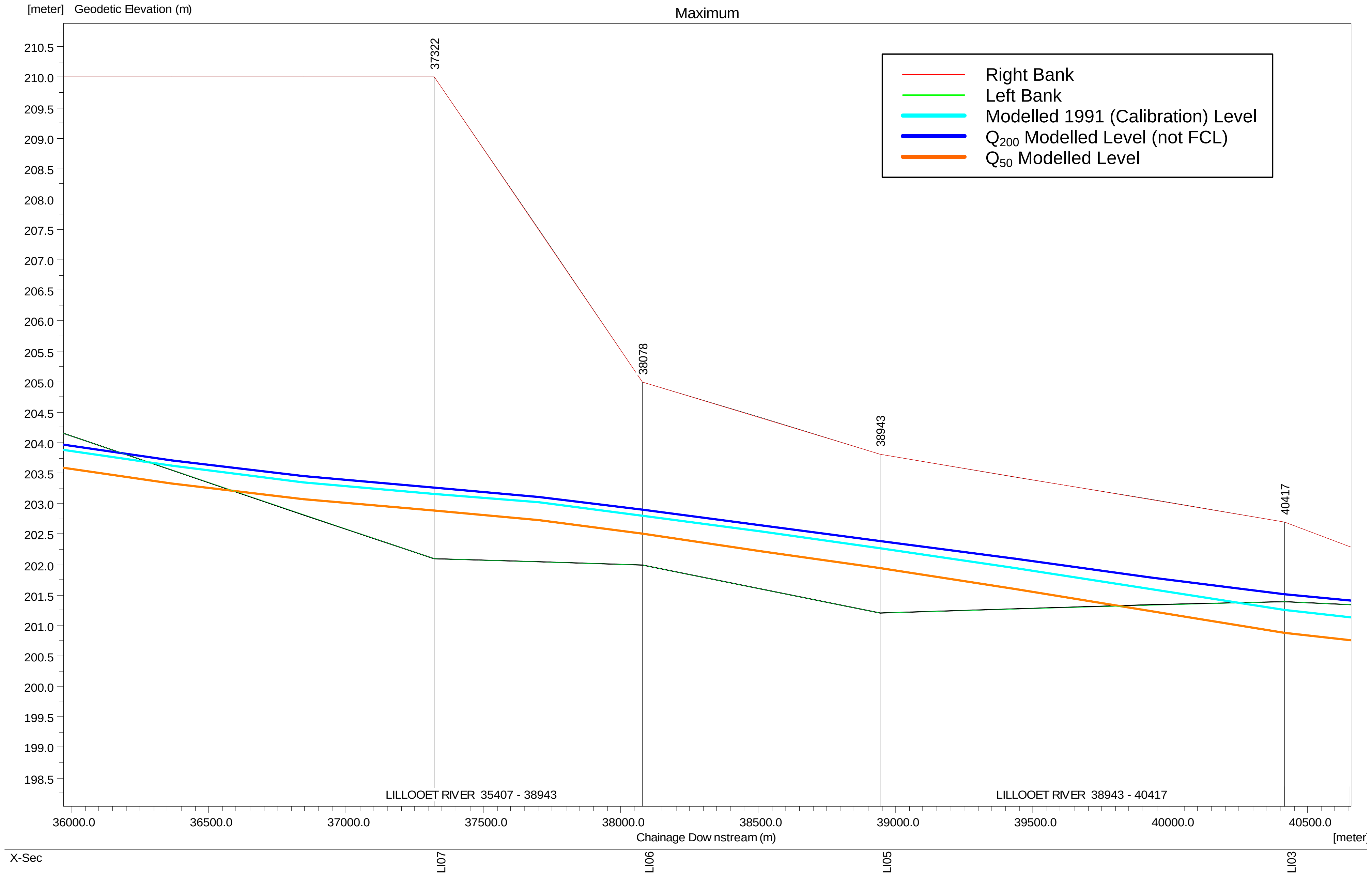


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Figure 7-4j

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Lillooet River Modelled Flood Profile

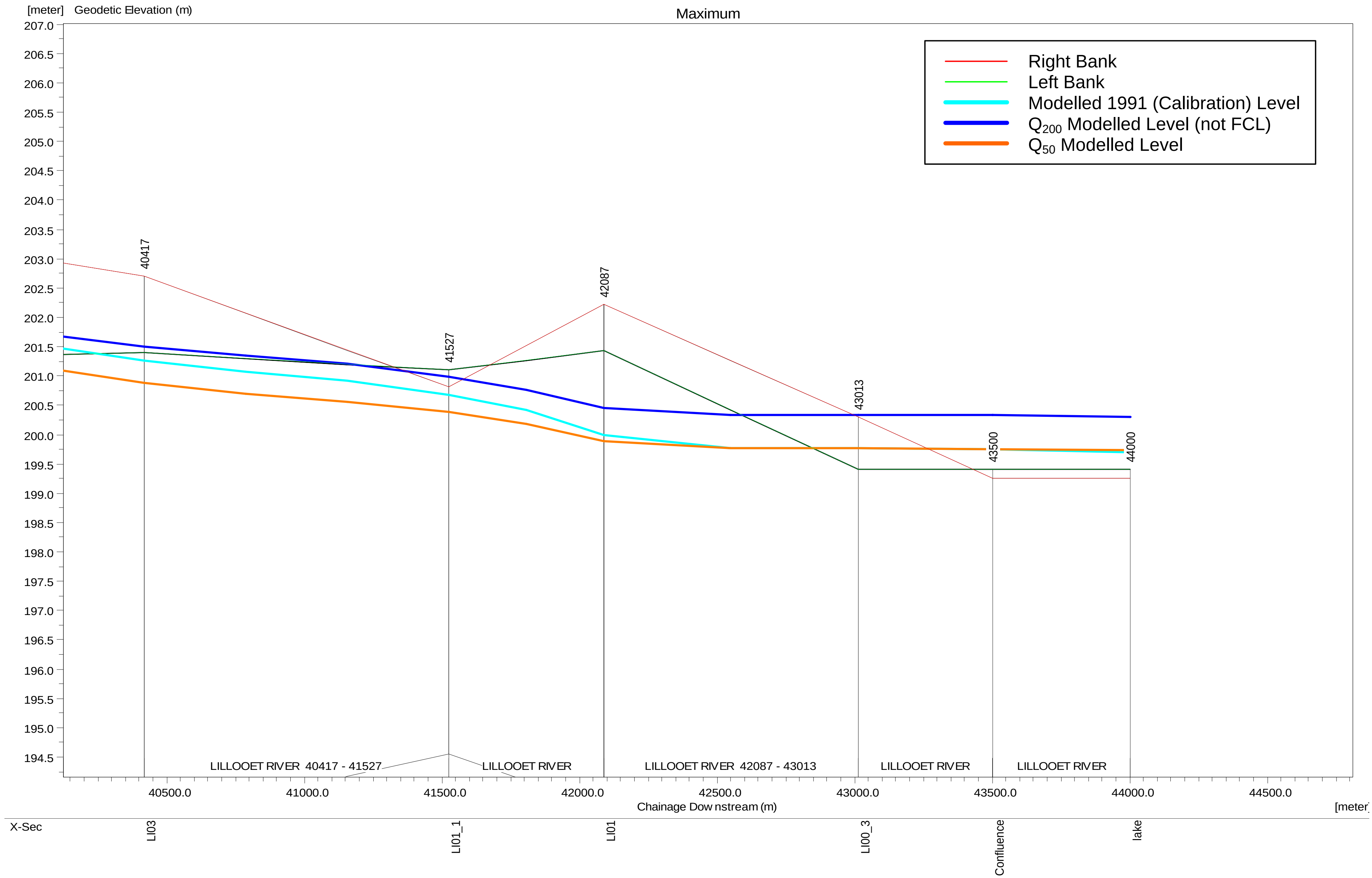

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Figure 7-4k

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Birkenhead River Modelled Flood Profile

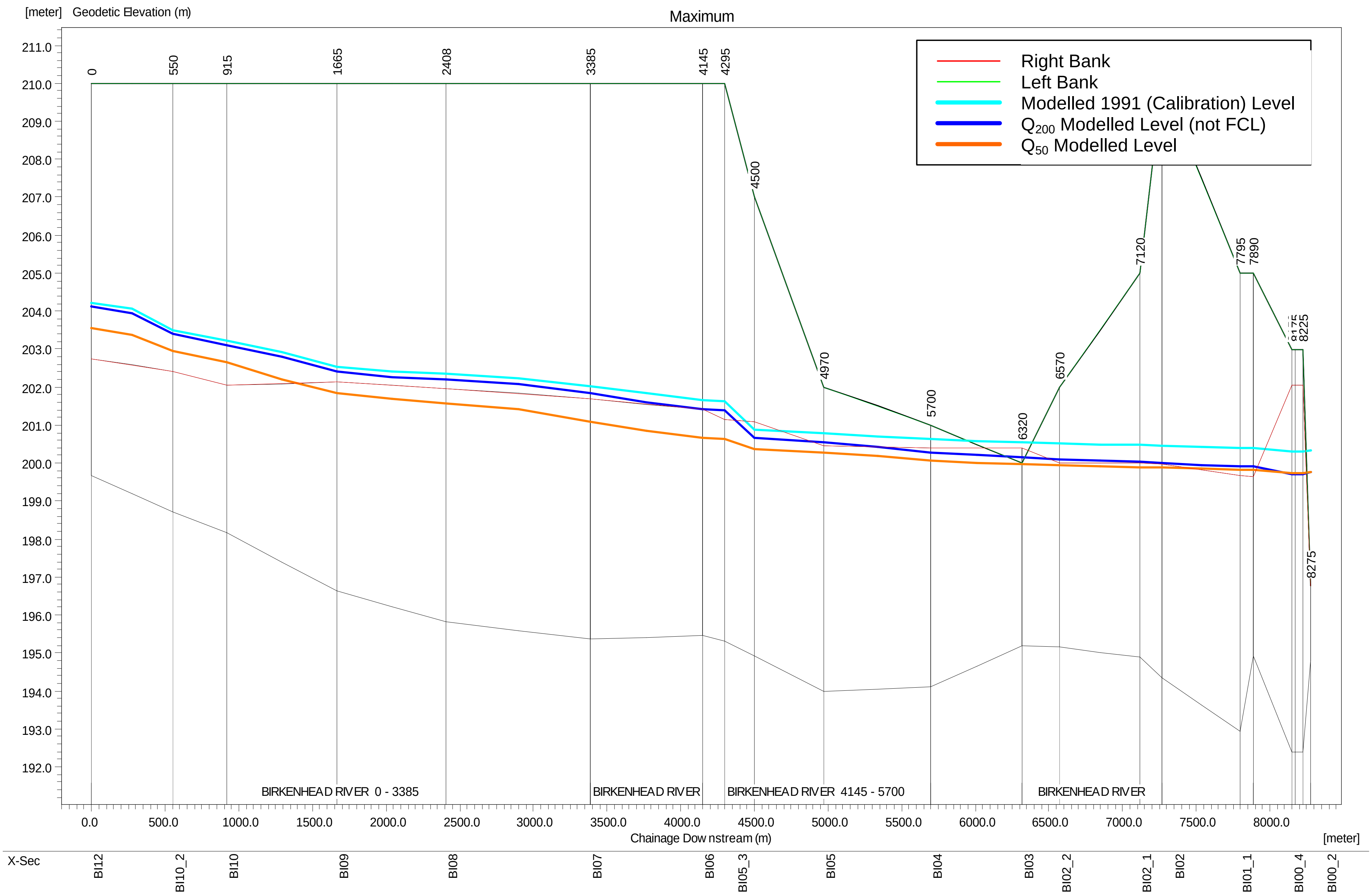

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Figure 7-5

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Green River Modelled Flood Profile

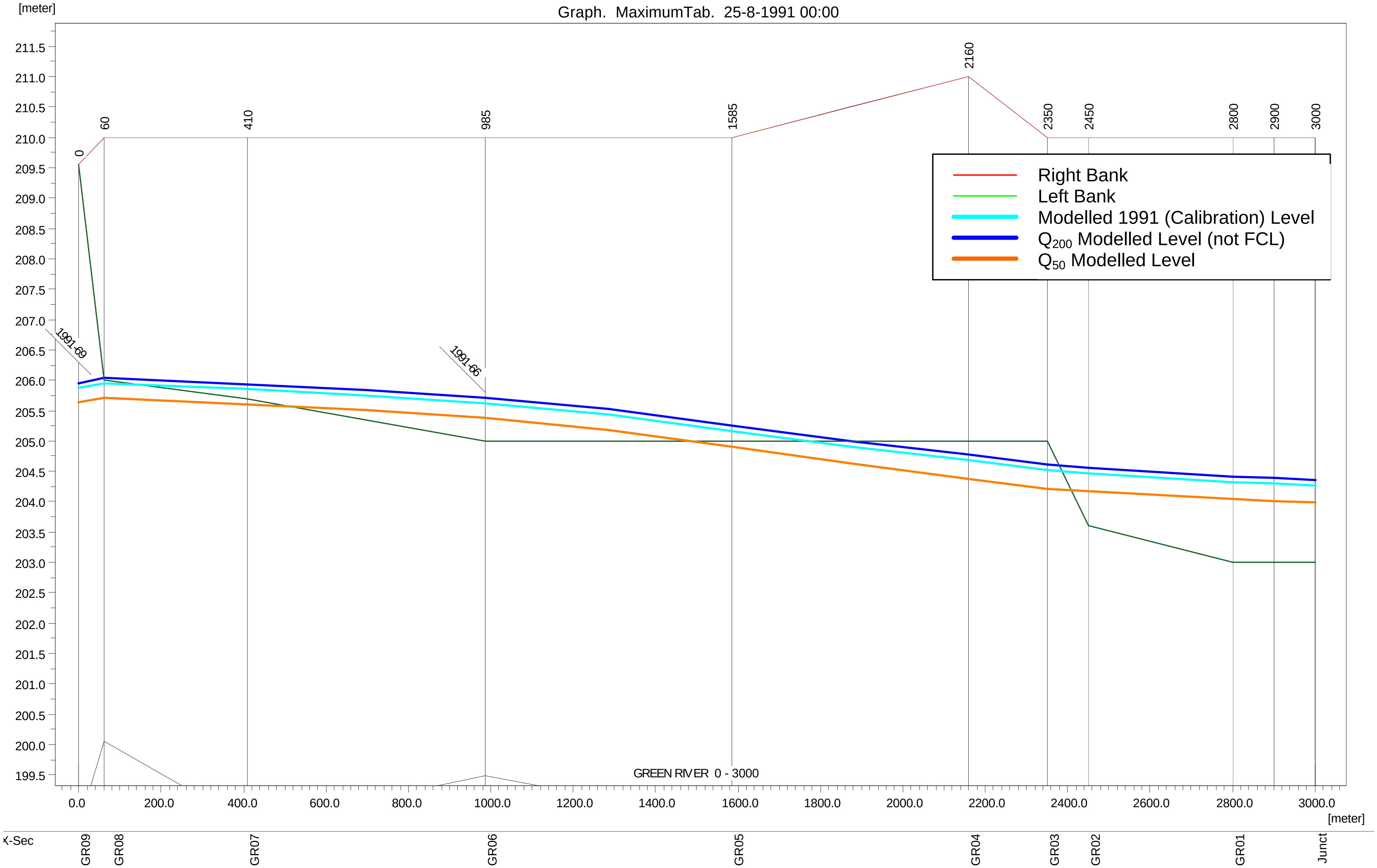

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Figure 7-6

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Ryan River Modelled Flood Profile

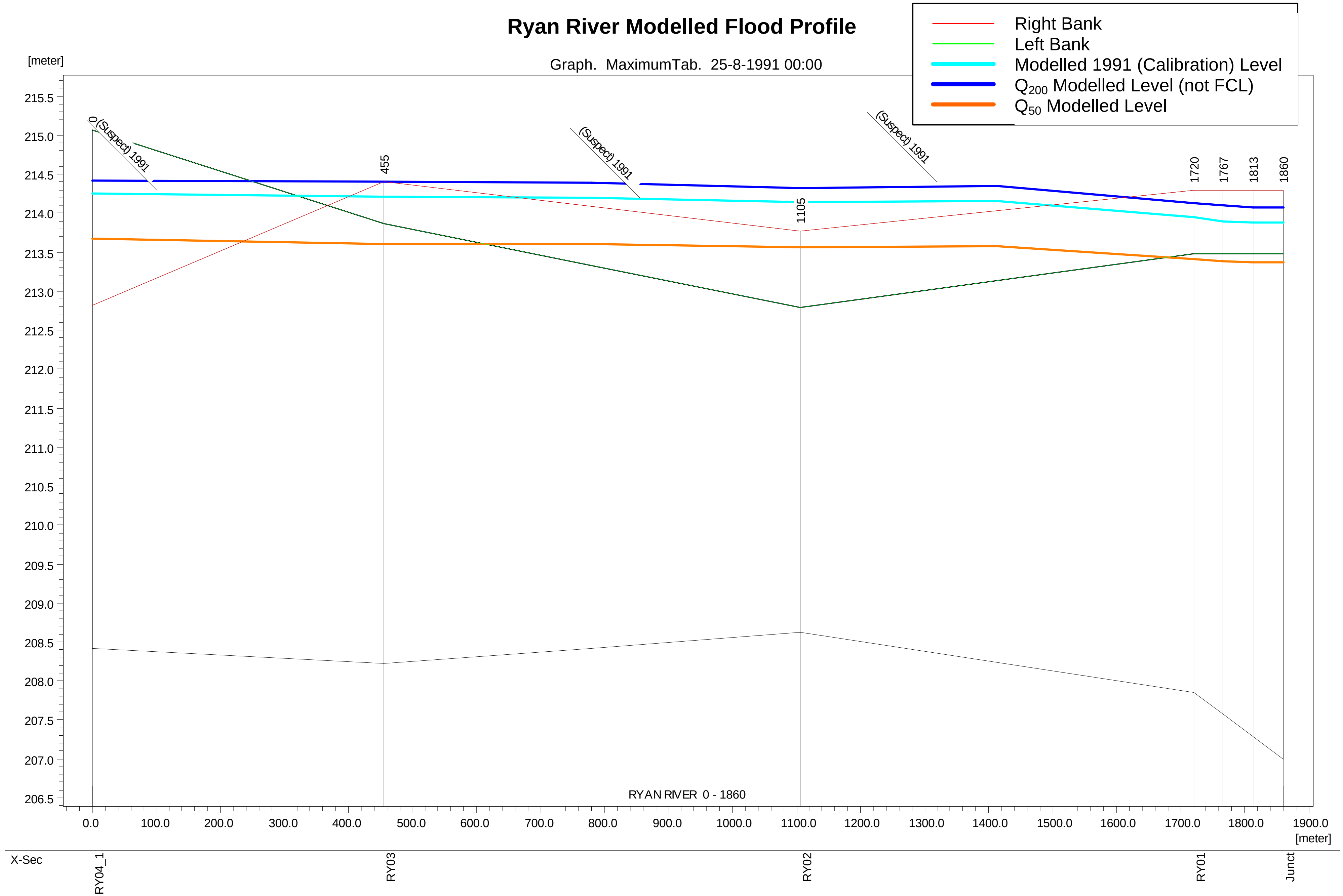

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Figure 7-7

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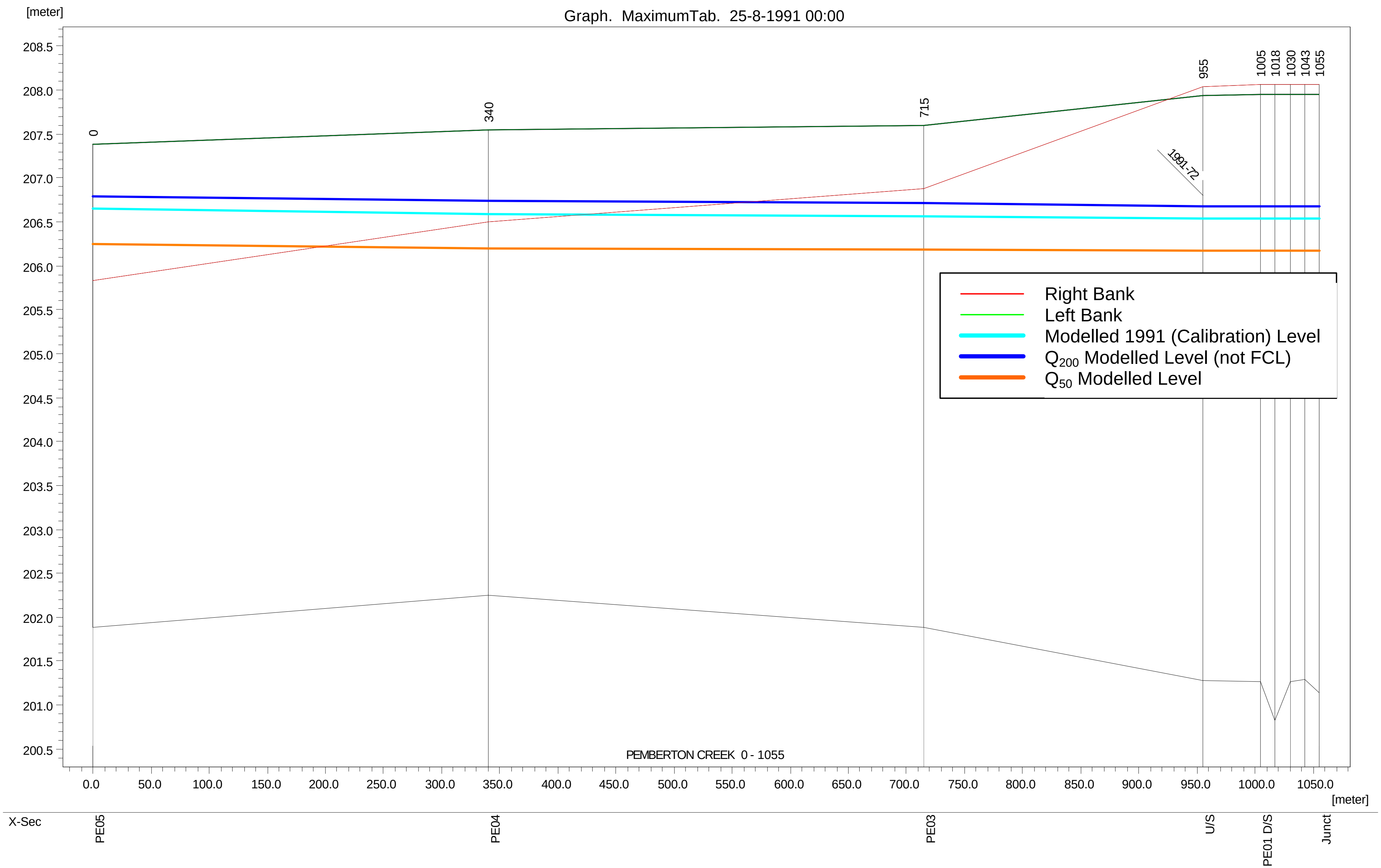
Pemberton Creek Modelled Flood Profile



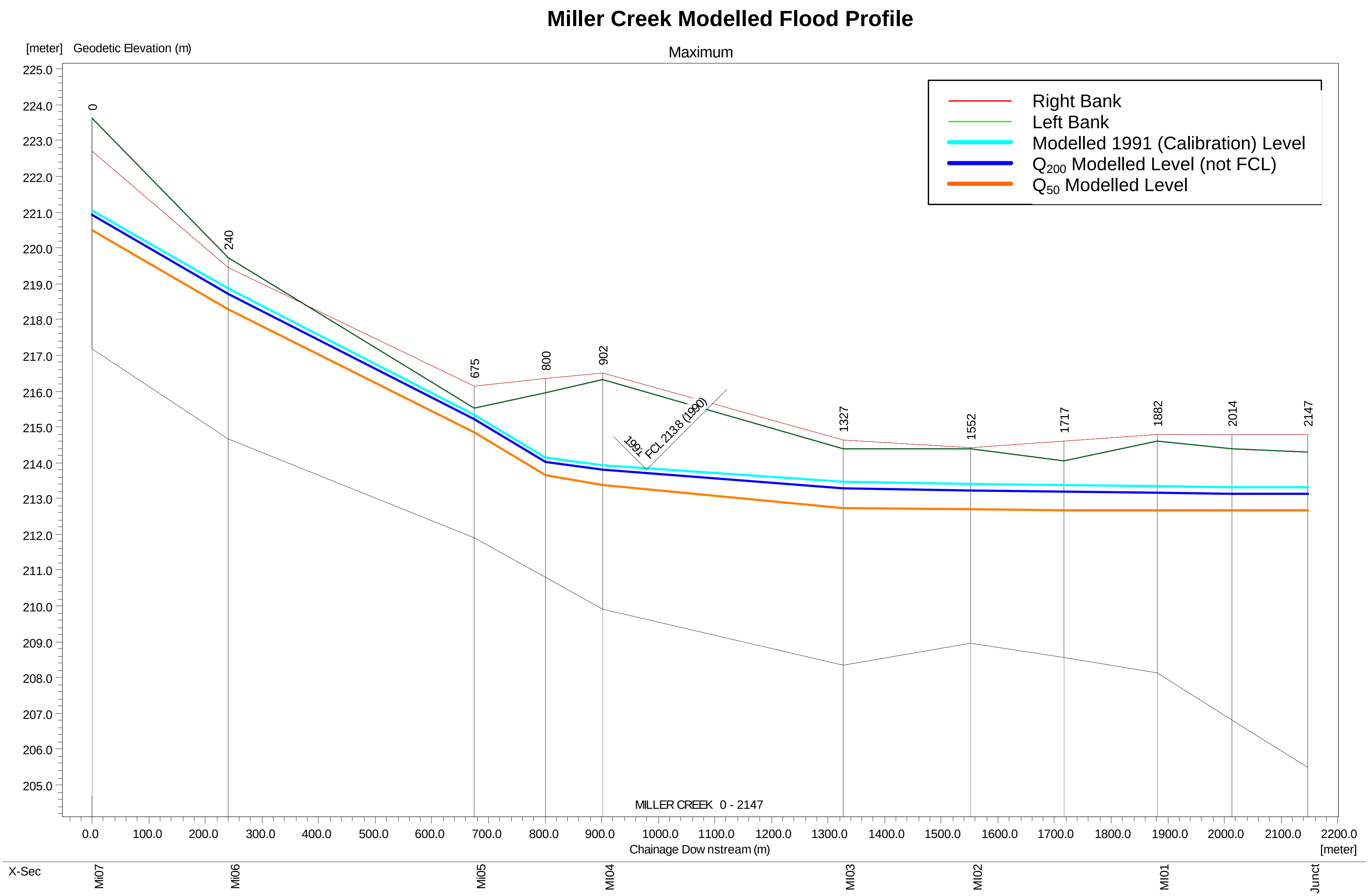


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Figure 7-9

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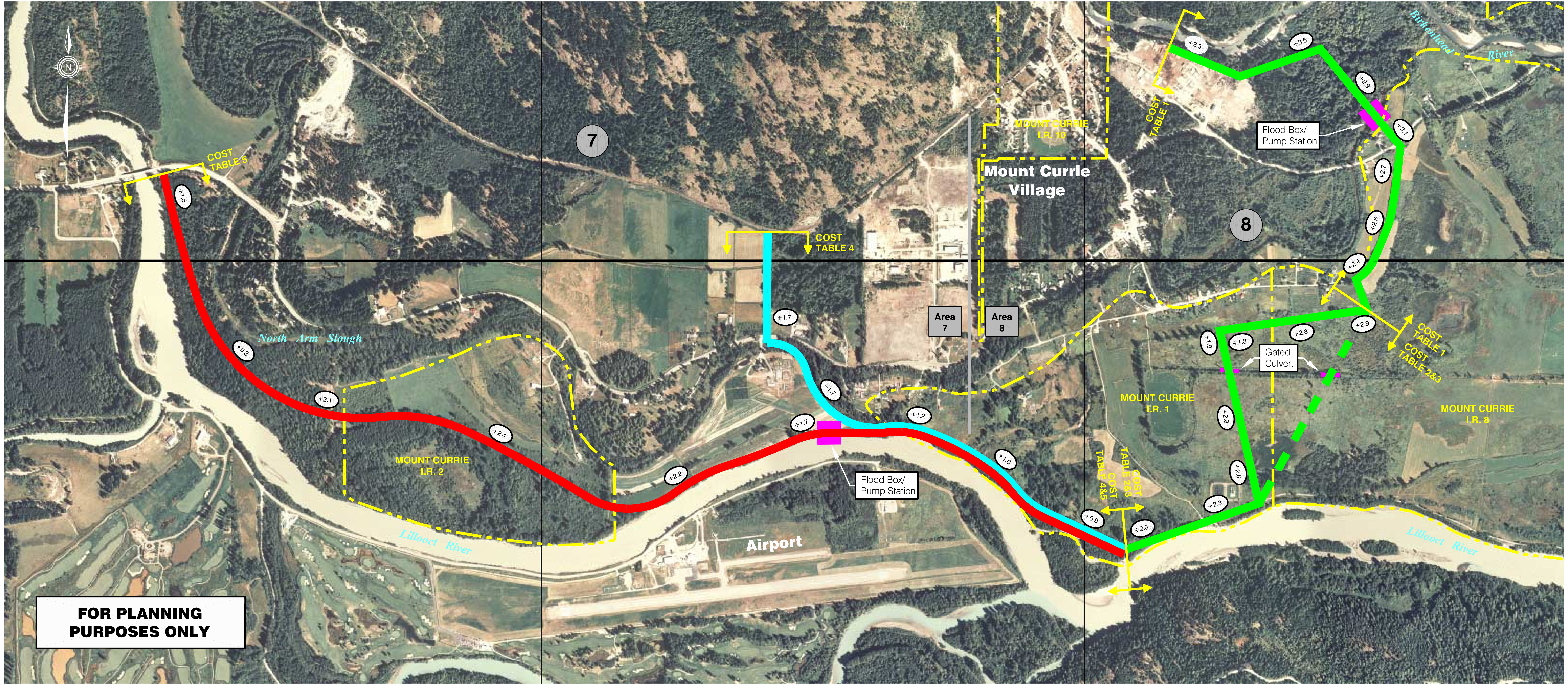

Engineering Study for Lillooet River Corridor

LEGEND

- Dyke Scenario 1
- Dyke Scenario 2
- Dyke Common Scenario 1&2
- +3.1 Typical Dyke Height Above Ground
- Area 8 Area - See Report Reference Below
- Reserve Boundary

For Planning Purposes Only

NOTE:

Alignment of dyke scenario 1 taken from 1985 drawing No. 85-13-9. sheet 4 of 5, which accompanies the report "Pemberton Valley Flood Protection 1985 Study" by H.H. Nesbitt-Porter, Ministry of Environment, Water Management Branch, File P72-3.

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250 0 250
Scale in Metres

Project No.	Date
713-002	December 2002

Planning Exercise: Proposed Dyking Scenarios for Areas 7 and 8

Figure 7-10

Dec.11/02

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Section 8

Management Implications

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8. MANAGEMENT IMPLICATIONS

This section discusses the management implications of the geomorphology and flood modelling work undertaken in this study. This includes the following:

- comment on the existing level of flood protection;
- generalized review of the need for dyke upgrading;
- identification of river management considerations; and
- identification of land use planning considerations.

8.1 FLOOD PROTECTION ISSUES

The 1985 Water Management Branch (Nesbitt-Porter) study defined areas for the Pemberton Valley which provide an effective basis for reviewing site-specific issues. The areas are defined numerically commencing from the upstream end of the study area, and are summarized in Table 8-1. The areas are also indicated on the map figures in Appendix B.

Table 8-1
Definition of Pemberton Valley Sub-Areas

Area No.	Description
1	Outdoor School Farm Area
2	Salmon Slough to Ryan River
3	Ryan River to Miller Creek
4	Miller Creek to Pemberton Creek
5	Pemberton Creek to Green River (Excluding Airport)
6	Airport Area
7	North Arm Plug to Mount Currie I.R. No. 1
8	Mount Currie I.R. No. 1 to Lillooet Lake

In view of the modelling results of Section 7, the existing level of flood protection in each of these areas is discussed below, with reference to defined sub-areas where appropriate. Identified flood protection issues are also noted for each area. Adequacy of dykes is indicated on the map figures in Appendix B through colour coding.

AREA 1 – OUTDOOR SCHOOL FARM AREA

Area 1 is adjacent to the left bank of Lillooet River above the forestry bridge. It comprises an agricultural area which is partially dyked at present. Significant further dyking would be required to protect this area to the 200-year return period standard, as the dyke would be overtopped by approximately 0.8 m during a 200-year return period event (excluding freeboard). In order to effectively cut off upstream overbank flows, such a dyke would have to tie into high ground at the upstream end.

Some development in this area is located on the Wolverine Creek fan, and protection from this source should also be considered. Drainage issues associated with Wolverine Creek and other minor surface water sources would need to be addressed in a dyking scheme.

Other issues for consideration in Area 1 are the need for further bank protection works and the stability of a former meander cut-off in this area.

AREA 2 – SALMON SLOUGH TO RYAN RIVER

Area 2 is a large, predominantly agricultural area lying between the right bank of Lillooet River and the left bank of Ryan River.

Lillooet River

This expansive 21 km long reach includes the McKenzie Cut. Throughout this reach, Lillooet River is generally confined by the valley sidewall on the left bank. Dyking is primarily, therefore, an issue on the right bank. Much of the area is bounded by the left bank of Ryan River which also presents a flood hazard.

The flood vulnerability of the right bank of Lillooet River in Area 2 during a 200-year return period event is summarized as follows:

- the upper 2 km (above the forestry bridge) has adequate capacity to pass the flow, but the amount of freeboard available is marginal to inadequate;
- the next 15 km is generally not subject to overtopping; and
- the downstream 2 km of this reach become progressively more vulnerable to flood overflows, to a depth of up to about 0.4 m (excluding freeboard). The most inundated area is the confluence of Ryan River and Lillooet River.

Other issues for consideration along this reach of Lillooet River are the need for further bank protection works, and the stability of the river channel (especially the artificial McKenzie Cut).

Ryan River

Despite previous dyking projects, the upper reach of the left bank of Ryan River remains potentially vulnerable to flood overflow in association with extreme debris flood events on Ryan River. Only the lower 2.1 km of Ryan River was modelled in this study and this reach was found to be vulnerable to flood overflow to a depth of 1 m to 2 m (excluding freeboard) during a 200-year return period event.

The concept of a partial diversion of some Ryan River flow to the Lillooet River has been raised in the past, and may warrant further consideration in the future. Quantification of the debris flood hazard would be an essential first step in determining an effective approach to hazard mitigation on Ryan River.

Other issues for consideration along this reach of Ryan River are bedload aggradation in general and bank erosion at many locations.

AREA 3 – RYAN RIVER TO MILLER CREEK

This small area is bounded on three sides by the right bank of Ryan River, a very short section of the right bank of Lillooet River, and the left bank of Miller Creek. Flood vulnerability during a 200-year return period event is summarized as follows:

- the area is very vulnerable to flooding from Ryan River, especially near the highway bridge where the flood overtopping depth is about 1.6 m (excluding freeboard);
- the Lillooet River right bank dyke appears to have at least 0.6 m freeboard; and
- the Miller Creek left bank dyke (Boneyard Dyke) has at least 0.6 m freeboard in the modelled area.

Ryan River was only modelled below the Pemberton Meadows Road bridge, so the flood vulnerability of the upstream reach has not been established.

Flood protection in Area 3 depends on effective upstream tie-ins of the dykes along the right bank of Ryan River. Several roads present complications for dyking that would need to be overcome.

As this area is located at low gradient reaches of both Ryan River and Miller Creek, bed aggradation is a significant issue. Miller Creek, in particular, has been subject to frequent bedload removal in this area to maintain channel capacity. Dyke slope and bank protection is also an issue.

AREA 4 – MILLER CREEK TO PEMBERTON CREEK

Area 4 is bounded by the right bank of Miller Creek, the right bank of Lillooet River and the left bank of Pemberton Creek. It includes the Village of Pemberton, but much of the development area is above the river floodplain.

Miller Creek

Miller Creek was modelled to almost the apex of the alluvial fan. This area was not found to be vulnerable to overtopping during a 200-year return period event, however the amount of freeboard available downstream of the road bride may not be fully sufficient.

Flood protection in this area depends on an effective upstream tie-in of the Miller Creek right bank dyke. As with Area 3, bed aggradation is also a significant issue, along with bank and dyke slope protection.

Lillooet River

The Lillooet River dyke in Area 4 is approximately 8.4 km in length. The left bank is generally confined by valley sidewalls. A dyke protecting Area 4 is located along the right bank.

FPAF-funded work in 2002 raised approximately 2,700 m of dyke in Area 4 on the right bank of the Lillooet River. The modelling results show that the dyke along the right bank of Lillooet River is high enough to prevent overflow of the 200-year return period flood event.

Bank and dyke slope protection is an issue at several locations in this reach. At the BC Rail bridge crossing, a site-specific issue is the hydraulic capacity of the bridge and upstream channel stability. The railway bridge has been observed to cause a flow obstruction during some previous flood events. The mount of freeboard available at the bridge is approximately 0.6 m, and this is not adequate to pass floating log debris.

Pemberton Creek

Only the lower 1 km of Pemberton Creek was modelled, and the left bank dyke (Pemberton Creek Dyke) in this reach adjacent to Area 4 appears to be high enough to withstand the 200-year return period flood event with an adequate amount of freeboard.

The flood vulnerability of the upstream reach (on the Pemberton Creek fan) has not been established. Bed aggradation is an issue on lower Pemberton Creek, along with bank and dyke slope protection.

AREA 5 – PEMBERTON CREEK TO GREEN RIVER (EXCLUDING AIRPORT)

Area 5 is bounded by the right bank of Pemberton Creek, the right bank of Lillooet River and the left bank of Green River. It includes primarily agricultural and golf course developments. An arbitrary line at the west end of the airport forms the boundary between Area 5 and Area 6. There is the possibility of integrating flood protection for these two areas.

Pemberton Creek

Only the lower 1 km of Pemberton Creek was modelled, and the right bank dyke (Private Dyke) in this reach is vulnerable to flood overflows to a depth of up to about 0.3 m during a 200-year return period flood event (excluding freeboard). The flood vulnerability of the upstream reach (on the Pemberton Creek fan) has not been established, but is considered to be much less of an issue for Area 5 than for Area 4.

As with Area 4, bed aggradation is an issue on lower Pemberton Creek, along with bank and dyke slope protection.

Lillooet River

The airport access road acts as a dyke along the right bank of Lillooet River in this reach. The flood modelling shows that the road surface is up to 0.3 m below the 200-year return period flood level (excluding freeboard).

The need for upgrading the paved road along Lillooet River into a standard dyke should be considered as part of an overall dyking strategy for Areas 5 and 6. Bed aggradation and bank protection are particular issues in this area.

Green River

Green River was not modelled in this reach, so the vulnerability to flooding has not been established. Some private dykes are located in this reach on the Big Sky golf course and adjacent agricultural lands.

Green River is very active in this reach, with bed aggradation, bank erosion, and channel shifting. Further dyking on the Area 5 reach of Green River should be considered as part of an overall dyking strategy for Areas 5 and 6.

AREA 6 – AIRPORT AREA

Area 6 is bounded by the right bank of the Lillooet River and the left bank of the Green River. It includes the Pemberton Airport. Although this area is contiguous with Area 5, flood protection can be considered in isolation or in combination.

Lillooet River

The airport access road acts as a dyke along the right bank of Lillooet River through some of this reach. The flood modelling shows that the road surface is up to 0.3 m below the 200-year return period flood level (excluding freeboard).

As with Area 5, bed aggradation and bank protection are particular issues in this area. The need for dyking along this reach of Lillooet River should be considered as part of an overall dyking strategy (a setback dyke would be preferable) for Areas 5 and 6. The impact of further dyking and bank protection works in this area need to be carefully considered in view of potential erosion impacts across the river at Mount Currie I.R. No. 2.

Green River

Green River is not formally dyked through this area, and remains a flood threat to the airport. Construction of a setback dyke has been previously proposed to protect the airport, and should only be considered as part of an overall dyking strategy for Areas 5 and 6.

AREA 7 – NORTH ARM PLUG TO MOUNT CURRIE I.R. NO. 1

Area 7 is along the left bank of Lillooet River. At the upstream end of the reach, the inlet to the North Arm is closed off with a plug dyke between the upstream end of Area 7 and the highway bridge. Mount Currie I.R. No. 2 is located in this area.

Flood vulnerability during a 200-year return period event is summarized as follows:

- the North Arm plug dyke is subject to overtopping by up to 0.5 m (excluding freeboard); and
- the left river bank below the North Arm plug dyke is subject to overtopping by up to 0.8 m (excluding freeboard).

In addition to dyke crest elevation at the North Arm plug, bank protection is also an issue. The environmental and floodplain management implications of further reinforcing the North Arm plug should be considered before proceeding in this direction. The need for dyking and bank protection in the lower part of the reach also needs to be considered. A setback dyke on Mount Currie I.R. No. 2 was previously proposed.

AREA 8 – MOUNT CURRIE I.R. NO. 1 TO LILLOOET LAKE

Area 8 comprises predominantly Band lands between the left bank of Lillooet River and the right bank of Birkenhead River. It includes Mount Currie I.R. No. 1, Mount Currie I.R. No. 8, Mount Currie I.R. No. 10 and Nesuch I.R. No. 3.

Lillooet River

The left bank of Lillooet River in this reach extends for a length of approximately 9 km. A road dyke adjacent to I.R. No. 1 provides partial flood protection in this area, but it is subject to flood overflow by up to 0.3 m (excluding freeboard) during a 200-year event. Downstream of this dyke, flooding increases to well above 1 m, and gets progressively in the downstream direction.

There is a need to develop a comprehensive dyking plan for this area prior to proceeding with individual projects in a piecemeal fashion.

There are a number of sloughs crossing the floodplain in this area, and the environmental and floodplain management implications of further slough cut-offs should be carefully considered before proceeding in this direction.

Bed aggradation and bank protection is an issue through this reach.

Birkenhead River

The length of Birkenhead River through Area 8 is approximately 11 km, mostly on the Birkenhead River fan.

The right bank is partially dyked on I.R. No. 10, although some deficient areas remain. The vulnerability of the right bank to flood overflows during a 200-year return period event is highly variable, ranging from adequate in some locations to almost 2 m too low (excluding freeboard) in other locations.

A comprehensive dyking plan should be prepared in conjunction with the Lillooet River in Area 8.

Bed aggradation and bank protection are issues through this reach.

8.2 NEED FOR DYKE UPGRADING

DESIGN CRITERIA

The review of flood protection issues in Section 8.1 is based on design flood levels (which exclude freeboard) generated by Mike 11 modelling using the 200-year return period peak instantaneous flow. Where the peak instantaneous flow is used, it is customary to apply a freeboard allowance of 0.3 m. If the peak daily flow were to be used, a freeboard allowance of 0.6 m would normally apply. The former condition appears to govern throughout the Lillooet River corridor.

In consideration of sediment allowance and unknown climate change influences, an additional 0.3 m of freeboard has been requested by MWLAP for application below Ryan River(personal communication: Mr. Neil Peters, November 21, 2002). The additional freeboard is to apply to Lillooet River downstream of Ryan River. Refer to Appendix L for details.

In some areas, dyking to a lower standard than the 200-year return period standard may be considered sufficient. In the past, an ‘agricultural’ dyke standard has been used in some areas based on a 50-year return period.

The topic of freeboard is discussed further in Subsection 10.2

PEMBERTON VALLEY DYKING DISTRICT

For the areas of PVDD responsibility (Areas 1 through 7), the primary issue is the standard of the existing dykes. A prioritized plan for dyke upgrading would be appropriate in Areas 1, 2, 3 and 4. The primary objectives would be to upgrade existing dykes through dyke raising, dyke slope protection and bank protection.

In Areas 5, 6 and 7, should improved flood protection be desired, an updated concept plan for dyke upgrading should be prepared prior to significant dyke upgrading activities being undertaken. Significant dyke upgrading or construction of new dykes should be subject to the river management considerations noted in Section 8.3.

MOUNT CURRIE BAND

The dyking system for the Mount Currie reserves is less developed than that for the PVDD. A comprehensive dyke upgrading plan should be prepared as the next step in providing a higher level of flood protection. The river management considerations noted in Section 8.3 would be applicable in preparing a dyke upgrading plan.

8.3 RIVER MANAGEMENT CONSIDERATIONS

LILLOOET RIVER DELTA AND LILLOOET LAKE

The question has been raised whether dredging of the lower river would result in a reduction in the flood profile. While there are flood protection benefits to such an option, the experience from dredging in the late 1940s indicates that any benefits are short-lived.

Two factors are against dredging the lower river. First, if sand is dredged below the Green River confluence, it will tend to redeposit since a wedge of gravel accumulating upstream of the confluence controls the lower bed elevation (only minor amounts of gravel move beyond this point so the gravel wedge acts as a sill). Second, the slope of the river is very gentle and during a flood, the lake level is sufficiently high that it backwaters a considerable distance upstream.

It has also been postulated that lowering the lake level could reduce the flood profile of the lower river. Again, the benefits would be short-lived since reducing the base level of the lake would have only a minor impact on the base elevation of the gravel wedge. While the immediate response would be degradation in the lower river, it would refill with fine sediment as noted above.

The Nesbitt-Porter report (1985, pp 19-22) notes that negligible benefits are expected from further dredging of Lillooet Narrows as a means of flood level reduction. The report notes that the benefits of reducing the lake level by 1.3 m during a Q_{200} flood diminish by Lillooet River cross section 4 (approximately 850 m downstream of the south end of Seymour Road). Similarly, the benefits of reducing the lake level by 3.0 m diminish by cross section 11 (adjacent to the downstream end of the airport runway).

While benefits would likely result for some of the Mount Currie Band lands, the monetary and environmental costs of dredging the lower river and/or lowering the lake are likely prohibitive.

ROADS, BRIDGES AND LINEAR FILLS

The Lillooet River channel and floodplain are crossed by many roads, bridges and linear fills. These need to be carefully considered in future dyking activities.

Roads adjacent to river channels limit dyke options in several locations. Where feasible and justifiable, road relocation should be considered in a long term context.

Bridge crossings of the Lillooet River are variable in standard. Bridge standards and the implication to dyking and flood protection should be discussed with the bridge owners. A management plan should be put in place where there are significant concerns.

Linear fills on the floodplain, most often associated with roads and railways, affect flood overflow routing in several floodplain areas. These have not been an issue for the river modelling for this study because of the assumed confinement of the river by the dykes in many places, and the severe nature of the flooding considered. Floodplain inundation and routing have not been modelled in detail.

RIVER REGIME

The Lillooet River has been the subject of many alterations over the past century. The river has adjusted to some of these changes, and continues to adjust to others. The primary result of the changes is a straightened, narrowly confined river channel. In view of the dynamic watershed geomorphology, the Lillooet River is expected to continue to be very active in bedload movement, debris movement and channel shifting in the future.

Over the next century, it may be appropriate to identify opportunities to allow the river to revert to a more natural condition with a wider river corridor. This would involve consideration of restoring meanders, reactivation of side channels, dyke relocation and possible property acquisition. A wider river corridor would have the benefit of reduced flood level, lower dyke height, and a lessened flood hazard.

Consideration should also be given to a systematic program of gravel removal that approaches the yearly influx of gravel to lower reaches (less than 10,000 m^3/yr). If gravel is allowed to accumulate over a number of years, the level of protection provided by the dykes will be gradually compromised. Gravel removal would need to be systemically monitored and should incorporate habitat features where possible. Such a program is currently in place for gravel removal on Fraser River, and could be used as a template for a similar program on Lillooet River.

The important hydraulic and environmental value of past and current river side channels needs to be carefully considered as future dyking decisions are made.

FISH AND WILDLIFE RESOURCE VALUES

The Lillooet River and tributaries are extremely important fish and wildlife resources. Dyking and bank protection activities in the last century have caused a significant degradation of these environmental values. The engineering objective of a wider, more natural river corridor is compatible with environmental objectives.

Even without overall changes to the existing dyke system, there are opportunities to improve fish and wildlife resource values as part of ongoing dyke management activities and special restoration projects. Some possibilities in this respect are environmentally sensitive bank protection techniques, restoration of riparian vegetation, in-channel complexing, and side channel enhancement.

8.4 LAND USE PLANNING CONSIDERATIONS

Land use planning in the Lillooet River valley generally falls under the Squamish Lillooet Regional District, the Village of Pemberton, and the Mount Currie Band.

SQUAMISH LILLOOET REGIONAL DISTRICT AND VILLAGE OF PEMBERTON

For the Squamish Lillooet Regional District and the Village of Pemberton, official community plans, land use zoning and development approvals should consider flood hazard issues and incorporate floodplain management considerations. Development in sensitive or unprotected areas should be discouraged. Floodplain management bylaws are an effective tool that should continue to be applied.

MOUNT CURRIE BAND

For the Mount Currie Band, development is guided by the Physical Development Plan. Reserve boundaries generally dictate where development may occur, but there are opportunities to plan new developments away from sensitive or unprotected areas. The next update to the Physical Development Plan should be based on a comprehensive long term dyking plan.

Section 9

Lillooet River Gravel Management Plan

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9. LILLOOET RIVER GRAVEL MANAGEMENT PLAN

9.1 NEED FOR GRAVEL REMOVAL

In the upper reaches, the gradient of Lillooet River is sufficiently steep (0.003 to 0.007 m/m) that large quantities of gravel are transported on an annual basis and a braided morphology persists. In the lower reaches, the channel gradient declines to less than 0.0015 m/m as it approaches the forestry bridge at km 40 (Figure 5-1). Because of the reduced channel gradient, the river cannot continue to move all of the sediment and a significant portion of the bedload is deposited. Upstream of the forestry bridge, the result is a relatively abrupt change in channel morphology from braided to meandering.

However, the river continues to transport gravel-sized sediment beyond the forestry bridge. The contemporary annual gravel transport rate at the forestry bridge is estimated at 40,000 m^3/yr (see Section 5.3). The channel gradient continues to decline further downstream and Lillooet River loses its ability to move the coarser portion of its sediment load. The end result is that the entire gravel load of the river (and substantial interstitial sand) is deposited upstream of km 6 to 8.

The largest material is deposited first and these gravel deposits form an alluvial fan (a wedge of sediment). A characteristic of alluvial fans is that they continue to accumulate sediment as long as the river delivers more sediment than can be transported across the fan and beyond. As the bed of the river rises (aggrades), the water surface level also rises for a given flow. Over a period of years, the level of protection afforded by dykes is reduced.

For lower reaches of Lillooet River, a systematic raising of the channel bed has not been observed over the past thirty years. This can be partially attributed to the spacing of the cross sections (approximately 800 m), which is generally inadequate to quantify aggradation between surveys. [Lane et al. (1994) observed significant loss of information with cross section spacing greater than 3 m in a 10 to 20 m wide stream] A general absence of aggradation in lower reaches, however, is more likely in response to past gravel removal. Over the past two decades, gravel removal from Lillooet River are close to the lower bound estimates of input rates for some periods.

Without the previous gravel removal, the channel bed of Lillooet River would most certainly be higher than it is today downstream of km 20. Despite the removals, gravel accumulation has reduced the flood capacity locally. For example, a loss of approximately 20% of the cross sectional area was observed in 1997 adjacent to the junction of the airport road and the road to the Green River crossing. As shown on Sheet 4 (Appendix B), a gravel bar has formed at this location. Due to concerns of reduced flood conveyance, approval was granted by DFO and MWALP to remove gravel from the bar. (The removal volume has not been verified by the authors of this report.)

It should be noted that volumes discussed in this report are bulk volumes. Therefore, 40,000 m^3 refers to both the gravel and interstitial sand (which averages about 30% in lower reaches), and also porosity.

OPTIONS FOR MITIGATING THE HAZARDS OF GRAVEL DEPOSITION

The preceding example illustrates the need for a management plan that addresses aggradation in the lower reaches. There are a number of means by which this ongoing hazard might be mitigated, including:

- raising the dykes;
- reconstructing the dykes with greater setbacks; and
- lowering the river bed by gravel removal.

The hydraulic modelling results show that the dyke along the right bank of Lillooet River in Area 4 is high enough to prevent overflow of the 200-year return period flood event. However, the amount of freeboard available is highly variable and not fully adequate. In the other area of concern for gravel accumulation (Area 5), the flood modelling shows that the road surface is up to 0.3 m below the 200-year return period flood level (excluding freeboard).

While the dykes can be raised to offset their deficiencies, aggradation in the lower reaches will continue to be a problem for decades. Because the dykes can not continue to be raised in perpetuity, dyke raising on its own is not a practical option for long-term management of the gravel deposition. Reconstructing the dykes with greater setbacks is a viable option (particularly since gravel accumulations are relatively small), but is extremely expensive and there are increasing pressures for development on the floodplain.

The remaining option (other than accepting the flood hazard through flood construction levels and zoning) is to remove gravel from the river so the bed is prevented from rising. While gravel removal appears to be a viable solution, concerns about the ecological impact on the river must be addressed.

RIVER MORPHOLOGY AND ECOLOGY

In Lillooet River, significant gravel transport occurs for a relatively short period of the year – during the spring freshet and at the peak of fall storms. Once entrained, bed material does not typically travel a long way. In a simplified meandering system, sediment tends to be eroded on the outside of bends and redeposited on the inside of the next bend where flow velocities are reduced. The deposited material forms a major bar that redirects the flow to the adjacent bank. In this manner, bed material is staged down the river over many years.

While extensive bank engineering (particularly bank protection works) has reduced the lateral instability of Lillooet River, the transport and deposition of gravel is responsible for the creation and maintenance of fish and benthic invertebrate (aquatic insect) habitat. Any significant changes to the sediment transport regime are likely to result in corresponding changes to fish habitat.

While there have been no detailed studies of fish habitat in the mainstem channel, the river is known to support a wide variety of species including chinook, chum, coho, pink and sockeye salmon as well as Dolly Varden char, cutthroat trout, rainbow trout and steelhead (see Appendix A). Therefore, the meandering reach of the river probably provides conditions conducive to spawning of some species and rearing of others.

OTHER STUDIES

The key objective is to ensure appropriate flood protection in the Pemberton Valley in a manner that is consistent with maintaining the ecological character of the river. While gravel removal may appear to be incompatible with this objective, gravel management plans that consider ecological impacts have been established for both Fraser River and Vedder River in the Lower Mainland.

The Fraser River study (Church et al, 2001) in particular has been used as a template for this study. In that report, recommendations are made for the removal of gravel from the gravel bed reach of Fraser River between Laidlaw and Sumas Mountain for the purpose of maintaining flood security. While the morphology and ecology of Fraser River is considerably more complex than lower reaches of Lillooet River, some of the results and procedures used in the Fraser River study are directly transferable to Lillooet River.

Because fish habitat in the river is a consequence of channel morphology and sediment transport, the next section is a summary of Chapter 5. Experience of gravel removal from other rivers is then discussed, followed by a suggested approach to gravel management.

9.2 SUMMARY OF SEDIMENT TRANSPORT IN LILLOOET RIVER

Sediment transport and channel changes in lower reaches of Lillooet River can be summarized as follows:

- The annual gravel transport rate past the forestry bridge (km 40) is approximately 40,000 m^3/yr.

- Due to a progressive reduction in channel gradient, the entire gravel load of the river is deposited upstream of km 6 to 8.

- The current spacing of the cross sections (approximately 800 m) is inadequate to quantify aggradation between surveys. Gravel deposition below the forestry bridge tends to occur in well defined sedimentation zones. These zones are separated by long stable reaches (generally riprapped) that exhibit few channel changes and act as effective conduits for downstream gravel transport. In many cases, the existing monumented cross sections do not intersect these sedimentation zones.

- Because sedimentation tends to be localized, the potential for reduced channel conveyance during flooding is also localized. The implication is that flood management can concentrate on several points along the river rather than along its entire length.

- The annual bedload transport rate of 40,000 m^3/yr represents an average bed level increase of 0.12 m over a ten-year period. Hence, there is not a concern of rapid aggradation along the channel bed that would require immediate attention.

- The extensive engineering works conducted in the late 1940s have resulted in significant channel simplification, particularly downstream of the BC Rail bridge at km 15.5 (Figure 3-1). From an ecological perspective, there has been a considerable reduction in rearing habitat due to the loss of side and off channel habitat.

- Construction of the meander cutoffs in the late 1940s has resulted in 3 to 4 m of channel degradation upstream of the confluence with Ryan River and Miller Creek (km 20). Downstream reaches have also degraded (2 to 2.5 m) in response to the lowering of Lillooet Lake.

- The channel degradation has created a deeper, narrower channel. As a result, back channels have been cut off and river-edge wetlands have dewatered (a number are mapped on the 1945 sheets).

- The combined effects of lake lowering and channel straightening increased the channel gradient sufficiently that the limit of gravel transport has migrated downstream about 8 km.

The above summary addresses trends of sedimentation along the river and over time, which is essential in determining how much gravel might be removed from the river and where.

9.3 EXPERIENCE OF GRAVEL REMOVAL FROM OTHER RIVERS

A comprehensive review of the effects of gravel removal from rivers has been documented by Church et al. (2001). For the purposes of this report, only the general conclusions drawn by Church et al. are repeated here. These are:

- Gravel removal from a channel at rates larger than the rate of gravel recruitment produces lowering of the channel bed. Extraction ratios only modestly larger than 1.0 have been shown to result in degradation.

- Gravel removal at a point, or within a limited reach, can result in upstream and downstream propagating degradation. [These effects are most evident when sediment is removed from a pit within the channel.]

- Gravel removal from a bar causes loss of gravel from neighbouring bars upstream and downstream.

- Gravel removal from bars creates a wider, more uniform channel with less lateral variation in depth. The prominence of the pool-riffle sequence is also reduced.

- Channel morphology is simplified as the result of degradation following gravel removal. [Degradation creates a deeper, narrower channel. As a result, back channels are cut off and wetlands are dewatered.]

- Gravel removal from the channel may accelerate erosion and sediment transport locally in the short-term. [The effects are due to the removal of the bed surface armour of coarser stones, which regulates sediment entrainment.]

It is of interest to note that channel straightening and lake lowering on Lillooet River have produced many of the effects typically induced by intensive gravel removal.

Documented gravel removal from other systems mostly demonstrate the dramatic effects of removing quantities far in excess of supply. However, where the extraction ratio is modest (that is equal to or less than supply), there are few studies. For Fraser River, the extraction ratio is estimated to be between 0.3 and 0.4 for the past half century. There have been no obvious impacts to the channel morphology in the same time period. [With the exception of deep pit mining in a large side channel.] Therefore, it appears that impacts to channel morphology and hence fish habitat can be minimized by ensuring that gravel removal does not exceed the rate of supply.

After Church et al. (2001), additional sound gravel removal practices with respect to river ecology are ones that:

- preserve the topographic variability of the channel;
- maintain the normal range of topographies;
- maintain sedimentary features; and
- avoid dramatic changes in the duration or severity of ecologically stressful conditions in the channel.

Adoption of such practices to Lillooet River will assist in the maintenance of aquatic habitat for fish and benthic invertebrates. On Fraser River for example, high bar top habitat has found to be very important to juvenile fish during high flow conditions. Elimination of such habitat over a long reach could have adverse impacts.

9.4 SUGGESTED APPROACH TO GRAVEL MANAGEMENT

This section presents the suggested approach for gravel removal from Lillooet River, should it be decided that selected gravel removal represents a viable strategy for managing flood levels in the river. Gravel removal is only one of several strategies that might be pursued to manage flood levels. Consultation with the environmental agencies and stakeholders will be required to determine the preferred course of action.

HOW MUCH GRAVEL SHOULD BE REMOVED FROM THE RIVER?

Geomorphic analysis indicates that the annual gravel transport rate past the forestry bridge (km 40) is approximately 40,000 m^3/yr. This entire volume is not transported as far downstream as the more populated reaches of Area 4 nor is it evenly distributed along the channel. Gravel transport rates tend to fall off exponentially toward the downstream limit of entrainment.

Figure 9-1 illustrates two hypothetical sedimentation distributions. The first indicates linear transport with an equal distribution of the sediment along the channel. Under this scenario, approximately 15,000 m^3/yr of gravel is transported downstream of the confluence with Miller Creek and Ryan River at km 20. The more likely scenario is an exponential decrease in downstream sediment transport. In this case, it is thought that 8,000 to 10,000 m^3/yr is transported downstream of km 20.

Sediment transport past km 20 has important implications from the perspective of gravel removal. The hydraulic modelling of Section 7 indicates that the Q_{200} peak instantaneous flood level ranges between 1.25 to 4 m below the height of the right bank downstream of the forestry bridge. This level of protection decreases toward the confluence with Ryan River and Miller Creek (Figure 7-4). Considering the amount of degradation that has occurred in response to channel straightening this is not a surprising result.

In light of this result, gravel removal is not recommended between km 20 and km 40. If approximately 8,000 m^3 of gravel is transported past km 20 on an annual basis, then 32,000 m^3 is deposited upstream. This represents an average bed level increase of 15 cm over a ten year period. While the sedimentation will not be evenly distributed along the channel, there does not appear to be a need for systematic gravel removal given the existing level of flood protection. However, one-time gravel removal may be required over the next couple of decades if a specific area is shown to be accumulating gravel such that overbank flooding will occur. While no such areas are apparent presently, it should be noted that limited cross sections were resurveyed between km 20 and km 40 in 2001 and that few of the cross sections intersect the zones of sedimentation. If a specific reach is believed to have a significantly reduced channel conveyance, a cross section(s) can be surveyed and the information inserted into the Mike-11 hydraulic model to determine if overbank flooding would occur during the design flow.

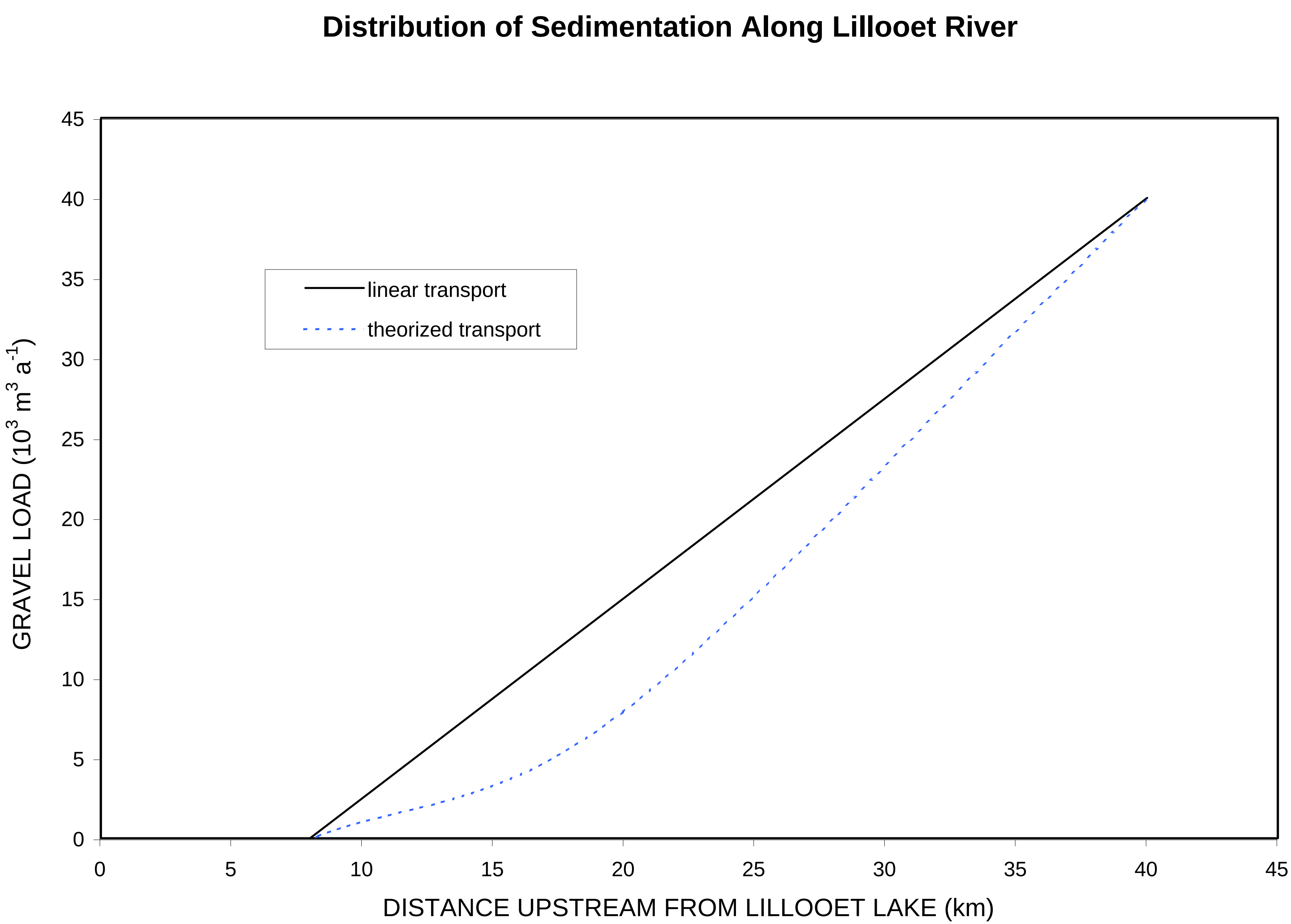

Distribution of Sedimentation Along Lillooet River
linear transport
theorized transport
GRAVEL LOAD (10^3 m^3 a^{-1})
DISTANCE UPSTREAM FROM LILLOOET LAKE (km)
0
5
10
15
20
25
30
35
40
45

Figure 9-1

One apparent solution to the downstream sedimentation is to remove 40,000 m^3 on an annual basis upstream of the forestry bridge. In this way, the gradual rise in the channel bed could be all but eliminated (some gravel would still be supplied from bank erosion and from the three major tributaries – Ryan River, Miller Creek and Pemberton Creek). However, the rate of gravel removal should not approach the total transport rate at the forestry bridge because sufficient gravel is required for downstream reaches to maintain normal turnover and renewal of gravels (i.e. maintenance and renewal of fish habitat). Annual gravel removal in the order of 40,000 m^3 would starve downstream reaches of gravel.

Following the above discussion, two approaches as to how much gravel should be removed are suggested.

1. To reduce the amount of downstream gravel transport, up to 10,000 m^3 (25% of the estimated load) could be removed annually from the numerous bars upstream of the forestry bridge. This would reduce the amount of gravel removal required downstream of km 20 (approximately 6,000 m^3/yr on average) and reduce sedimentation between km 20 and the forestry bridge.

2. If no gravel is removed upstream of the forestry bridge, then the rate of gravel removal downstream of km 20 should not exceed 8,000 m^3 on average.

The overall objective of gravel removal is to maintain the existing bed levels in Lillooet River downstream of Miller Creek and Ryan River. Several dyke sections in Areas 4 and 5 require upgrading and it is hoped that further future dyke raising will not be necessary if the existing deficiencies are resolved. An annual volume of 40,000 m^3 is not recommended for removal given the historic degradation between km 40 and km 20, and the desire to minimize ecological impacts.

If it is decided to proceed with selected gravel removal, then a decision needs to be made as to the volume to be removed. The removal of gravel upstream of the forestry bridge results in larger removal volumes but the need for downstream removal is reduced. On the other hand, ongoing removal upstream of the bridge would be more costly given the increased trucking costs.

Considerations of Variability in Gravel Transport

Two considerations are worthy of note with respect to the removal volumes. First, the supply of gravel is variable being dependant on peak flows. A number of years can pass when flows do not exceed the mean annual flood. In these years, small quantities of gravel are recruited into the reach.

More importantly, the sediment transport rate of 40,000 m^3/yr is subject to error. While there is confidence in the result, the average transport rate may vary by as much as $\pm$ 50%. A more precise estimate is made difficult by an incomplete record of gravel removal and the wide spacing of the cross sections. Therefore, the suggested removal volumes of 6,000 to 8,000 m^3 should be recognized as limit amounts.

Removal volumes should be restricted to the amount required to mitigate the flood hazard. In some years this would entail removal volumes smaller than the limit rate and in other years greater volumes. Determining the annual removal volume with respect to mitigating the flood hazard is discussed in the next section.

WHERE SHOULD GRAVEL BE REMOVED?

In the previous section, it was stated that removal volumes should be restricted to the amount required to mitigate the flood hazard. The cross section data suggests almost no increase in bed levels between km 20 and the downstream limit of gravel transport in the past thirty years. However, a lack of aggradation has been attributed to two factors:

- previous gravel removal; and
- in general, the cross sections do not intersect the gravel bars where most of the aggradation is likely to occur.

The majority of gravel removal has occurred on the gravel bars downstream of km 20. Since 1980, the documented volume of gravel removed averages about 9,000 m^3/yr. This value is most certainly a lower bound estimate as a number of undocumented small removals have probably occurred over the past two decades. During the same period, the cross section data show no general rise in the channel bed. In fact, a number of sections indicate small increases in channel area (Table 5-6). These trends suggest that aggradation in lower reaches can be controlled by selected removals from gravel bars. Furthermore, the suggested removal volume of 8,000 m^3/yr is consistent with past removal rates. [A value moderately in excess of 8,000 m^3/yr would appear to result in slight degradation.]

Accordingly, it is suggested that the gravel bars downstream of km 20 be regularly surveyed to monitor aggradation. By monitoring the growth of the bars, it can be determined when and how much gravel should be removed to mitigate the flood hazard.

For example, downstream of the confluence with Ryan River and Miller Creek gravel bars have been identified at:

- on the right bank at km 18 (Miners Bar);
- on the right bank at km 16.5 immediately downstream of the WSC gauge;
- on the left bank immediately upstream of the BC Rail bridge at km 15.8;
- on the right bank at km 14.5 (Beem Bar);
- on the right bank immediately upstream of the confluence of Pemberton Creek at km 11.5 (One-Mile Bar); and

- on the right bank near the junction of the Airport Road and the road to the Green River crossing (Hammer Bar, km 11).

While no decision has been made with regards to gravel removal, the above bars are candidate sites for removals in the future. Of these six bars, access difficulties (no road access and small side channels separate the bars from the bank) would probably preclude gravel removal at One-Mile Bar and the bar immediately upstream of the BC Rail bridge. In addition, gravel removal at the BC Rail Bridge could increase flows toward the left bank and cause problems at the bridge abutment. The WSC bar is also not a favourable site as local gravel removal could result in channel adjustments that impact discharge readings at the gauge. There remain three bars from which gravel could be removed with few difficulties: Miners Bar, Beem Bar, and Hammer Bar (Figure 9-2).

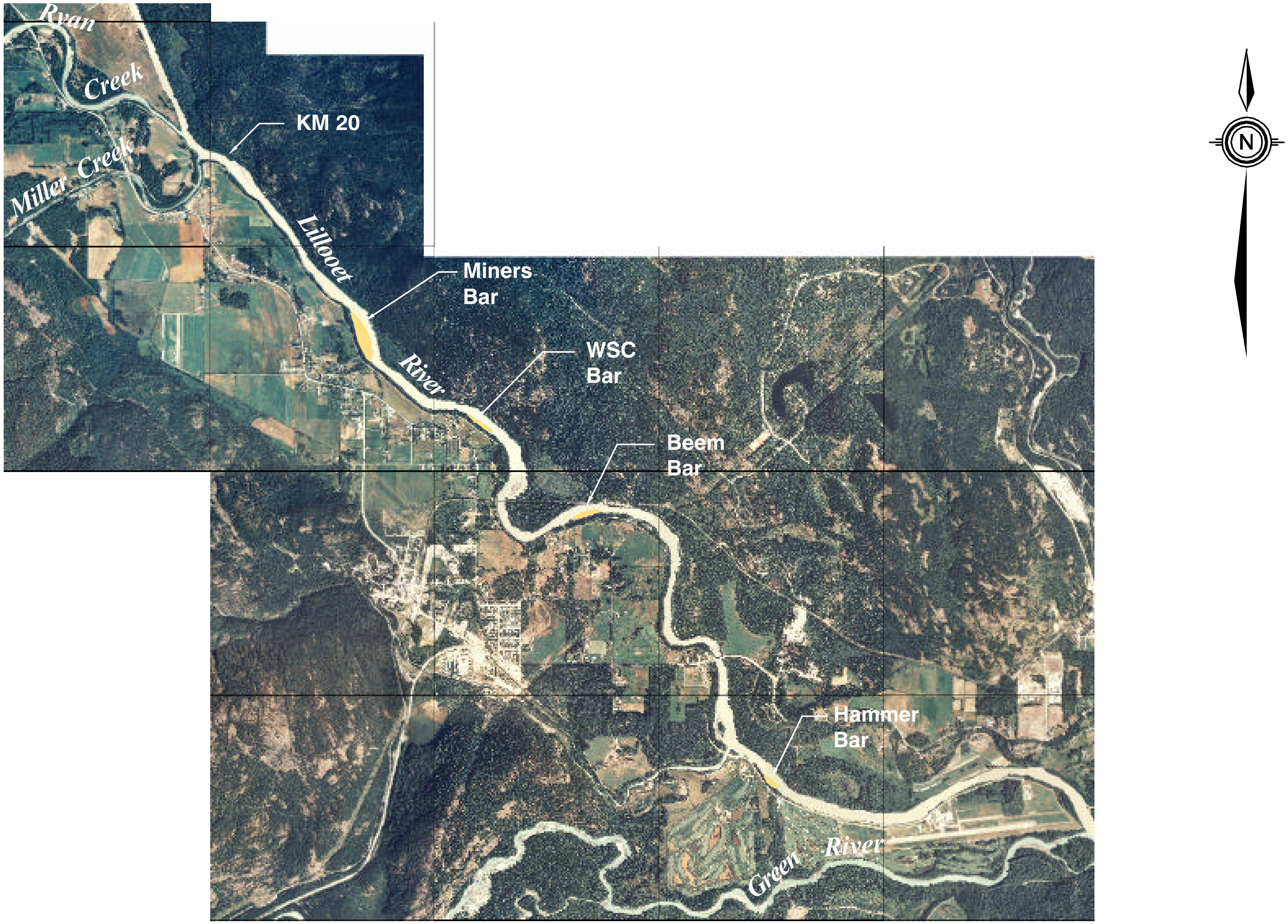
If gravel removal at these sites were to be considered, each of the bars should be surveyed during low flow conditions. One full cross section should also be surveyed where the channel area appears to be at a minimum. The extra cross sections would then be inserted into the Mike-11 hydraulic model to ensure that the existing bar configurations do not result in flooding for the design flow.

If flooding would not occur, the existing bar topographies would be considered the desired configuration to be maintained. If the hydraulic model indicated flooding, the required volume of gravel could be removed and the resulting topography would be considered the base condition. Surveys in subsequent years would indicate aggradational volumes at each of the bars and dictate the frequency and volumes of removals.

It is anticipated that the suggested approach to gravel removal will be sufficient to maintain bed levels downstream of km 20. Due in part to channel constraints, gravel accumulations appear to concentrate at discrete depositional areas. That is, gravel removal would not be required between the bars to maintain the channel bed.

There exists the possibility that gravel could continue to accumulate at the BC Rail Bar and One-Mile Bar such that the flood levels become compromised. However, it is also likely that bar development has an upper limit before excess material is transported further downstream. For example, XS 19.4 intersects the BC Rail Bar and no change in channel area occurred between 1985 and 2001. Nonetheless, both bars should continue to be observed in the event that an obvious problem develops, including a change in alignment that results in the abutments of the BC Rail bridge coming under direct attack.

There is also the potential for gravel to start accumulating at new locations. As such, the proposed sites should be considered subject to change over the next decade.



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Engineering Study for
Lillooet River Corridor

Project No.	Date
713-002	December 2002

1000 0 1000
Scale in Metres

Gravel Bar Locations Along Lower Reaches of Lillooet River

Figure 9-2

Sand Reach

Downstream of the gravel reach, the lower 6 to 8 km of the river is primarily a sand bed. As alluded to in Section 5.5, sedimentation in the sand reach is dependent on gravel accumulation further upstream. If gravel did not accumulate in the gravel reach, the channel bed in the sand reach would not aggrade significantly. An expected lack of aggradation is a function of the channel gradient and flow regime, which are sufficient to transport sand and finer sediment to the delta without depositing. That is, if the bed level at the gravel-sand transition remains stationary, sand and finer sediment transported by the river will be transported through to the delta. While the delta would advance due to the continuous supply of fine sediment, the channel would retain a relatively constant gradient.

Bed levels in the sand reach are not only controlled by the channel bed level at the gravel-sand transition. The water level at Lillooet Lake also controls bed levels. As shown by the lake lowering in the 1940s, the sand reach responded by degrading to a similar level. Conversely, if the lake level was to be raised, the sand reach would aggrade accordingly.

The question remains as to whether the removal of sand is recommended in the sand reach. Two scenarios are possible. In the first scenario, a program of gravel removal is implemented for the gravel reach. It is anticipated that the suggested approach to gravel removal would be sufficient to maintain bed levels downstream of km 20. Hence, bed levels in the sand reach would not be expected to increase.

In the second scenario, no gravel removal occurs and the gravel reach aggrades over a period of years to decades. While increases in bed level would not be extreme, an *average* of 0.12 m over a ten-year period (average is emphasized as gravel deposition occurs locally and is not distributed evenly along the channel), the sand bed would also aggrade in response to an increase in bed level near the gravel-sand transition. While the aggrading sand could be removed to reduce flood levels locally, the effect would be temporary. Removing the sand would not change the bed level at the upstream or downstream end of the sand reach, which ultimately controls the bed level in the sand reach. As such, any sand removed from the channel would be replaced within a few years by additional sand to approximately the same level. In conclusion, sand removal is not recommended downstream of km 8.

WHAT IS THE BEST WAY TO REMOVE GRAVEL?

Gravel could practically be removed from Lillooet River by two methods: (1) a pit dug in the channel or (2) bar scalping.

The former approach is most effective when sediment transport rates are high and the sediment can be intercepted in the pit. For control of gravel aggradation, this approach has been followed on Vedder River for a number of years and for Miller Creek in recent years. The disadvantage of pits is that the interception of a substantial portion of the bed load starves downstream reaches of gravel. The result is potential degradation and a reduction in habitat renewal. Deep pits have proven to be effective for Miller Creek as the pit is located near the downstream end of the system. On Vedder River, the sediment transport rate is considerably greater than lower reaches of Lillooet River. The high sediment transport rate results in a braided morphology and pits can be strategically located to minimize downstream impacts. Both cases have unique differences to the study area and as such, deep pit mining is not considered an ecologically sound approach for Lillooet River.

The alternative approach is bar scalping, which is the most common method for gravel removal. Bar scalping is usually conducted on dry surfaces and generally has little to no immediate impact on water quality or spawning sites. Sediment is removed from the bar with heavy machinery and the end result is generally a smooth surface that slopes down to the water. After Church et al. (2001), bar scalping presents several problems from a habitat perspective:

- It disturbs a relatively large area of the bed in comparison to the volume removed. The end result is a loose surface that is more susceptible to entrainment during subsequent peak flows in comparison to the normal, armoured bar surface.

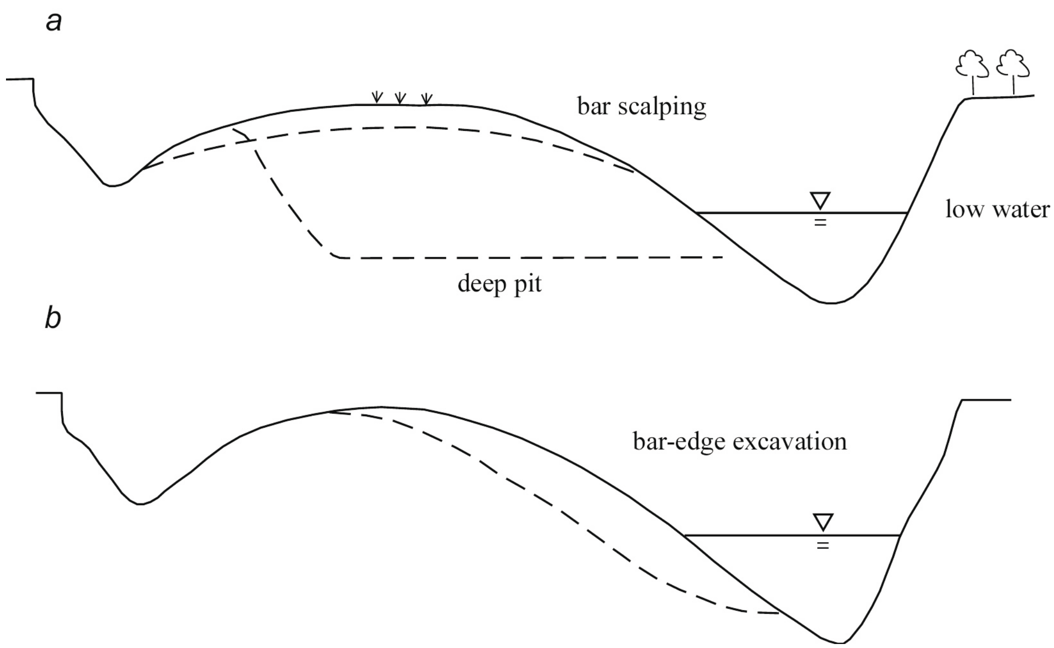
- It reduces the elevation of the bartop area, which probably provides the available habitat at normal high flows.

From an engineering perspective, bar scalping may also have minimal impact in reducing flood levels. Because bar scalping results in a shallow excavation, the channel conveyance (and flood levels by extension) may not increase significantly as relatively little water moves across bar tops, even in high flows.

To maintain the morphological features of the river and to provide an effective method for reducing flood levels locally, Church et al. (2001) have recommended an alternative strategy for gravel removal called *bar-edge scalping*. This type of scalping involves the removal of a wedge of sediment from the edge of the bar to widen the channel. As shown by Figure 9-3, the excavated section preserves the bar geometry and minimizes the loss of bartop area. The thickness of the excavation is varied in relation to the desired removal volume.

A disadvantage of bar-edge scalping is the need for excavation in the water. With respect to the river ecology, potential concerns are:

- fine sediment (predominantly fine sand and silt) would be released from the bed;
- disturbance of spawning areas; and
- potential impacts to benthic invertebrates.



Gravel Removal Strategies

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Figure 9-3

The first concern has probably been overstated for larger rivers such as Lillooet River where suspended sediment concentrations are extremely high (evidenced by the rapid growth of the delta). Avoidance of spawning sites by proper scheduling partially eliminates the second concern. There remains the concern of altering the gravel quality at potential spawning sites. However, permanent damage should be avoided as the gravel removal are proposed such that sedimentation processes are re-established after excavation (i.e. the excavations are not concentrated at one location to intercept the entire bedload).

The potential impacts of gravel removal to benthic invertebrates remains a relatively unknown subject. Species that can move only locally (10s of metres) may be strongly affected for some time. Conversely, species that can swim strongly, drift into position or are inoculated at a site from airborne adults may recover quickly. In Fraser River, ecological studies are currently underway to assess the effect of bar scalping on the occurrence of fish and benthic invertebrates. No significant effects have been detected so far, but none of the sites has been subject to repeated gravel removal.

Whether bar scalping or bar-edge scalping is adopted at a site, *it is important that the head of the bar not be disturbed*. The overall stability of the bar depends on the stability of the bar head, which is heavily armoured in comparison to the mid and lower bar area. Hence, gravel extractions should be limited to the lower two-thirds of the bar.

Baseline Ecological Study

A determination of whether bar scalping or bed-edge scalping is best suited for Lillooet River is not made in this study. While clear trends in sedimentation have been established, there is almost no information on the river ecology. The technique of gravel removal best suited for the river will depend on existing fish habitat.

As a result, the next step would be to undertake a baseline study of fish habitat be conducted for Lillooet River. The purpose of the study would be to identify the habitat requirements of fish occupying lower reaches of the river. Operational questions that should be addressed include:

- What fish species occupy the channel, during what months of the year, and for what life cycle purpose?
- What channel features provide important fish habitats through the year?
- How much variability is shown within species of fish with respect to habitat requirements?

For example, sedimentation features on bars have been found to provide microhabitats for juvenile fish on Fraser River. Therefore, Church et al. have recommended that irregular bar edges be created during gravel removal in order to maintain physical microhabitat features. Whether the same habitat features exist on Lillooet River is of interest and should be incorporated into any proposed removal scheme if present.

The suggested approach is similar to that used for the Fraser River study. However, it is recognized that similar resources are not available for Lillooet River. As a result, the fish habitat study for Lillooet River would be a considerably scaled down version of the ongoing Fraser study.

HOW FREQUENTLY SHOULD GRAVEL BE REMOVED FROM A SITE?

The frequency of gravel removal will be dictated by bar aggradation as determined by repeat surveys. In general, however, it is recommended that gravel not be removed in consecutive years at any site. This recommendation is primarily directed toward Miners Bar, which is the largest bar in the lower river and the first bar below km 20. The concern is that annual removal at this site would trap all the incoming sediment and starve downstream reaches. Even if an individual year results in significant recruitment, Miners Bar should be allowed to accumulate gravel for at least two years to ensure the onward transport of additional gravel. An exception is where best engineering judgement indicates that a removal is critical to manage flood levels.

If gravel is to be removed upstream of the forestry bridge, annual gravel removal is also not recommended. Although removal volumes would be larger if conducted every two or three years, the temporal aspect of any potential ecological impacts would be minimized.

SUMMARY

It is important to remember that engineering works have already contributed to morphological simplification of Lillooet River (i.e. channel straightening, lake lowering and significant bank protection). These activities have influenced sediment transport and continue to in the case of the bank protection. Thus, there are long-term ecological consequences with or without gravel removal.

The suggested locations for gravel removal should be considered as an adaptive experiment that is subject to change.

9.5 GRAVEL REMOVAL COST

This subsection pertains to the cost associated with the gravel management plan. While significant information has been presented on the development of such a plan, additional work needs to be completed. Funding is required to:

- complete a fish habitat assessment;
- establish baseline topographic surveys of the gravel bars where gravel removal is proposed;
- incorporate the additional survey information into the Mike-11 hydraulic model; and

- synthesize the above information into a working gravel management plan.

A proposed work program for establishing a Lillooet River Gravel Management Plan is outlined in the following table.

Table 9-1
Proposed Work Program for Lillooet River Gravel Management Plan

Task	Description
1. Project Initiation	▪ Define requirements of gravel bar survey and fish habitat study. ▪ Consult with PVDD, Mt. Currie Band, MWALP, and DFO to determine project objectives, refine work program, and prepare project implementation plan. ▪ Project initiation meeting at Pemberton. ▪ Walk gravel bars (where removal is proposed) to assess current conditions and confirm extent of project.
2. Bar Surveys	▪ Use monumented cross sections for survey control. ▪ Complete detailed topographic surveys of selected gravel bars between km 20 (approximately XS-25) and km 10 (approximately XS-10). ▪ Install control points (rebar) on the banks for follow-up surveys. ▪ Survey one cross section at each of the selected gravel bars where the conveyance capacity is at a minimum.
3. Hydraulic Modelling	▪ Import surveyed cross sections into Mike 11 model of Lillooet River to ensure channel has sufficient capacity for the design flow (200-year return period peak instantaneous flow) at the proposed gravel removal sites.
4. Fish Habitat Study	▪ Identify the habitat requirements of fish occupying lower reaches of the river. ▪ Sample fish (beach seines, gill nets and gee traps) in three distinct macro-habitats: the main channel, gravel bar edges and gravel bar tops. ▪ Sample a number of times through the year (e.g. pre, during and post freshet) to characterize the temporal use of the habitats. ▪ Take physical measurements of flow velocity, water depth, and substrate composition in conjunction with fish sampling to determine what physical conditions are associated with the various habitats. ▪ Determine whether microhabitats are an important component of the river morphology.
5. Analysis	▪ Prepare a report on fish habitat that documents results of fish sampling. ▪ Determine whether gravel can be removed such that ecological impacts are minimized. ▪ Incorporate ecological constraints and additional survey results into Lillooet River Gravel Management Plan (GMP).

Task	Description
6. Review	▪ Project review meeting committee with PVDD, DFO, and MWLAP. ▪ Obtain feedback on results and documents. ▪ Submit final GMP to PVDD.
7. Implementation Plan	▪ Work with PVDD to develop an implementation plan for gravel removal considering working and environmental constraints.

The above work program is estimated to cost about \$50,000 to \$80,000.

ANNUAL COSTS

Once the Lillooet River Gravel Management Plan has been finalized, there will be additional costs in years when gravel is removed. These costs include:

- follow-up surveys to determine volumetric changes;
- a short letter report from an experienced geoscientist detailing where to remove gravel, how much gravel to remove, and how to remove the gravel;
- extraction and transport costs;
- a provincial royalty to Land and Water BC Inc.; and
- environmental monitoring.

The first two items are expected to cost about \$5,000 to \$10,000. The follow-up surveys should be less expensive as control points will have been established at the various gravel bars.

The cost for gravel extraction and transport is more difficult to establish. It is expected that the PVDD will perform in the excavation work as they own a backhoe and have removed gravel from Miller Creek for a number of years. Therefore, there may be no external cost associated with the extraction of the gravel. With respect to transport of the gravel, it is expected that there is sufficient demand from the community for the gravel resource. That is, the gravel would either be trucked away for free, or at a cost per cubic metre. By charging money for the gravel, there is the potential for the gravel removal operation to occur at no cost to the PVDD. However, gravel removal from other rivers in British Columbia have traditionally been subject to a provincial royalty payment – an activity that is monitored by Land and Water BC. Royalties are generally in the range of \$0.50 to \$1 per m^3. The province may relax this requirement if there is low demand from the community for gravel that costs in excess of \$1/m^3. The final cost associated with gravel removal is environmental monitoring, which is estimated at about \$2,000 to \$5,000 per project.

Section 10

Implementation of Further Flood Protection Works

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10. IMPLEMENTATION OF FURTHER FLOOD PROTECTION WORKS

This section addresses the implementation of further flood protection works in the Pemberton Valley. This includes:

- review of the 'standard dyke' criteria and applicability to the study area;
- recommended freeboard allowance in establishing design dyke crest elevations; and
- an implementation plan to improve the level of flood protection.

This provides a basis for the Pemberton Valley Dyking District and the Mount Currie Band to move forward with appropriate projects. In some cases, the parties can act independently, and other cases it will be necessary to work together. Consultation with other interests will continue to be necessary in many situations.

10.1 DESIGNATION OF STANDARD DYKES

Standard dykes are defined by MWLAP as follows:

- crest elevation to 200-year return period flood level plus freeboard;
- minimum crest width of 3.6 m for equipment access;
- satisfy cross section requirements based on good engineering practice;
- include erosion protection where warranted on a technical basis;
- under management of a recognized authority for operation and maintenance;
- located on a right-of-way or easement in favour of the maintenance authority;
- O&M manual and as-constructed drawings have been developed, and ;
- a gravel management plan has been developed (where appropriate).

An appropriate objective is for all development in floodplain areas to be protected by standard dykes, with high population density and high-value development being the highest priority. In the study area, only newly upgraded dyke sections in Area 4 appear to meet the standard dyke criteria at present. Approximately 2,700 linear metres of dyke were upgraded in Area 4 in September 2002 to Q_{200} elevation plus 0.6 m freeboard. The Village of Pemberton (remaining parts of Area 4) and the core area of Mount Currie would be the top priorities for dyke upgrading to meet the standard dyke designation. Other areas could follow as funding and environmental approvals are obtained.

10.2 FREEBOARD ALLOWANCE

As noted in Sub-section 8.2, the design dyke crest elevation is determined by adding a freeboard allowance to the 200-year return period flood level (the design flood level). For a standard dyke, the more conservative of the following two cases determines the minimum dyke crest elevation:

- the 200-year return period peak instantaneous flood level plus 0.3 m; or
- the 200-year return period average daily flood level plus 0.6 m.

For Lillooet River and tributaries, the former criterion generally governs.

Additional freeboard above the minimum standard may be appropriate on rivers that are active in terms of bedload movement / sediment transport. The intention would be to make a reasonable allowance for deposition to occur between gravel removal activities. This situation is considered applicable to Lillooet River and tributaries.

For Lillooet River below Ryan River, based on the gravel management plan of Section 9, an additional freeboard allowance of 0.3 m is appropriate, providing a total freeboard allowance of 0.6 m above the peak instantaneous flood level. A greater allowance should be considered if the gravel management plan is not implemented.

For tributary rivers, a greater additional freeboard amount should be provided to reflect the fact that sediment deposition may be more intermittent and highly variable. For critical applications, site-specific investigation should be undertaken. However, as a general statement, the total freeboard allowance could be increased to 0.6 m where there is regular gravel removal (i.e. Miller Creek) or 1.0 m where there is no regular gravel removal.

For other than standard dyke purposes, freeboard can be considered on a site-specific basis in view of the project requirements.

10.3 IMPLEMENTATION PLAN FOR FLOOD PROTECTION IMPROVEMENTS

The suggested approach to flood protection in the Pemberton Valley is summarized in this sub-section. Actions for implementation by the Pemberton Valley Dyking District and the Mount Currie Band are identified, along with joint actions where appropriate.

ACTIONS BY PEMBERTON VALLEY DYKING DISTRICT

The Pemberton Valley Dyking District may lead the following projects as resources permit (in approximate order of priority):

1. **Area 4 Dyke:** Upgrade the Area 4 dyke (Appendix B, sheets 4, 5 and 6) to the standard dyke criteria (along Miller Creek, Lillooet River and Pemberton Creek). This will involve some further dyke raising and possibly widening. The design flood levels determined by this study, plus freeboard, would provide the design dyke crest elevations. However, in the downstream part of the reach, the design dyke crest may need to be raised slightly if Area 7/8 is to become dyked (see below). Dyke widening should generally occur on the land side slope of the existing dyke. Any practical opportunities for increasing the dyke setback from the river should be considered.

Note that approximately 2,700 m of dyke were raised in Area 4 along Lillooet River in September 2002.

2. **Gravel Management Plan:** Implement the gravel management plan for Lillooet River, commencing with a biophysical overview, selection of a gravel removal strategy, and development of an implementation plan. This action will require consultation with environmental agencies and other interests.

3. **Ryan River Management Plan:** Conduct additional cross section surveys, perform more detailed river modelling, evaluate sediment processes, and develop appropriate management plans to protect Areas 2 and 3.

4. **Area 5 Dyking:** If appropriate, develop and implement a dyking plan for Area 5 with respect to Pemberton Creek and Green River. Flood protection improvements along the Lillooet River reach of Area 5 would best be addressed in conjunction with Area 6 and Area 7, and should be considered a joint project (see below).

5. **Area 3 Dyke Upgrading:** If appropriate, upgrade the Area 3 dyke in accordance with the management plan of Item 3.

6. **Area 2 Dyke Upgrading:** If appropriate, upgrade the Area 2 dyke. The Lillooet River portion could be based on the flood levels determined by this study. The Ryan River portion would reflect the management plan of Item 3.

7. **Area 1 Dyking:** If there is a desire to provide an increased level of flood protection for this area in the future, the design flood level needs to be determined more accurately, an appropriate freeboard allowance needs to be determined, and a dyke alignment needs to be selected. Following these steps, dyke upgrading/construction could be undertaken. This action would most likely be under the direction of the Pemberton Valley Dyking District.

The above actions are river and dyke management tasks that fall directly under the jurisdiction of the PVDD. However, there is a need for further work to develop appropriate FCLs in the developed areas of the floodplain, especially in Area 4 which is subject to the greatest development pressure. The FCLs would reflect site-specific flood levels resulting from a dyke breach (MWLAP requires floodproofing of new development in floodplain areas, even with standard dykes). To this end, PVDD would need to work with the Village of Pemberton and the Squamish Lillooet Regional District, who are the local approved authorities, in undertaking a floodplain inundation study.

In April 2002, the PVDD applied for funding for dyke upgrade work in Area 4 (three sites), development of a detailed Gravel Management Plan, and a survey of Ryan River, based on the preliminary results of this study. The dyke upgrade work was subsequently approved and completed.

ACTIONS BY THE MOUNT CURRIE BAND

The Mount Currie Band may lead the following projects as resources permit (in approximate order of priority):

1. **Birkenhead River Management Plan:** Conduct additional cross section surveys, analyze sediment processes, extend the Mike 11 model, consider the need for additional freeboard allowance and/or gravel removal, and develop a design flood profile. This should involve consultation with the environmental agencies. Follow with implementation of the plan.

2. **Birkenhead River Dyke:** Upgrade the existing dyke, with possible downstream extension (to the end of the developed area) on a setback alignment.

3. **Floodproofing for I.R. Nos. 3 and 8:** Consider site-specific floodproofing (raising buildings, etc.) in this sparsely developed area. Ensure any further developments are elevated to the design flood level plus freeboard. Consider site-specific bank protection works on a project basis in consultation with the environmental agencies. If any comprehensive dyking or bank protection works are proposed, these should be preceded by a river study.

4. **In-Shuck Highway:** Consider future opportunities to raise or protect the highway in order to provide safe access/egress in times of flood. This should involve consultation with a broad group of stakeholders.

JOINT ACTIONS BY PEMBERTON VALLEY DYKING DISTRICT AND MOUNT CURRIE BAND

The following projects will require joint implementation by the Pemberton Valley Dyking District and Mount Currie Band:

1. **Area 7/8 Dyke:** Undertake a preliminary design study to select a preferred alignment and determine the resultant increased flood level (any dyking alternative will cause at least some localized increase in the flood profile). Input from property owners, environmental agencies and other stakeholders will be necessary.

   The objective of the study would be to select a preferred dyke alignment along Lillooet River from the highway bridge past Mount Currie. To protect Mount Currie against backwater flooding, this dyke would also need to extend across the floodplain to tie into the Birkenhead River dyke. This would involve a dyke crossing of Grandmother Slough, which would necessitate an environmentally sensitive floodbox. A range of possible dyke alignments for area 7/8 are shown on Figure 7-10.

   A key issue to be considered is the protection of the sloughs crossing the floodplain as these provide valuable fish habitat. Preliminary design drawings and construction cost estimates would be produced, along with an implementation plan that addresses right-of-way requirements, construction logistics, and maintenance responsibility. The impacts of any proposed Area 7/8 dyke on Areas 4, 5 and 6 need to be quantified such that the dyking plans for those areas can be adjusted accordingly. Following an approved plan, construction of works by the Pemberton Valley Dyking District and Mount Currie Band could proceed.

2. **Area 6 Dyke:** Undertake a preliminary design study to develop a plan for dyke upgrading for Lillooet River and Green River at the airport, preferably in conjunction with the study for Area 7/8. It may also be appropriate to include Area 5 in this study. A key issue to be addressed is the need to raise or relocate the existing road to the airport which presently acts as a dyke. Setback dykes should be favoured, especially along the Green River.

BANK PROTECTION WORKS

Minor bank protection works could be undertaken by both PVDD and Mount Currie Band, provided that environmental approvals are obtained. Preference should be given to bio-engineering (vegetation) methods on upper slopes, where practical, as an alternative to riprap alone. Where bank protection works may affect another party, such effects need to be addressed during the design of the works. Extensive works should be subject to detailed investigation.

LONG TERM RIVER RESTORATION

Over the next century, river processes such as meandering and erosion should be allowed to occur, and even be encouraged, where they would not result in a significant increase in flood risk increase to development areas. A wider river corridor would be highly desirable, and this should involve consideration of relocating dykes away from the river bank where there are opportunities to do so. In order to keep this option open, it is strongly suggested that construction of buildings near primary dykes (within roughly 200 m) be discouraged. As development occurs, opportunities to relocate dykes away from the river should be considered in critical areas, or a future setback alignment dyke right-of-way designated. Creation of a more natural river corridor will be a significant benefit to the fisheries resource, and supplemental in-stream fish habitat enhancements would also be appropriate.

COMMON ADMINISTRATIVE ACTIONS

Administrative actions that are common to both PVDD and Mount Currie Band include the following (in no particular order):

- ongoing dyke maintenance activities (vegetation control, crest surfacing, removal of obstructions, etc.);

- periodic survey monitoring of river cross sections every 5 to 10 years;

- careful monitoring of large flood events to obtain high water levels and compare them with the Mike 11 model results;
- continued acquisition of dyke rights-of-way or easements; and
- installation of additional hydrometric stations to obtain better flood flow data.

These actions will require ongoing attention.

FLOODPLAIN MANAGEMENT

It is suggested that the Mount Currie Band update its Physical Development Plan to reflect the results of this study. Future development should be concentrated in areas protected by standard dykes. Provision for floodproofing developments in other areas should also be provided.

Similarly, the Squamish Lillooet Regional District and the Village of Pemberton should be encouraged to enhance their floodplain management activities. This should involve consideration of building bylaws, careful scrutiny of proposed subdivisions, and building permit requirements.

10.4 NEXT STEPS

This sub-section provides suggested next steps for PVDD and the Mount Currie Band in order to proceed with the implementation plan outlined in Sub-section 10.3.

PEMBERTON VALLEY DYKING DISTRICT

Dyke Maintenance

PVDD should continue to implement a dyke maintenance program for all dykes under its jurisdiction. Priority should be given to the Area 4 dyke since it protects the greatest amount of development. Future dyke maintenance activities should include periodic survey monitoring of river cross-sections and dyke crests.

Flood Protection Improvements

PVDD should also proceed with implementation of the flood protection improvement actions identified in Sub-section 10.3 on a priority basis. This will involve upgrading of existing dykes, consideration of some new dykes and development of river management plans.

A dyke system that meets the standard dyke criteria should be PVDD's ultimate objective. The rate of implementation will be dictated by available funding. At present, the primary source of funding is local taxpayers, and this is supplemented by cost-sharing grants from MWLAP for approved projects. Future dyking activities may necessitate a review of the local dyke tax rate, particularly since the continuation of funding from MWLAP is uncertain. Other new funding sources could also be pursued.

Dyke Right-of-way

A recent condition of MWLAP funding is that dykes be located on rights-of-way in favour of the dyking authority. In order to maximize the funding contribution from MWLAP, PVDD will need to continue to be proactive in right-of-way acquisition.

PVDD operations should include legal surveys in priority areas to determine the location of the dyke structure in relation to the right-of-way boundaries. Where there are discrepancies, further right-of-way acquisition could be triggered.

Cooperation with Mount Currie Band

PVDD should work cooperatively with the Mount Currie Band in developing a dyking plan for Areas 7 and 8. If PVDD wishes to initiate dyke improvements in Areas 5 and 6, these are also subject to coordination with the Mount Currie Band.

Floodplain Management

PVDD may wish to refer the issues of floodplain management (including flood inundation modelling and floodproofing) for future development to the Village of Pemberton and the Squamish Lillooet Regional District. This should include consideration of possible long term setback dyke alignments and the need, where possible, to avoid construction of buildings near (within roughly 200 m) of dykes.

Bank Protection

dPVDD may consider the need for site-specific bank protection works on an as-required basis, subject to approval from the environmental agencies. The objective of bank protection works should only be to stabilize critical river bank areas, not to stabilize the entire river.

Long Term River Restoration

Over the long term, river processes such as meandering and erosion should be allowed to occur where there is no risk to the dyke system. Alternatives for providing a wider river corridor through setback dykes should be considered.

Monitoring

Future flood events should be carefully monitored in order to provide a basis for possibly refining the river modelling work at some future date. This would be facilitated by installation of additional hydrometric stations.

MOUNT CURRIE BAND

Dyke Maintenance

The Mount Currie Band should develop a dyke maintenance program for all dykes under its jurisdiction.

Flood Protection Improvements

The Mount Currie Band should also proceed with implementation of the flood protection improvement actions identified in Sub-section 10.3 on a priority basis. This will involve upgrading of existing dykes, consideration of new dykes and development of river management plans.

A dyke system that meets the standard dyke criteria should be the Mount Currie Band's ultimate objective. The rate of implementation will be dictated by available funding. At present, the primary source of funding is INAC through its capital project funding programs. On the basis of this study, the Band is now in a position to make a funding submission for the Birkenhead River Management Plan.

Cooperation with PVDD

The Mount Currie Band should work cooperatively with PVDD in developing a dyking plan for Areas 7 and 8. This could form the basis of a second funding submission to INAC for some portion of the funding for a preliminary design study.

Dyke Right-of-way

The Mount Currie Band should ensure that its dykes are located on Band property or on rights-of-way in favour of the Band.

Floodplain Management

The Mount Currie Band should update its Physical Development Plan to reflect the results of this study. Development should be concentrated in dyked areas, with consideration of floodproofing measures.

Bank Protection

The Mount Currie Band may consider the need for site-specific bank protection works on an as-required basis, subject to approval from the environmental agencies. The objective of bank protection works should only be to stabilize critical river bank areas, not to stabilize the entire river.

Long Term River Restoration

Over the long term, river processes such as meandering and erosion should be allowed to occur where there is no risk to the dyke system. Alternatives for providing a wide river corridor through setback dykes should be considered.

Monitoring

Future flood events should be carefully monitored in order to provide a basis for possibly refining the river modelling work at some future date. This would be facilitated by installation of additional hydrometric stations.

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