FLOOD CONTROL AND SEDIMENT TRANSPORT STUDY
OF THE VEDDER RIVER
by
DAVID GEORGE McLEAN
B.A.Sc., University of British Columbia, 1975

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF
THE REQUIREMENTS FOR THE DEGREE OF
MASTER OF APPLIED SCIENCE
in
THE FACULTY OF GRADUATE STUDIES
The Department of Civil Engineering

We accept this thesis as conforming to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA
April, 1980

© David George McLean
In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for extensive copying of this thesis for scholarly purposes may be granted by the Head of my Department or by his representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of Civil Engineering

The University of British Columbia
2075 Wesbrook Place
Vancouver, Canada
V6T 1W5

Date April 12, 1980
ABSTRACT

The Chilliwack River flows through the Cascade Mountains until reaching a narrow gorge near Vedder Crossing where it flows onto the Fraser Lowlands and eventually meets the Fraser River. Below Vedder Crossing, the river is actively building an alluvial fan by depositing its sediment load of gravel and sand. This deposition has resulted in frequent channel shifts over the fan surface with the most recent migration occurring around 1894 when the river shifted down Vedder Creek.

Over the last century the Vedder River has been undergoing very complex changes in response to changes in the incidence of severe floods, changes in sediment supply and interference from river training.

Extensive channelization works carried out in the 1960's induced temporary degradation over part of the channel which was accompanied by rapid aggradation in the reach immediately downstream. This rapid channel adjustment ceased in less than 10 years.

In 1975 a flood having a return period of about 10 years deposited 260,000 cubic yards of sediment onto the fan which increased the mean bedlevel by nearly 1 foot. By comparison, the average annual deposition rate was estimated to be 72,000 cubic yards per year. Based on bedload transport calculations, approximately
700,000 cubic yards of sediment could be deposited by a 50 year rainstorm flood.

In order to provide long term flood control, either the upstream sediment supply will have to be reduced or dredging will have to be carried out on the lower river. It is not feasible to eliminate aggradation by transporting the incoming bedload through the system and into the Fraser River.

Some strategies are considered which, by controlled dredging and training would maintain the channel permanently in its present position. More severe floods would be contained by set-back dikes. It is thought that, with care, these measures could be consistent with salmon habitat requirements.
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>CHAPTER</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>ii</td>
</tr>
<tr>
<td>TABLE OF CONTENTS</td>
<td>iv</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>vi</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>viii</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>xi</td>
</tr>
<tr>
<td>I</td>
<td>INTRODUCTION</td>
</tr>
<tr>
<td>II</td>
<td>HISTORY OF RIVER ACTIVITY</td>
</tr>
<tr>
<td>2.1 Site Location</td>
<td>6</td>
</tr>
<tr>
<td>2.2 Early History and Settlement</td>
<td>6</td>
</tr>
<tr>
<td>2.3 River Training and Flooding Along the Vedder River</td>
<td>12</td>
</tr>
<tr>
<td>III</td>
<td>HYDROLOGY</td>
</tr>
<tr>
<td>3.1 Basin Characteristics</td>
<td>24</td>
</tr>
<tr>
<td>3.2 Climate</td>
<td>25</td>
</tr>
<tr>
<td>3.3 Streamflow Data</td>
<td>25</td>
</tr>
<tr>
<td>3.4 Basin Runoff</td>
<td>26</td>
</tr>
<tr>
<td>3.5 Flood Hydrology</td>
<td>27</td>
</tr>
<tr>
<td>3.6 Flood Frequency Analysis</td>
<td>34</td>
</tr>
<tr>
<td>IV</td>
<td>GEOLOGY AND PHYSIOGRAPHY</td>
</tr>
<tr>
<td>4.1 Physiography</td>
<td>45</td>
</tr>
<tr>
<td>4.2 Glacial History</td>
<td>47</td>
</tr>
<tr>
<td>4.3 Formation of the Chilliwack River Fan</td>
<td>49</td>
</tr>
<tr>
<td>V</td>
<td>RIVER PROCESSES UPSTREAM OF VEDDER CROSSING</td>
</tr>
<tr>
<td>5.1 Data Available</td>
<td>54</td>
</tr>
<tr>
<td>5.2 Analysis of River Processes</td>
<td>58</td>
</tr>
<tr>
<td>5.3 The Sediment Supply of the Chilliwack River</td>
<td>78</td>
</tr>
<tr>
<td>CHAPTER</td>
<td>PAGE</td>
</tr>
<tr>
<td>---------</td>
<td>------</td>
</tr>
<tr>
<td>VI</td>
<td></td>
</tr>
<tr>
<td>CHANNEL PROCESSES BELOW VEDDER CROSSING.</td>
<td>83</td>
</tr>
<tr>
<td>6.1 Data Available.</td>
<td>83</td>
</tr>
<tr>
<td>6.2 Channel Pattern of the Vedder River.</td>
<td>86</td>
</tr>
<tr>
<td>6.3 Channel Hydraulics.</td>
<td>92</td>
</tr>
<tr>
<td>6.4 Bed Material Characteristics.</td>
<td>99</td>
</tr>
<tr>
<td>VII</td>
<td></td>
</tr>
<tr>
<td>SEDIMENT TRANSPORT</td>
<td>105</td>
</tr>
<tr>
<td>7.1 Suspended Load.</td>
<td>105</td>
</tr>
<tr>
<td>7.2 Bedload</td>
<td>106</td>
</tr>
<tr>
<td>VIII</td>
<td></td>
</tr>
<tr>
<td>AGGRADATION ON THE VEDDER RIVER.</td>
<td>118</td>
</tr>
<tr>
<td>8.1 The Process of Aggradation.</td>
<td>118</td>
</tr>
<tr>
<td>8.2 Historical Deposition Rates on the Chilliwack River</td>
<td>121</td>
</tr>
<tr>
<td>8.3 Aggradation During the 1975 Flood</td>
<td>132</td>
</tr>
<tr>
<td>8.4 Prediction of Future Aggradation.</td>
<td>135</td>
</tr>
<tr>
<td>IX</td>
<td></td>
</tr>
<tr>
<td>FLOOD CONTROL ON THE VEDDER RIVER.</td>
<td>141</td>
</tr>
<tr>
<td>9.1 Some Examples of Flood Control Practice.</td>
<td>141</td>
</tr>
<tr>
<td>9.2 Flood Control on the Vedder River.</td>
<td>146</td>
</tr>
<tr>
<td>X</td>
<td></td>
</tr>
<tr>
<td>CONCLUSIONS.</td>
<td>162</td>
</tr>
<tr>
<td>BIBLIOGRAPHY</td>
<td>166</td>
</tr>
<tr>
<td>TABLES</td>
<td>174</td>
</tr>
<tr>
<td>FIGURES</td>
<td>207</td>
</tr>
<tr>
<td>APPENDIX 1</td>
<td>266</td>
</tr>
<tr>
<td>TABLE</td>
<td>TITLE</td>
</tr>
<tr>
<td>-------</td>
<td>-------</td>
</tr>
<tr>
<td>1</td>
<td>SUMMARY OF FLOOD CONTROL WORK ON THE VEDDER RIVER.</td>
</tr>
<tr>
<td>2</td>
<td>BASIN RUNOFF CHARACTERISTICS.</td>
</tr>
<tr>
<td>3</td>
<td>MAJOR FLOODS ON CHILLIWACK RIVER.</td>
</tr>
<tr>
<td>4</td>
<td>SUMMARY OF SEASONAL FLOOD DATA.</td>
</tr>
<tr>
<td>5</td>
<td>FLOOD FREQUENCY FLOWS.</td>
</tr>
<tr>
<td>6</td>
<td>MAXIMUM INSTANTANEOUS FLOOD FREQUENCY ESTIMATES.</td>
</tr>
<tr>
<td>7</td>
<td>FREQUENCY OF HISTORICAL FLOODS.</td>
</tr>
<tr>
<td>8</td>
<td>SUMMARY OF MAPS/AIRPHOTOS ABOVE VEDDER CROSSING.</td>
</tr>
<tr>
<td>9</td>
<td>BED MATERIAL DATA ABOVE VEDDER CROSSING.</td>
</tr>
<tr>
<td>10</td>
<td>SUMMARY OF HYDROLOGIC DATA ALONG CHILLIWACK RIVER.</td>
</tr>
<tr>
<td>11</td>
<td>SUMMARY OF GEOMORPHIC FEATURES ALONG CHILLIWACK RIVER.</td>
</tr>
<tr>
<td>12</td>
<td>SUMMARY OF CHANNEL HYDRAULICS ALONG CHILLIWACK RIVER.</td>
</tr>
<tr>
<td>13</td>
<td>INCIPIENT MOTION FOR BED MATERIAL ALONG CHILLIWACK RIVER.</td>
</tr>
<tr>
<td>14</td>
<td>SUMMARY OF WIDTH CHANGES ABOVE VEDDER CROSSING.</td>
</tr>
<tr>
<td>15</td>
<td>SUMMARY OF CHANNEL CHANGES AND SEDIMENT EROSION ABOVE VEDDER CROSSING.</td>
</tr>
<tr>
<td>16</td>
<td>SUMMARY OF CROSS SECTION DATA BELOW VEDDER CROSSING.</td>
</tr>
<tr>
<td>TABLE</td>
<td>TITLE</td>
</tr>
<tr>
<td>-------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>17</td>
<td>WATER SURFACE SLOPE SURVEYS</td>
</tr>
<tr>
<td>18</td>
<td>BANKFULL PROPERTIES OF BRAIDED SUB-CHANNELS ALONG THE VEDDER RIVER</td>
</tr>
<tr>
<td>19</td>
<td>COMPOSITE BED MATERIAL SIZE DISTRIBUTION ALONG VEDDER RIVER</td>
</tr>
<tr>
<td>20</td>
<td>DOWNSTREAM BED MATERIAL CHANGES ON GRAVEL RIVERS.</td>
</tr>
<tr>
<td>21</td>
<td>SUMMARY OF SUSPENDED LOAD DATA ON VEDDER RIVER.</td>
</tr>
<tr>
<td>22</td>
<td>SUMMARY OF BEDLOAD MEASUREMENTS COLLECTED ON VEDDER RIVER</td>
</tr>
<tr>
<td>23</td>
<td>SUMMARY OF AGgradation ESTIMATES PRIOR TO 1975 FLOOD</td>
</tr>
<tr>
<td>24</td>
<td>BEDLOAD TRANSPORT FOR VARIOUS FLOOD EVENTS.</td>
</tr>
<tr>
<td>25</td>
<td>SUMMARY OF CHANNEL DATA ON FOUR RIVER CONTROL PROJECTS.</td>
</tr>
<tr>
<td>26</td>
<td>REGIME EQUATIONS FOR ACTIVE GRAVEL RIVERS.</td>
</tr>
<tr>
<td>A.1</td>
<td>SUMMARY DESCRIPTION OF MEASUREMENT SITES</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>TITLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Site Location.</td>
<td>208</td>
</tr>
<tr>
<td>2</td>
<td>Study Area Below Vedder Crossing</td>
<td>209</td>
</tr>
<tr>
<td>3</td>
<td>Chilliwack River Channels Below Vedder Crossing.</td>
<td>210</td>
</tr>
<tr>
<td>4</td>
<td>Channel Pattern of the Vedder River 1930-1976.</td>
<td>211</td>
</tr>
<tr>
<td>5</td>
<td>Flood Paths Along Vedder River in 1951, 1975.</td>
<td>214</td>
</tr>
<tr>
<td>6</td>
<td>Chilliwack River Basin</td>
<td>215</td>
</tr>
<tr>
<td>7</td>
<td>Monthly Met Data for Chilliwack Basin.</td>
<td>216</td>
</tr>
<tr>
<td>8</td>
<td>Summary of Stream Gauging Operations Along Chilliwack River</td>
<td>217</td>
</tr>
<tr>
<td>9</td>
<td>Monthly Flows Along Chilliwack River.</td>
<td>218</td>
</tr>
<tr>
<td>10</td>
<td>Historical Occurrences of Past Floods</td>
<td>219</td>
</tr>
<tr>
<td>11</td>
<td>Comparison of Two Snowmelt and Rainstorm Floods</td>
<td>220</td>
</tr>
<tr>
<td>12</td>
<td>Correlation Between Nooksack River and Chilliwack River Floods.</td>
<td>221</td>
</tr>
<tr>
<td>13</td>
<td>Predicted and Recorded 1975 Flows.</td>
<td>222</td>
</tr>
<tr>
<td>14</td>
<td>Predicted Flood Hydrographs for 1932 and 1951.</td>
<td>223</td>
</tr>
<tr>
<td>15</td>
<td>Time Series of Annual Floods at Vedder Crossing.</td>
<td>224</td>
</tr>
<tr>
<td>16</td>
<td>Flood Frequency Analysis at Vedder Crossing.</td>
<td>225</td>
</tr>
<tr>
<td>FIGURE</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>17</td>
<td>Stream Profile of Chilliwack River</td>
<td>226</td>
</tr>
<tr>
<td>18</td>
<td>Fan Profiles</td>
<td>227</td>
</tr>
<tr>
<td>19</td>
<td>Channel Cross-Sections Above Vedder Crossing</td>
<td>228</td>
</tr>
<tr>
<td>20</td>
<td>Hydraulic Geometry at Hydrometric Gauges Above Vedder Crossing</td>
<td>229</td>
</tr>
<tr>
<td>21</td>
<td>Channel Reach Descriptions</td>
<td>230</td>
</tr>
<tr>
<td>22</td>
<td>Comparative Airphotos—Vedder Crossing to Liumchen Creek</td>
<td>235</td>
</tr>
<tr>
<td>23</td>
<td>Channel Pattern Upstream of Vedder Crossing</td>
<td>236</td>
</tr>
<tr>
<td>24</td>
<td>Channel Changes Near Vedder Crossing</td>
<td>239</td>
</tr>
<tr>
<td>25</td>
<td>Conceptual Bedload Movement Above Vedder Crossing</td>
<td>242</td>
</tr>
<tr>
<td>26</td>
<td>Approximate Location of Vedder River Sections</td>
<td>243</td>
</tr>
<tr>
<td>27</td>
<td>Cross-Sections Along Vedder River</td>
<td>244</td>
</tr>
<tr>
<td>28</td>
<td>Sections Showing Floodplain and Channel Topography</td>
<td>247</td>
</tr>
<tr>
<td>29</td>
<td>Effect of Fraser and Vedder River Flows on the Stage in the Vedder Canal</td>
<td>248</td>
</tr>
<tr>
<td>30</td>
<td>Hydraulic Geometry at Yarrow and Vedder Crossing</td>
<td>249</td>
</tr>
<tr>
<td>31</td>
<td>Comparison of Resistance Formulas Using Yarrow Data</td>
<td>250</td>
</tr>
<tr>
<td>32</td>
<td>Downstream Changes in Bed Material Size Along Vedder River</td>
<td>251</td>
</tr>
<tr>
<td>33</td>
<td>Bedload Grain Size Measured at Yarrow</td>
<td>252</td>
</tr>
<tr>
<td>FIGURE</td>
<td>TITLE</td>
<td>PAGE</td>
</tr>
<tr>
<td>--------</td>
<td>----------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>34</td>
<td>Bedload Size Distribution Curves at Different Flow Conditions.</td>
<td>253</td>
</tr>
<tr>
<td>35</td>
<td>Bedload Transport at Yarrow</td>
<td>254</td>
</tr>
<tr>
<td>36</td>
<td>Comparison of Measured and Computed Bedload at Yarrow</td>
<td>255</td>
</tr>
<tr>
<td>37</td>
<td>Bedload Estimates at Vedder Crossing</td>
<td>256</td>
</tr>
<tr>
<td>38</td>
<td>Theoretical Aggradation Profiles</td>
<td>257</td>
</tr>
<tr>
<td>39</td>
<td>Comparison of Mean Bedlevels, 1963-1975</td>
<td>258</td>
</tr>
<tr>
<td>40</td>
<td>Effect of Channelization on Channel Geometry</td>
<td>259</td>
</tr>
<tr>
<td>41</td>
<td>Specific Gauge Record Near Yarrow</td>
<td>260</td>
</tr>
<tr>
<td>42</td>
<td>Specific Gauge Record at Vedder Crossing</td>
<td>261</td>
</tr>
<tr>
<td>43</td>
<td>Variation in Deposition Along Vedder River</td>
<td>262</td>
</tr>
<tr>
<td>44</td>
<td>Derived Hydrographs for Snowmelt and Rainstorm Floods</td>
<td>263</td>
</tr>
<tr>
<td>45</td>
<td>Wide Vedder River Flood Control</td>
<td>264</td>
</tr>
<tr>
<td>46</td>
<td>Possible Set-Back Dike and River Training Alignment</td>
<td>265</td>
</tr>
<tr>
<td>A.1</td>
<td>Comparison of Bedload Formulas on Vedder River</td>
<td>280</td>
</tr>
<tr>
<td>A.2</td>
<td>Comparison of Bedload Formulas on Elbow River</td>
<td>281</td>
</tr>
<tr>
<td>A.3</td>
<td>Comparison of Bedload Formulas on North Saskatchewan River</td>
<td>282</td>
</tr>
<tr>
<td>A.4</td>
<td>Comparison of Bedload Formulas on Clearwater River</td>
<td>283</td>
</tr>
<tr>
<td>A.5</td>
<td>Comparison of Bedload Formulas on Snake River</td>
<td>284</td>
</tr>
</tbody>
</table>
ACKNOWLEDGEMENTS

The author is very grateful to the financial support and encouragement given by his supervisor, Professor M. C. Quick. Also, he would like to thank Dr. S. O. Russell and Dr. M. Church for their many helpful discussions.

Appreciation is also extended to the staff of Water Survey of Canada and the B.C. Water Resources Service who provided unpublished data that was used in this study.

Finally, to my wife, Cathy. This thesis could not have been completed without your patience and support.
CHAPTER I
INTRODUCTION

The Chilliwack River rises in the Cascade Mountains 100 miles east of Vancouver, British Columbia and flows for 25 miles through a rugged, mountainous valley. Near the town of Vedder Crossing the river emerges from a narrow canyon and flows across the Fraser Lowland until joining the Fraser River near Chilliwack.

Downstream of Vedder Crossing the Chilliwack River has built a conical alluvial fan by depositing its sediment load of sand and gravel, periodically causing the river channel to shift across a wide area. Below Vedder Crossing the river is known as the Vedder River, since the last major channel shift diverted the Chilliwack River down Vedder Creek. Continued channel aggradation in recent times has contributed to frequent flooding to farms and towns located near the river. In December 1975 flooding on the Vedder River caused approximately 260,000 cubic yards of gravel to be deposited over a distance of approximately three miles.

Past attempts at flood control have produced strong criticism from environmental groups and fisheries agencies due to destruction of spawning and rearing areas for salmon and steelhead trout. The Vedder River supports important commercial pink and chum salmon runs as well as a sport fishery for coho and steelhead.
This conflict between flood control and fisheries management illustrates the need for long term planning of development along the Vedder River. Ideally, any design of flood control measures should recognize the characteristic morphologic and sediment transport processes occurring on the river. Unfortunately, river engineering is in a relatively primitive stage of development compared to many other engineering disciplines. At present, river processes are only poorly understood and the effects of interfering with these processes cannot always be predicted accurately. As a result, many examples have been recorded where altering a river's regime has caused adverse environmental consequences and unexpected changes to the earlier pattern of river behaviour (Parker and Andres, 1976). Therefore, until considerable advances have been made in understanding basic river processes, engineers involved in river problems must rely on records of past experiences and on systematic observations of river behaviour.

Although alluvial fans occur throughout the world, most previous studies have been carried out in semi-arid or arid environments (Bull, 1962, 1964; Denny, 1965; Hooke, 1967; Lustig, 1965; Lustig and Busch, 1967). There has been considerably less documentation of alluvial fans located in temperate, mountainous environments. Studies have been carried out in the Alberta Rockies (McPherson
and Hurst, 1972; Smith, 1972; NHCL, 1979), in British Columbia (Ryder, 1970, 1971; NHCL, 1975, 1976) and in the Arctic (Legget et al, 1966; Anderson and Hussey, 1962). In addition Malcovish (1974) carried out a number of model experiments to illustrate some general principles of alluvial fan behaviour.

In order to understand the nature of the sedimentation and flooding processes on the Vedder River, a number of key questions must be answered. Some of the most important are:

1. What are the magnitudes and associated frequencies of floods on the river?

2. What is the volume of sediment that can be deposited on the fan during these floods?

3. Where does most sediment deposition occur? What factors govern the pattern of deposition along the river?

4. How has past channelization and flood control activities altered the pattern of sedimentation?

5. What have been the most important lateral and vertical activities on the river, and how do these affect the present-day and future river processes?

6. Can a particular channel alignment be found that will provide long term flood protection and at the same time provide a suitable spawning habitat for the salmon?
7. What are the main sediment sources in the Chilliwack River basin and can sedimentation problems on the fan be solved by upstream sediment control?

The objective of this study is to try to provide answers to most of these questions. Unfortunately, most of these can only be answered incompletely or in a qualitative fashion. However, it is hoped that this study will contribute to understanding the nature of the flooding problem on the Vedder River and, as well, provide some general observations on sedimentation processes on gravel rivers.

This study has been subdivided into nine sections. Chapter 2 provides historical and background information on the flood control problem on the Vedder River.

Two of the most important factors governing channel morphology are the hydrological characteristics and geomorphic setting of the basin. These topics are discussed in Chapters 3 and 4 respectively.

In order to understand the depositional processes on the fan, the morphological processes going on upstream of Vedder Crossing must also be considered. Therefore, a brief overview of the morphological characteristics of the Chilliwack River between Vedder Crossing and Chilliwack Lake is presented in Chapter 5.

Chapters 6 and 7 outline the most important hydraulic and sediment transport characteristics of the Vedder River
while Chapter 8 describes the depositional processes that have occurred on the fan over the last century.

Finally, Chapter 9 mentions some possible methods of providing flood control on the fan and makes some comments on their long term effectiveness.

Relatively little discussion has been made on the fisheries resources of the Vedder River in this study. This is because the main emphasis of this thesis is to document the physical processes on the river. However, further discussion of the impact of various flood control measures of the salmon stocks in the Vedder River is contained in the study by Peters (1978).
CHAPTER II
HISTORY OF RIVER ACTIVITY

2.1 Site Location

A large scale map indicating the location of the Chilliwack River relative to Vancouver is shown on Figure 1. In addition, a site plan showing most of the prominent geographical features downstream of Vedder Crossing has also been included in Figure 2. Features on this plan will be referred to frequently throughout this chapter and in subsequent sections.

2.2 Early History and Settlement

The first white man to enter the Chilliwack River basin was Simon Fraser during his exploration of the Fraser River in 1808. However, some of the earlier natural history of the Chilliwack River is recorded in the traditions and legends of the Chilliwack Indian tribes, whose lives were constantly affected by the river's behaviour. Some of this early history is contained in reports by Duff (1952), Wells (1965) and Ramsey (1975).

According to legend at one time the Chilliwack river flowed west below Vedder Crossing into Sumas Lake (Figure 2). During a large flood the river left its course and flowed north, reaching the Fraser River upstream from Chilliwack Mountain. Eventually new channels
were formed as the mouth of the river worked westward, down the Fraser River (Duff, 1952). The Stalo Indians who inhabited the Fraser Valley at this time named this channel "Thewlnum" meaning "river that changed its course." By the time settlers arrived in 1850 the river was split into two channels below Vedder Crossing: the Chilliwack River with its mouth near Chilliwack Mountain and Luck-a-Kuck Creek (Duff, 1952). Unfortunately, the date of this major channel shift is unknown. However, explorations by Francois Annance in 1828 indicated a single major channel did not flow into Sumas Lake (Ramsey, 1975) suggesting the channel shift preceded his visit.

By 1866 settlements had been established at Chilliwack, Sumas and Sardis. In 1873 a second major channel shift commenced when part of the Chilliwack River found its way into Vedder Creek and flowed into Sumas Lake. In 1875 a very severe flood caused the Chilliwack River to abandon its channel and the main flow was diverted down Luck-a-Kuck Creek (LeBaron, 1908). According to Ramsey (1975) the channel of the Luck-a-Kuck increased its width from 30 feet to 200 feet almost overnight washing out all bridges spanning the creek. Eventually the channel shifted again and Vedder Creek began to capture most of the flow until both the Luck-a-Kuck and the old Chilliwack channels were abandoned (LeBaron, 1908).
Figure 3 shows a reproduction of an early map of the Chilliwack area prepared by the Royal Military Engineers between 1868 and 1903, during this period of channel shifting. This map shows the river split into three channels with the Luck-a-Kuck channel apparently taking most of the flow. In 1894 the main channel began to shift once more down Luck-a-Kuck Creek until a log jam formed preventing further diversion. Residents from Sumas who lived near the banks of Vedder Creek tried to break the jam but were prevented by the Chilliwack settlers so that the Vedder Channel became the sole outlet of the Chilliwack River. Although popular accounts of this period suggest residents from Chilliwack aided in the river's diversion down Vedder Creek (Sinclair, 1961; Ramsey, 1975), historical reports indicate the channel shifts were probably entirely natural processes. For example, a summary of the report on the flooding by Colonel Baker, Provincial Secretary, stated

"A log jam had formed in Luck-a-Kuck Creek which prevented the main flow of the Vedder waters from going down the Luck-a-Kuck into Sardis. The Sumas people tried to break up the jam but were stopped by the Chilliwack people. Feelings ran high and both sides threatened to use force of arms." (Annual report of the Lands Service for 1946)

After the flooding of 1894 the Provincial Government tried to stabilize the Vedder River by building a rock filled crib and revetment across the old entrance to
Luck-a-Kuck Creek (LeBaron, 1908). Since this time the river has been maintained in the Vedder Channel so that downstream of Vedder Crossing the Chilliwack River is referred to as "Vedder River."

Aside from winter flooding by the Chilliwack River, early settlers were faced with frequent flooding by the Fraser River. During the summer freshet the Fraser River flooded the lowlands between Sumas and Chilliwack Mountain, backing water up into Sumas Lake, Sumas River and the Vedder River. After severe flooding in 1894, settlers petitioned the provincial government to reclaim Sumas Lake for farmland and to construct dikes along the Fraser River. Almost all engineers involved in reclamation studies agreed the key problem was to convey the Vedder River securely and permanently across Sumas Prairie to the Fraser River. Prior to 1908 all schemes involved diverting the river back down Luck-a-Kuck Creek, thereby taking the river completely out of the diking district. Although this plan would have simplified the reclamation of Sumas Lake, opposition from residents living in Sardis (near the old Luck-a-Kuck channel) prevented the plan from being accepted. Later plans by Brice and Smith (1913) and Sinclair (1918) proposed to divert the river from its course about one half mile east of Sumas Lake and confine it in an excavated channel north across Sumas Prairie and into the Fraser River west of Chilliwack
Mountain (Sinclair, 1961). Eventually the Sinclair plan was adopted and construction commenced on the reclamation of Sumas Lake in 1919. The objectives of the project were threefold:

1. to shut out the Fraser River during freshet from the Sumas Valley and adjoining lowlands
2. to drain Sumas Lake in order to increase the area available for agriculture in the district
3. to channelize the Vedder and Sumas River along their routes across Sumas Prairie to the Fraser River

The Sinclair plan included a series of stream diversions to dispose of internal drainage and construction of the north and south Vedder dikes, extending on both sides of the Vedder River to the B.C. Electric Railway bridge embankment near Yarrow (Sinclair, 1961). The north Vedder dike tied into a dike running along the Fraser River to Chilliwack Mountain while the south Vedder dike extended to a control dam on Sumas River. This dam prevented the Fraser River backwater from penetrating up the Sumas River during the spring freshet. Under normal conditions the Sumas River was passed through sluice gates while during floods the river was pumped over the dam. In addition a smaller dam and pumphouse were constructed on McGillivary Creek to dispose of internal drainage from the East Sumas Prairie.
The major requirements of the Vedder Canal have been:

1. to contain the Fraser River backwater which extends up the Vedder River during the spring freshet

2. to pass flood flows on the Vedder River without scouring its banks or to lose channel capacity due to excessive sediment aggradation

Although the canal was planned to be self-scouring, Sinclair anticipated that movement of gravel in the Vedder River would make it necessary to remove gravel bars from time to time near the entrance of the canal (Sinclair, 1961).

The final canal design called for a channel excavated approximately seven feet into Sumas Prairie, with dikes spaced 500 feet apart at the crest, and a channel slope of approximately 0.00028. The canal was excavated with an electric powered suction dredge while the dikes were constructed from the hydraulic fill pumped from the channel.

The Sinclair plan was completed in 1924 at a final cost of $3,400,000. The total area reclaimed by the project was approximately 33,000 acres, subdivided as follows:

- East Sumas Prairie: 5,000 acres
- Sumas Lake Area: 10,000 acres
- West Sumas Prairie: 15,000 acres
- Area in Washington State: 3,000 acres
2.3 River Training and Flooding Along the Vedder River

Although the original Sumas diking and drainage project was completed in 1924, the north and south Vedder dikes which border the Vedder Canal were never extended past the B.C. Electric Railway bridge. Therefore, although Yarrow has been given some degree of flood protection (provided the dikes and railway embankment are not overtopped), the land upstream of the town has remained relatively unprotected. As a result, this land has been subject to frequent flooding by the Vedder River. In the past, much of this land remained undeveloped or was used for agricultural purposes so that actual flood damages have remained low. However, in recent times there has been a trend towards increased residential development along the river so that demands for improved flood protection have increased.

Attempts at controlling flooding above Yarrow have relied mainly on construction of rip-rapped bank protection and on periodic channel maintenance. This work has been conducted by the federal government (Unemployment Relief during the depression), Provincial Department of Public Works, Department of Highways, Provincial Water Resources Service and District Municipality of Chilliwack (Marr, 1964). For the most part work has been done on an emergency basis whenever flood problems arose or whenever money was made available for flood control works. This
has resulted in very little effective long term flood protection upstream of the railway bridge.

In the early years of channel maintenance most activity went towards preventing the Chilliwack River from reoccupying one of its abandoned channels near Vedder Crossing. A permanent rock-filled crib and revetment was constructed across the entrance to the Luck-a-Kuck Channel in order to stabilize the river in a single channel.

The largest flood recorded at the Water Survey of Canada gauge at Vedder Crossing occurred on December 29, 1917 and reached 27,000 cfs (mean daily flow). However, it appears this flood caused little damage except for washing out the highway bridge near Vedder Crossing. In fact, old newspaper accounts of the flood did not mention any damage to the towns of Yarrow or Sumas:

"The water is higher in the Vedder River than it has been for many years and although the traffic bridge on the trunk road is still standing . . . there was just as much water sweeping past the bridge as under it." (The Vancouver Sun, January 1, 1918)

"B.C. Electric officials were glad to learn the (railway) bridge crossing the Vedder River was damaged to only an insignificant extent." (The Vancouver Sun, January 4, 1918)

Marr (1964) reported that the Vedder River overtopped the north end of the railway embankment in 1932, washed out 1,000 feet of track and flooded Sumas East Prairie. Following this flood, the north Vedder Dike was
extended about 5,000 feet in order to parallel the railway embankment.

According to historical accounts, extensive flooding took place in the Sumas-Yarrow area in 1935, 1948 and 1951 (Sinclair, 1961; Marr, 1964). The flood of January 1935 was one of the most severe in the history of the Fraser Valley and resulted in large areas of Sumas Prairie to be under water (The Vancouver Province, January 25, 1935). However, it appears most of the flood damage was caused by the Sumas River and not the Vedder River. According to Bruce Dixon, Provincial Diking Commissioner at that time, the 1935 flood was due mainly to:

(1) drifts of six to eight feet of snow on Sumas Prairie followed by rain and rapid thaw

(2) an ice jam on the Sumas River sent water over its bank and tore away 300 feet of Sumas dike

(3) the Sumas pumping station was put out of commission when the electric power failed. (The Vancouver Province, January 22, 1935)

Marr (1964) reported the railway embankment was overtopped again in 1948. Based on airphotos taken on July 4 and on newspaper accounts, most of the flooding this time was caused by the record flows on the Fraser River. According to Sinclair (1961), high water levels on the Fraser River overtopped its dikes and inundated much of the low reclaimed Sumas Lake area. High water levels in the Fraser
River would have created a high backwater condition in the Vedder River which could have extended well past the railway bridge at Yarrow. Therefore, this backwater may have been an important factor in the overtopping of the railway embankment.

The flood of February 10, 1951 was triggered by heavy rains, totalling over 11 inches in three days and by mild temperatures which reached up to $54^\circ\text{F}$ at Chilliwack. The Vedder River broke out of its banks at several locations (Figure 5) between Vedder Crossing and the railway bridge, with the overbank flow eventually directed back into the main channel by the railway embankment.

According to newspaper reports:

"The hardest hit point was at Yarrow where a log jam sent the Vedder River off its course over homes, fields and roads. At noon the RCMP broadcast a special alert to the 1,200 citizens of Yarrow telling them to be prepared to evacuate their homes." (The Vancouver Sun, February 10, 1951)

However, the threat to Yarrow was eased when "the B.C. Electric rail embankment north of the town was dynamited to get the Vedder closer to its proper course" (The Vancouver Sun, February 12, 1951). As a result, only minor flood damage was reported along the north side of the river (Marr, 1964).

Following the flood of 1951, proposals were made for a comprehensive flood control project along the Vedder River. In 1952, the Provincial Public Works
Department proposed to construct a 240 foot wide straight channel, bordered by 12 foot high dikes extending from Vedder Crossing to the B.C. Electric Railway bridge. The cost of this project was estimated to be about $500,000 with the provincial government offering a 50-50 cost sharing agreement with the District Municipality of Chilliwack. Although the municipality supported the project, lack of funds caused the eventual abandonment of the scheme. A modified proposal was presented again in 1964 (Marr, 1964), however this project was also abandoned due to lack of financing.

Although a comprehensive flood control project was never carried out on the Vedder River, annual channel maintenance continued under cooperation between the provincial government and the District Municipality of Chilliwack. A summary of some of the work carried out on the Vedder River between 1951 and 1974 is shown in Table 1. During this period channel maintenance seemed to be required most frequently in three areas:

- opposite Peache Road about one mile downstream of Vedder Crossing
- opposite Browne and Lickman Roads
- near the B.C. Electric Railway bridge.

Two main methods of channel maintenance were used on the Vedder River. During low water in fall or winter, snags were removed and bulldozers were used to pile gravel
along the banks in order to establish a well defined channel. This tactic was often unsuccessful as the river frequently adopted a new channel upstream in which case the work was completely bypassed. Alternatively, during emergency conditions, rock wing dams were often constructed in order to divert the river current away from eroding banks. Unfortunately wing dams were liable to initiate erosion on the opposite river bank and also aroused protests from fisheries officials.

Probably one of the earliest cases where fisheries agencies intervened in flood control operations occurred on September 24, 1956. At this time the Department of Fisheries demanded the removal of a wing dam near Browne Road in order to allow salmon access to a side channel. The next major confrontation occurred in 1964 when gravel was dredged from the river's main channel in order to build dikes along Hopedale Slough. According to fisheries officials, the dredging was carried out in an area where approximately 10,000 chum and over 25,000 pink salmon had spawned, and at a time when many of the eggs were still in the gravel. The different attitudes towards flood control and environmental protection expressed by the Department of Fisheries and Municipality of Chilliwack is illustrated below:

"I wish to bring to your attention a recent development which has resulted in the loss of many millions of chum salmon and pink
salmon eggs. This has been the direct result of flood control measures in the lower Vedder River."

W. R. Hourston
Department of Fisheries
March 16, 1964
(letter to the Reeve of the District Municipality of Chilliwack)

"When I think of this river in spate on its mad terrifying plunge through our area, I thank the Almighty for the protection works carried out."

W. G. R. Simpson
Reeve, Township of Chilliwack
March 17, 1964
(reply to W. R. Hourston)

After 1964 the Department of Fisheries appears to have taken a more active role in all flood control works on the Vedder River. For example, in 1967 Fisheries refused to permit the Municipality to dredge 6,000 cubic yards of gravel from the river near Hopedale Road. Due to Fisheries restrictions, most flood control work since 1964 has involved placing riprap along the river banks instead of channel excavation. Fisheries also restricted commercial gravel removal operations on the Vedder River at this time. Prior to 1964 several private companies excavated gravel from an area approximately 1 mile downstream of the railway bridge (P. Heptner, personal communication). At present, Fisheries has granted only one license to remove 5,000 cubic yards of gravel per year near the head of the Vedder Canal.

Over the last 25 years construction of bank protection and increased flood plain encroachment has induced
many obvious changes to the Vedder River (Figure 4). Some of the most common results of these developments have been:

- cutoff of side channels and sloughs from the main channel
- general river straightening by elimination of meanders or bends
- reduction in channel width due to construction of bank protection.

Probably the most drastic modifications to the Vedder River have occurred between the railway bridge and Ford Road. In this 1½ mile reach construction of bank protection has resulted in a reduction in channel width from an average of 670 feet in 1956 to 350 feet in 1969, representing a loss of approximately 75 acres of river channel.

This reclamation of river bottom land has come about through an unusual set of circumstances starting back in 1875 when the Chilliwack River shifted its course down Vedder Creek. Since most of Sumas had been divided into lots by 1858, many property owners lost considerable areas of land when the Chilliwack River shifted. Rather than sell this land back to the government, some residents retained possession of land occupied by the river, in the hope that their property could some day be re-claimed. Therefore residents have tried in the past to
regain as much land as possible by building bank protection to constrict the river. This policy seemed to receive unofficial approval from the provincial government (Kierans, 1959) and the Water Investigations Branch (Marr, 1964). For example, one of the design criteria adopted in a proposed river protection project for the Vedder River in 1964 was:

"to confine the river within a narrow strip of land and possibly reclaim land within the present erosion belt." (Marr, 1964)

Although this project envisioned a series of groins to confine the channel and was never carried out due to lack of funding, parts of the Vedder River have gradually evolved to the general outline suggested in this study.

The flood of December 3, 1975 reached a maximum instantaneous discharge of 27,800 cfs, corresponding to roughly a one in ten year event. Flood waters overtopped the river banks in two main areas - near Webster Road on the north bank and near the railway bridge on the south bank (Figure 5). Although a few homes were flooded between Webster and Lickman Road, most of the water in this area was diverted back into the Vedder River by the northern end of the railway embankment.

On the south bank, flood waters overtopped the bank near Hopedale Road and ponded behind the railway embankment. When the embankment failed, water poured through the town of Yarrow and entered Sumas Prairie,
forcing the evacuation of about 160 people and causing about $770,000 damage (The Vancouver Sun, December 5, 1975).

Surveys conducted by the Water Investigations Branch indicated that approximately 250,000 cubic yards of gravel had been deposited between Vedder Crossing and the B.C. Railway bridge, and that the channel bed had aggraded on average 1 to 2 feet during the flood (Tempest, 1976).

As a result of the 1975 flooding, extensive flood control measures were carried out by the provincial government. After consultations between the Water Investigations Branch, the Provincial Fish and Wildlife Branch, Federal Fisheries Service and Fraser River Joint Advisory Board, a three-phase flood control program was adopted in March 1976. The first phase of this program was to be completed before the 1976 spring freshet and consisted of raising existing banks by three feet, removing gravel from exposed bars above the water line and providing rip-rap protection where required. Work carried out under the phase I program was intended to provide a channel capacity of 19,000 cfs at bankfull stage. The phase II program was carried out in August 1976 and consisted of in-channel excavation of 750,000 cubic yards of gravel in order to increase the channel capacity to approximately 30,000 cfs. The third phase of the flood
control program consisted of a recommendation that set-back dikes be constructed along the Vedder River in order to provide protection against floods reaching 44,000 cfs (200 year flood). It was also suggested that periodic gravel removal operations would be required from year to year in order to prevent deterioration of the channel capacity due to sediment aggradation.

The Phase I and II measures were completed at a cost of roughly $1,300,000 (Peters, 1978), while at the time of writing, the Phase III program has not been implemented.

The flood control measures that were adopted produced strong criticism from environmental groups who were concerned that the Phase II dredging would cause severe damage to the fisheries resources of the river (B.C. Outdoors Magazine, 1976). Although previous channelization and dredging operations carried out in the 1950's and 1960's had undoubtedly reduced the escapement of the river, the Vedder was still an important salmon and steelhead stream. The environmental organizations contended that the most satisfactory solution to providing flood control along the river while still protecting the fisheries resource was to:

(1) construct set-back dikes along the river upstream of the railway bridge
(2) allow the river to return to a more natural condition by removing some of the bank protection that had cut off former side channels and had constructed the channel.

It was felt that if the channel upstream of the railway bridge was widened to its former condition, then the need for extensive channel dredging would be minimized (B.C. Outdoors Magazine, 1976). At present the feasibility of carrying out such a program has not been confirmed.

Peters (1978) has described an analysis to determine the effect of various flood control alternatives on the pink and chum salmon escapement. It is hoped that this present study will provide some answers on the effect of flood control measures on the pattern of sedimentation and flood levels along the river.
CHAPTER III
HYDROLOGY

3.1 Basin Characteristics

The Chilliwack River basin has an area of 474 square miles above Vedder Crossing and a total length of about 45 miles. The river flows in a westerly direction through the northern Cascade Mountains which frequently exceed 8,000 feet in elevation, and enters the Fraser River at an elevation of only about 10 feet above sea level. Approximately one half of the basin area is above an elevation of 3,700 feet and nearly 25% lies above 4,500 feet. Most of the uplands in the basin are mantled by very thin soils overlying bedrock and bare rock is exposed frequently along the higher ridges and mountain peaks. Below an elevation of about 6,000 feet the basin is covered by stands of mixed coniferous timber.

Above Vedder Crossing several large tributaries flow through steep mountainous valleys into the Chilliwack River (Figure 6). Proceeding upstream the largest tributaries include Liumchen Creek, Tamahi Creek, Slesse Creek, Foley Creek, Centre Creek and Nesakwatch Creek. The basin also includes two major lakes - Chilliwack Lake which is located about 30 miles upstream of Vedder Crossing, and Cultus Lake which drains into the Chilliwack River one half mile upstream of Vedder Crossing through Sweltzer River.
3.2 Climate

The climate is characterized by warm dry summers and cool moist winters. The mean annual precipitation recorded near Centre Creek (El. 1,600 feet) is 61.67 inches and the annual winter snowfall is 89.1 inches. The monthly variation in precipitation and temperature is shown in Figure 7.

Since all of the meteorological stations in the Chilliwack basin are located in the valley bottom, it is difficult to estimate orographic effects in the watershed. However, since the mean annual discharge at Vedder Crossing is 2,440 cfs, the corresponding annual depth of runoff is approximately 70 inches/year. Assuming an annual evapotranspiration of roughly 20 inches/year, the mean annual basin precipitation is probably close to 90 inches/year.

3.3 Streamflow Data

The location of all streamflow stations in the Chilliwack basin is shown in Figure 6 and a summary of their period of operation is shown in Figure 8.

Although a gauging station has operated near Vedder Crossing as early as 1911, there is a gap in the record for a period of 23 years, extending from 1932 to 1950. Unfortunately it is known that a number of large floods occurred during this time. Since 1951 the Vedder Crossing gauge has operated continuously and a total of 44 years
of records are available at this station. Gauging stations have also operated on the Chilliwack River below Slesse Creek, above Slesse Creek and at the outlet of Chilliwack Lake.

3.4 Basin Runoff

Some statistics on the discharges recorded along the Chilliwack River are summarized in Table 2. Based on this data, the long term mean discharge varies from 2,440 cfs at Vedder Crossing to 679 cfs at the outlet of Chilliwack Lake, which corresponds to a very uniform runoff of about 70 inches/year throughout the entire basin.

The long term monthly flows recorded at the four mainstream gauging stations are illustrated in Figure 9. The monthly flow regime is typical of many coastal streams in British Columbia, showing four distinct periods:

- spring freshet, extending from April to June and derived mainly from snowmelt
- summer recession, usually beginning in July and reaching a minimum in September
- autumn rise due to increased precipitation and mild temperatures extending from October to December
- winter recession beginning usually in January and reaching a minimum in March due to decreased precipitation and colder temperatures.
A comparison of the flow records at the four gauging stations on the Chilliwack River shows a very consistent runoff distribution along the basin with about 40% of the annual runoff occurring between April-June, 25% between July-September, 20% between October-December and 15% between January-March.

3.5 Flood Hydrology

3.5.1 Flood Generation

Extreme high flows generally occur on the Chilliwack River in two periods - between October-February and between April-June (Figure 10). Autumn and winter floods are caused by heavy rainstorms which develop from warm, moist Pacific low pressure systems. Generally orographic effects cause precipitation to increase sharply with elevation in the basin due to the moist air being forced to rise over the steep mountains which surround the Fraser Valley. Often, additional runoff may be generated by melting of the winter snowpack due to the sudden temperature rise accompanying these storms. Generally winter rainstorms cause characteristically "flashy" floods, having a high intensity but short duration lasting only one or two days.

Spring floods have occurred most frequently in June and usually result from rapid snowmelt due to rising temperatures. However, in some years such as in 1968 combined rainstorm and snowmelt flooding have also occurred.
The peak discharge from snowmelt is governed mainly by the spring snowpack distribution in the basin and on the sequence of daily temperatures during the critical melt period.

The snowmelt flood hydrograph generally shows a much more gradual rise and recession than rainstorm generated floods with high water in some years lasting for several weeks. However, although the discharge volume associated with snowmelt floods is generally much greater than rainstorm floods, the peak flows have been considerably lower. For example, based on discharge records at Vedder Crossing, the largest recorded spring flood reached 15,400 cfs in 1968, whereas at least seven winter rainstorm floods have exceeded this discharge.

In order to compare the flow regimes for the two flood types, the meteorological conditions and discharge hydrographs from a typical rainstorm and snowmelt generated flood are illustrated in Figure 11.

3.5.2 Record of Floods at Vedder Crossing

Table 3 lists all of the major floods that are known to have occurred on the Vedder River. Unfortunately several large floods occurred either before the gauge at Vedder Crossing was established or during the period between 1932-1952 when the gauge was not maintained. Although historical accounts were found describing the large floods in 1875 and 1898, it appears no discharge
estimates were made of these early floods. Also since the rainfall associated with these floods is not known, it is not possible to estimate the flows from the meteorological data. However, it was considered that sufficient data was available to at least roughly estimate the magnitude of some of the ungauged floods which occurred between 1932 and 1952.

Based on historical accounts, and examination of meteorological data and some intermittent stage records at Vedder Crossing, it was concluded that high flows occurred in the following years (see Table 3):

1932 (probably on November 12)
1948 (June 7)
1949 (November 27)
1951 (February 10)

Basically, two main procedures were used to estimate the ungauged flows:

1. by developing a simple correlation with flood flows recorded on the Nooksack River in Washington State
2. by estimating the flows on the Chilliwack River from recorded meteorological data.

The Nooksack River was chosen for developing a flow correlation because it is the only nearby basin with comparable physical and hydrological characteristics as the Chilliwack River. The Nooksack River drains the Cascade Mountains
immediately south of the Chilliwack basin and appears to have a roughly similar exposure and geology, although the relief is somewhat greater. The flood flow correlation was developed by using discharge records collected at the U.S. Geological Survey gauging station near Deming, Washington. The Nooksack River has a drainage area of 580 square miles at this site and has been gauged continuously since 1937.

Figure 12 shows the relation between annual extreme flows recorded at Vedder Crossing and Deming. For each of the years shown on this graph, the peak flows on both rivers occurred less than two days apart. Unfortunately, the recorded flows do not show a unique relationship but instead plot as an envelope curve. Therefore, for a given discharge on the Nooksack River only the approximate upper limit to the flows on the Chilliwack River can be estimated. This suggests that many of the past storms in this area have been quite localized, and have not always extended into both basins.

The U.B.C. watershed model was used for simulating flood discharges from meteorological data recorded at Chilliwack. This model has been described in detail by Quick and Pipes (1976). The computer program attempts to mathematically model the physical runoff processes in a watershed from recorded daily precipitation and temperature data. It was recognized that good flow simulation
would be difficult to achieve on the Chilliwack River due to the very mountainous nature of the basin and to the highly variable precipitation that could result. However, it was decided the results from the model would provide a useful comparison to the correlation analysis.

The model was calibrated from ten months of meteorological and discharge data recorded between March and December 1966. The calibration parameters used in the model were then kept constant for all remaining calculations. In order to check the consistency of the calibration, the model was then used to simulate the December 1975 flood. Figure 13 shows the predicted and recorded discharges at Vedder Crossing throughout the month of December. It can be seen that the model tended to underestimate the flows slightly, with the flood peak estimated at about 17,100 cfs while the recorded peak was 18,700 cfs (McLean, 1975). However, in general, these results were considered very satisfactory given the mountainous nature of the basin.

The model was first applied to the flood of November 1932 using meteorological data recorded at Cultus Lake. The predicted hydrograph and the corresponding temperature and rainfall data are summarized in Figure 14. The predicted peak flow of 14,000 cfs (mean daily) seems low considering the extent of the damages reported by Marr (1964). Unfortunately, the gauge on the Nooksack River was not
operating at this time so that the correlation results cannot be used as an independent check on the model predictions.

The final application of the watershed model was to simulate the flood of February 1951. As mentioned previously, this flood was triggered by heavy rains totalling over 11 inches in three days and by unusually mild temperatures. The predicted hydrograph and corresponding meteorological data are again summarized on Figure 14. Also shown are stage measurements collected at Vedder Crossing. Unfortunately since the gauge had been shifted prior to the 1951 flood and was subsequently washed out, it is impossible to relate these measurements to discharge values. The predicted flood peak reached 20,000 cfs on February 9th or one day before the reported maximum stage at Vedder Crossing. The Nooksack River received major flooding during this storm with the mean daily discharge reaching 34,900 cfs at Deming on February 10th. Based on the results of Figure 14, this would correspond to a discharge of around 18,500 cfs at Vedder Crossing which is quite close to the watershed model results. Therefore it was concluded that the flood of 1951 was probably at least as large as the flood of December 1975.

The watershed model was not applied to the floods in 1948 and 1949 due to a lack of time and because it was originally felt that the flows occurring in these years
were not as great as in 1932 and 1951. According to the stage measurements at Vedder Crossing, the floods in 1948 and 1949 were about 1.3 feet and 0.6 feet below the 1951 stage. Based on current stage-discharge data at Vedder Crossing and assuming the 1951 flood reached about 20,000 cfs, the 1948 and 1949 flows would have been roughly 5,000 cfs and 2,500 cfs lower. However, this assumes the channel control at Vedder Crossing remained stable throughout this period which does not seem very likely. The recorded flows on the Nooksack River were not used to estimate the 1948 summer snowmelt flood since the correlation results shown in Figure 12 were based only on rainfall flood data. However, the Nooksack River reached a peak flow of 24,400 cfs on November 27, 1949 which suggests the flood on the Chilliwack River was probably below 15,000 cfs.

In summary, based on the results from the U.B.C. watershed model, on comparison with flows recorded on the Nooksack River, and on an examination of historical accounts, high flows probably occurred on the Vedder River in the following years:

1932 November 12 probably less than 15,000 cfs
1948 June 7 unknown, probably less than 1949 flood
1949 November 27 probably less than 15,000 cfs
1951 February 10 about 20,000 cfs
It should be noted that all of these flows refer to mean daily discharge values and that the maximum instantaneous values could be as much as 50% to 100% greater (see Section 3.6).

3.6 **Flood Frequency Analysis**

Before carrying out a frequency analysis of floods on the Chilliwack River, it was decided to first assess the accuracy of the reported flows and to examine some of the statistical assumptions that are required in the analysis.

3.6.1 **Accuracy of Reported Flows**

Although a number of large floods have been recorded at the Vedder Crossing gauge, it is difficult to assess the accuracy of these measurements. Water Survey of Canada rates the quality of the gauge records from this station as only "fair." A comparison of all available gauge rating tables at Vedder Crossing showed that a new stage-discharge relation was determined after any major flood and often for each year. Therefore, it is likely that changes in channel control due to shifting of the river bed could make discharge estimates of major floods quite unreliable. Unless actual discharge measurements are carried out during the course of a flood, it is not possible to determine when this shift in control has occurred.
Unfortunately, very few rating measurements have been carried out during high flows at Vedder Crossing and very often long extrapolations of the stage-discharge relation have been required. In fact, prior to 1968, no rating measurements had been carried out at flows greater than 10,000 cfs, which is almost one-third of the reported flood of record. By simply examining the shape of the stage-discharge curves at Vedder Crossing and the magnitude of some of the past rating shifts, it seems likely that some of the reported floods could be in error by as much as ±25%.

Another problem with the gauge records is that prior to 1968 the river stage was not recorded continuously at Vedder Crossing but was read manually from a staff gauge. Although all of the recorded flood flows before this time have been reported as mean daily discharges, in fact, only spot readings may have been measured on the day of the flood. Therefore the reported flows are probably instantaneous measurements which may not correspond to the maximum daily or the maximum instantaneous discharge. If the Chilliwack River were not such a "flashy" stream, there would not be a significant difference between the instantaneous and daily flows. However, during the flood of December 1975, which was one of the few floods to be recorded continuously, the maximum instantaneous discharge (27,800 cfs) was nearly 50% greater than the maximum daily
flow (18,700 cfs). It is believed that such a wide range
in flows over a single day is typical of most winter floods
on the river.

A good example of the difficulty in interpreting
the recorded flow data can be found by examining the flood
of record which occurred in December 1917. This flood is
reported to have reached a maximum daily discharge of
27,000 cfs, which was arrived at by extrapolating the stage-
discharge curve for 1917 to the reported gauge height of
11.5 feet. However, the largest flow measurement used to
establish the rating curve was only 5,840 cfs, having a
gauge height of 5.2 feet. Unfortunately, even the gauge
reading of 11.5 feet is only a rough estimate since the
gauge was washed out on the day of the flood. Therefore,
the flood of record must be considered very approximate.
Unfortunately, the source of errors just described seem
typical of most flood data at Vedder Crossing.

3.6.2 Flood Frequency Analysis

Kite (1977) has outlined some of the most important
assumptions that must be made in carrying out a flood
frequency analysis. These are:

(1) all of the data must describe independent,
    random events
(2) the flood series should not exhibit any long
    term trends over the period of record and must
    be considered stationary with respect to time
(3) all of the recorded flood events should be from the same population and can be used to estimate the population mean, variance and skewness.

The first assumption regarding independence is probably valid since only annual maximum flows were used in the analysis. However, the second assumption of long term stationarity may not be as reasonable. In order to illustrate the long term pattern of annual floods on the Chilliwack River, all of the annual floods at Vedder Crossing were plotted as a time series (Figure 15). Examination of this record shows very clearly that few large floods have occurred in the last 25 years compared to the number in previous years. In fact, the flood in 1975 was the only flood to exceed 18,000 cfs since 1951, while this flow was exceeded at least seven times between 1906 and 1951.

Sporns (1962) noted a similar pattern in the occurrence of severe rainstorms in the Fraser Valley, with unusually intense storm activity occurring in the winter of 1931/32, 1934/35 and 1950/51. Sporns also concluded that

"Storms are not evenly distributed from year to year and . . . a marked cyclical trend in the frequency of occurrence of severe storms seems to be indicated in the period under study."

Therefore, the fluctuations exhibited by the flood record probably reflects a short term climatic cycle which appears to be characteristic of the Fraser Valley. It has been
assumed that such cycles do not represent a permanent change in the behaviour of the watershed so that the complete flood record was included in the frequency analysis. However, these short term cycles may cause periods of high flood activity, alternating with relatively "quiet" periods lasting a few decades and this pattern cannot be predicted.

The final assumption regarding a single population of flood events is unlikely to be strictly correct. It has been mentioned previously that two types of floods may occur on the Chilliwack River - fall or winter rainstorm floods and spring snowmelt floods. Since past rainstorm and snowmelt floods have exhibited very different characteristics, it was believed that the two flood types represent different populations.

In order to decide whether the two flood types have significantly different characteristics, a statistical test was carried out using the procedure described by Kite (1977).

A "Null hypothesis" and an "Alternative hypothesis" were stated as follows:

"null: The rainstorm and snowmelt flood peaks are from the same population of flood events.

alternative: The rainstorm and snowmelt floods are derived from different populations."

The level of significance of the test was computed using the Mann and Whitney test (Kite, 1977). The results
indicated the "Null hypothesis" could be rejected at a significance level of better than 5% so that it was concluded the differences between the two flood types were large enough to indicate two different populations exist. Therefore, it was decided to compute a separate flood frequency relation for each flood type and then determine the annual flood frequency by combining the two relations.

Daily flow records at Vedder Crossing were examined and the maximum daily discharge occurring in each year between April 1 - July 31 and August 1 - March 31 were tabulated (Table 4). The estimated value for the ungauged floods in 1951 was included in order to extend the record, giving 45 years of rainstorm events and 44 years of snowmelt events.

Each set of data was fitted to log Pearson III frequency distribution from the computed sample mean, standard deviation and skewness using the following relation:

$$\log Q = \bar{X} + kS$$

where $Q$ = discharge corresponding to the specified flood frequency

$$\bar{X} = \frac{\sum_{i=1}^{N} \log Q_i}{N}$$

$$S = \frac{\sum_{i=1}^{N} \log Q_i^2 - \bar{X}^2}{N - 1}$$

$k$ = a frequency factor depending on the sample skewness and the probability of exceedance
Although a number of frequency distributions have been used in flood frequency analysis, the log Pearson III distribution has been found to be one of the most reliable methods available and is currently recommended by the U.S. Water Resources Council (W.R.C., 1977).

The log Pearson III distribution simplifies to a log-normal distribution when the sample skewness approaches zero (W.R.C., 1977). The log normal distribution has also been often used in flood frequency analysis and in some cases may give more reliable estimates than the log Pearson III distribution when used on mountainous streams (Stolte and Dumontier, 1977). However, since the computed sample skewness was found to be quite low for both the snowmelt and rainstorm flood data, it was felt that the results from the two distributions would be nearly identical.

The derived frequency relations for the rainstorm and snowmelt data are shown on Figure 16. As expected, the rainstorm floods were found to be significantly larger than the snowmelt floods for nearly all probabilities. For example, the 100 year rainstorm flood was estimated to be 32,000 cfs which was over twice the corresponding snowmelt flood of 15,000 cfs. However, the two frequency relations tend to converge at lower return periods and, for floods with return periods less than 1.25 years, the snowmelt events were slightly larger than the corresponding rainstorm floods.
In order to determine the annual flood frequency relation, the two seasonal flood frequency curves were combined using the method suggested by Kite (1977).

If a discharge \( Q \) has a probability of exceedance \( P_r \) (rainstorm flood) and \( P_s \) (snowmelt flood), then the probability of \( Q \) not being equalled or exceeded is 
\[
(1 - P_r) \cdot (1 - P_s).
\]
Therefore, the probability of \( Q \) being equalled or exceeded during the year is:
\[
1 - ((1 - P_s) \cdot (1 - P_r))
\]

The rainstorm and snowmelt frequency relations were combined in this manner to derive the annual flood frequency relation shown in Figure 16 and summarized in Table 5. It can be seen that the annual frequency curve lies very close to the rainstorm flood curve for floods exceeding about 15,000 cfs. This is because snowmelt floods become so rare at this magnitude that the annual probability of exceedance is nearly the same as the probability of the rainstorm floods.

The preceding analysis requires considerably more work than a standard flood frequency analysis since close to 50 years of daily flow records must be examined in order to pick out the snowmelt and rainstorm flood occurring in each year. In order to check whether this additional effort is justified, a standard flood frequency analysis was carried out using the annual flood series with no attempt made to distinguish between different
populations. The data analyzed consisted of 45 years of records including the estimated flood in 1951 and, as before, the data was fitted to a log Pearson III frequency distribution. The flood frequency relation computed from the annual maximum daily discharge records was compared with annual frequency distribution derived by combining the rainstorm and snowmelt relations. It was found that between return periods of 2 to 10 years the two relations were essentially identical. However, for more extreme events, the standard frequency analysis underestimated the flood flows. For example, using the standard frequency analysis, the 100 year flood was estimated at 28,000 cfs which is about 10% lower than the estimate of 32,000 cfs using the combined snowmelt and rainstorm frequency relations. This difference is not very great considering the large uncertainties and inaccuracies associated with the flow data. On the other hand, the comparison shows that analyzing the rainstorm and snowmelt floods separately and then combining the results may lead to higher estimates of extreme flood values than indicated by a standard analysis of the annual flood records.

All of the estimated flood discharges discussed so far have been mean daily discharges since maximum instantaneous flows have only been recorded at Vedder Crossing since 1968. During this period the ratio of maximum instantaneous to mean daily discharge has varied from about
1.5 for the flood of December 3, 1975 to about 1.04 for the flood of May 24, 1969. Since there is not very much data available on the Chilliwack River, it was decided to examine records from nearby streams between Vancouver and Hope. The ratio of instantaneous to daily discharge was found to vary from about 1.1 to 1.8 for rainstorm floods and from 1.04 to 1.7 for snowmelt floods.

The Nooksack River at Deming, Washington has a long period of instantaneous and mean daily flood flow measurements and was used earlier in this study to estimate missing flood records at Vedder Crossing. A comparison of flood frequency relations based on instantaneous and mean daily flows on the Nooksack River showed the ratio varied from about 1.3 for the 100 year flood to about 1.5 at the mean annual flood. Based on all of the above information, the maximum instantaneous flows on the Chilliwack River were estimated by arbitrarily assigning a ratio of 1.5 to the mean daily flow data. The final adopted maximum instantaneous flows are summarized in Table 6.

With the derived flood frequency relation, it is now possible to estimate the frequency of some of the large historical floods on the Chilliwack River (Table 7). It can be seen that the December 1917 flood of record (27,000 cfs) probably had a return period of about 40 years. By comparison, the flood of December 1975 (18,700 cfs) was only a ten year event. Although the flood of 1975 cannot
be considered an extreme event, it was the largest flood to occur in the last 25 years. However, the lack of extreme floods in recent times should be considered an unusually fortunate circumstance. In fact, the probability of not receiving a flood equal to or greater than the 1975 magnitude in a 25 year period can be estimated as follows:

\[ Q = (1 - p)^n \]

where \( n = 25 \) years

\( p = \) probability of occurrence in one year

\( Q = \) probability of non-occurrence in \( n \) years

Given that the return period of the 1975 flood was about 10 years, the probability of not receiving at least one flood in 25 years is only about 0.07. Therefore, the encroachment which has taken place along the Vedder River in the last 25 years has occurred in an unusual period characterized by very low flood activity. However, if severe storm activity follows the cyclical pattern described by Sporns (1962), then it seems not too unlikely that a sequence of very severe floods could occur in a relatively short period of time in the future.
CHAPTER IV
GEOLOGY AND PHYSIOGRAPHY

The surficial geology of the Chilliwack valley is complex and has received only cursory study by geologists. However, some relevant information was obtained from the following sources:

- geological mapping of the Fraser valley by Armstrong (1959)
- geological mapping of the Columbia valley and Cultus Lake by Easterbrook (1975)
- unpublished B.A.Sc. theses on the geomorphology of the Chilliwack valley by Chubb (1966) and Munshaw (1975)
- unpublished studies on surficial deposits in the Chilliwack valley by Vickerson and Marks.

4.1 **Physiography**

The Chilliwack River flows through two major physiographic divisions - the Cascade Mountains and the Fraser Lowlands. The northern Cascades form a rugged mountain range throughout the basin with peaks frequently exceeding 7,000 feet in elevation. Bedrock consists predominately of granodiorite in the eastern portion of the watershed while west of Centre Creek a complex sequence of sandstones, limestones and conglomerate is found. Through-
out the uplands bedrock is intermittently exposed over widespread areas or covered by a thin layer of glacial drift. Along the valley bottom unconsolidated fluvial glacial and recent fluvial deposits may be found up to depths of several hundred feet (Armstrong, 1959).

The Chilliwack valley and some of the larger tributary valleys show a classic "U"-shaped cross-section indicating the influence of past glaciations. The mainstream valley has a broad flat floor and steep walls with an average depth of 4,000 feet and a width from rim to rim of nearly four miles. Downstream of Chilliwack Lake the valley cross-section shows increased modification by stream activity. Above Chipmunk Creek the glaciated, "U"-shaped valley section is most apparent and the river flows on top of a broad flat approximately one-half mile wide with bedrock bluffs forming the valley sides. Downstream of Chipmunk Creek the Chilliwack River has cut through this plain leaving a series of flat topped, discontinuous benches predominately along the north side of the valley up to 300 feet above river level. Between Tamahi Creek and Vedder Crossing a second discontinuous terrace level is found about 50 feet above the river level and the floodplain width increases until near Cultus Lake it reaches a maximum of 7,500 feet.
4.2 Glacial History

The last continental glaciation in southwest British Columbia, referred to as the Fraser Glaciation, occurred between 12,000 and 25,000 years ago (Armstrong, 1959). This period has been divided into three intervals of glacial advances (stades) and one non-glacial interval (interstade); chronologically the Evans Creek Stade, Vashon Stade, Everson Interstade and Sumas Stade (Armstrong, 1959). The earliest glacial advances widened and steepened the Chilliwack valley and deposited a thin layer of till throughout the uplands.

During the Everson Interstade, valley infilling occurred until ice readvanced throughout southwestern British Columbia when the Sumas Glaciation commenced approximately 11,000 years ago. Most deposits in the Chilliwack valley associated with the Sumas Glaciation are recessional, being laid down during the retreat of the valley glacier. During this period up to 150 feet of sand and gravel was deposited upstream of Tamahi Creek over top of earlier Everson Interstade sediments.

According to Easterbrook (1975), outwash deposits of sand and gravel over 400 feet thick were also laid down throughout the Columbia Valley in Washington State during this recessional period as meltwater from an ice-lobe near Cultus Lake flowed south into Washington State. It is also thought that Chilliwack Lake was formed at
this time when a glacial dam on Post Creek burst, carrying large quantities of boulders down the creek damming the Chilliwack valley (Munshaw, 1975). Extensive boulder deposits found in the channel of the Chilliwack River showing a rough sorting with boulder size decreasing downstream are probably derived from this event.

The end of the Sumas Glaciation has not been accurately dated. However, Armstrong (1959) suggested that the Sumas Ice finally disappeared from the Fraser Valley approximately 10,000 years ago.

Following the end of the Sumas Glaciation, it is believed the climate shifted, becoming warmer and drier than at present (Easterbrook and Rahm, 1970). This period, termed the "Altithermal Period," lasted until about 3,000 years ago. In response to the warmer, drier climate and the eventual decrease in sediment supply, the river dissected the post-glacial valley fill sediments and formed a series of terraces.

Degradation appears to have been arrested near Chipmunk Creek by outcropping bedrock which has formed a knickpoint in the stream profile (Figure 17). As a result, upstream of Chipmunk Creek the river flows on top of the valley fill sediments whereas downstream of this point the river drops through a steep canyon and then flows in an entrenched channel confined by high terraces. The upper terrace level (T1 on Figure 17) is up to 400
feet above the present river level and extends downstream from Chipmunk Creek to near Tamahi Creek. Remnants of a second lower terrace (T2 on Figure 17), approximately 40 feet above the present river level, are also found near Tamahi Creek and near the south end of Cultus Lake.

4.3  **Formation of the Chilliwack River Fan**

According to Armstrong (1959), the Fraser River flowed through the present day Sumas Valley prior to the Sumas Glaciation. The Fraser was probably diverted into its present channel when the advancing glacier blocked the channel and forced the river to flow west around Sumas Mountain.

After the Sumas ice retreated, the Chilliwack River was free to flow through the gap at Vedder Crossing and began depositing its load of gravel and sand on top of the older floodplain deposits. Therefore, the Chilliwack River fan is less than 10,000 years old.

The present Chilliwack fan has a roughly conical shape and extends over an area of approximately 27 square miles between Sumas Mountain and Chilliwack Mountain (Figure 2). The head of the fan at Vedder Crossing is at an elevation of 100 feet above sea level or about 90 feet above the Fraser River floodplain. However, the low terrace near Cultus Lake at an elevation of 140 feet above
sea level provides strong evidence that at one time the
fan head was approximately 30 feet higher than at present.

It is believed that the profile of an alluvial fan
reflects the history of erosion and deposition which has
taken place in the past (Bull, 1964). Figure 18 shows a
number of radial profiles measured down the Chilliwack
fan from Vedder Crossing to the Fraser River floodplain.
These profiles were prepared from 1 meter contour ortho-
photographs and 5 foot contour topographic maps. The
surface shows a gently concave profile which is typical
of many alluvial fans (Bull, 1964; Hooke, 1967; Malcovish,
1974). Based on a simple regression analysis, it was
found that the fan profile could be fitted to the expon-
ential equation:

\[ y = 140 e^{-8.75 \times 10^{-5} x} \]

where \( y \) = fan surface elevation (feet)
\( x \) = distance from Vedder Crossing
\( r^2 = 0.97 \)

It can be seen from Figure 18 that the fan profiles
show an abrupt change in slope approximately one-half
mile below Vedder Crossing. It is believed that this
change in the profile is related to past periods of down-
cutting by the Chilliwack River. When the Chilliwack
River flowed at the level of the low terrace near Cultus
Lake, the fan head would have been graded to approximately
this level. Therefore, when the river began its downcutting, the fan head would have been dissected and the deposition would have shifted from above Vedder Crossing to a point further downstream. Additional evidence of fan dissection such as remnants of the old fan head near Vedder Crossing cannot be found, possibly because the present day river has occupied channels across the entire fan surface.

It is possible to estimate the average long term sedimentation rate on the Chilliwack fan by measuring the volume of sediment laid down in the time interval of deposition. The sediment volume was determined by defining the fan's extent from surficial geology maps (Armstrong, 1959) and airphotos, and then planimetering the area from topographic maps. Unfortunately, the period of active sedimentation cannot be accurately determined. It is believed that the last glaciation ended approximately 10,000 years ago (Armstrong, 1959). However, Church and Ryder (1972) suggested that most post-glacial sedimentation took place in the period immediately following glaciation when the supply of sediment from glacial drift was much greater than the normal sediment supply during non-glacial periods. Ryder (1971) suggested that most post-glacial sedimentation in south-central British Columbia occurred within a 4,000 year period between 10,000 and 6,000 years ago. Using this time period and
assuming a fan head elevation of 140 feet, the average rate of sedimentation on the Chilliwack fan was estimated at 250,000 cubic yards per year. This figure probably represents a minimum estimate since the Fraser River floodplain probably aggraded during this period so that fan deposits below the present floodplain level were excluded from the estimate.

It is assumed that during the Altithermal period (which lasted until about 3,000 years ago) active sedimentation on the fan was low due to the decrease in precipitation occurring around this time. The present day sedimentation on the fan will be the subject of much of the remaining chapters in this thesis. At this time it is worth mentioning that aggradation has recommenced in modern times although at considerably lower rates than in early post glacial times. This can be illustrated by comparing the long term deposition rate in post glacial times with the deposition that occurred during the recent flood in December 1975. According to surveys carried out by the B.C. Water Resources Service, approximately 260,000 cubic yards of sediment was deposited on the fan during the December 1975 flood which corresponds closely to the long term post glacial average. However, the flood of 1975 was an extreme event with a return period of about once in ten years. Therefore, the average present day deposition rate is likely to be much lower than the amount occurring in 1975.
CHAPTER V

RIVER PROCESSES UPSTREAM OF VEDDER CROSSING

Before the river processes on the Vedder fan can be discussed, it is important to have at least a qualitative understanding of the river's behaviour upstream of Vedder Crossing. This is because in the long term the reach between Chilliwack Lake and Vedder Crossing governs the quantity and nature of the sediment supplied to the fan. For this reason the current pattern of sedimentation on the fan may be related to morphological processes occurring much further upstream in the basin.

This chapter attempts to discuss, mainly in a qualitative fashion, the regime of the Chilliwack River. Some of the most important questions that require answering include:

1. What are the major sediment sources to the Chilliwack River fan? Is sediment derived mainly from tributaries sources, from erosion of valley walls and floodplain deposits or from scour of channel bed material?

2. What are the most important river processes occurring on the Chilliwack River above Vedder Crossing at the present time? What are the most important factors which govern the morphology of the river?

3. How far does sediment move through the basin during a single flood event? Can sediment sinks (which
indicate temporary storage areas for sediment) be identified along the river?

4. Has the sediment yield of the basin been significantly altered by activities such as logging and road construction?

Unfortunately, these questions can only be partially answered in this study.

5.1 Data Available

This discussion is based mainly on observations and field data collected during a number of field trips between May 1976 and June 1977. During this period nearly the entire length of river between Vedder Crossing and Chilliwack Lake was visited. In addition, field trips were made to most of the major tributary creeks including Tamahi Creek, Slesse Creek, Chipmunk Creek, Nesakwatch Creek and Centre Creek. Finally, a reconnaissance was made of the entire length of river from an airplane in June 1977.

Additional data was provided from past reports published by the B.C. Water Resources Service (Marr, 1964; Goodyear, 1957) and from Water Survey of Canada Gauging Station records.

Maps and Airphotos of River

Figure 6, referred to previously, shows the entire length of the Chilliwack River below Chilliwack Lake
along with principal tributaries and hydrometric stations.

Most of the river above Vedder Crossing was studied by using topographic maps and high level air photographs, followed by several field trips to inspect areas of interest and to provide "ground truth" for the office study. However, it was decided to carry out a more detailed study between Liumchen Creek and Vedder Crossing due to the rapid channel shifting that had occurred throughout this reach in the past. Therefore, a number of channel overlays were prepared in this reach using successive years of air photographs flown between 1940 and 1976. Further discussion of these overlays is contained in section 5.6.

A list of all maps and airphotos used in this study is summarized in Table 8.

Channel Geometry

The upper Chilliwack River is generally a fast flowing shallow stream that cannot be easily waded even during very low flows. Therefore, channel cross-sections were not surveyed by the author except at convenient bridge crossings and on some of the tributary creeks. Additional data was obtained from earlier surveys by the B.C. Water Resources Service and from current meter measurements at some of the Water Survey of Canada gauging stations. A summary of the available data is as follows:
- 4 cross-sections surveyed by the B.C. Water Resources Service between Vedder Crossing and Liumchen Creek (Marr, 1964)
- 2 cross-sections surveyed by the author from bridges across the Chilliwack River near Centre Creek and Nesakwatch Creek
- 6 cross-sections surveyed by the author at Tamahi Creek and Slesse Creek
- cross-sections and velocity measurements at the following Water Survey of Canada gauging stations:
  Chilliwack River above Slesse Creek (8MH103)
  Chilliwack River below Slesse Creek (8MH55)
  Slesse Creek (8MH56)

Some of these sections have been shown on Figure 19 and the gauge rating measurements are summarized on Figure 20.

**Slope Surveys**

Channel slope surveys were carried out by the author by levelling along the water's edge at several sites along the river. These surveys were carried out on September 1, 1976 when the discharge at Vedder Crossing was 2,550 cfs (about the long term mean).

In addition, a longitudinal stream profile was prepared between Vedder Crossing and Chilliwack Lake using a number of contour maps prepared by the B.C. Water Resources Service (Goodyear, 1957), having scales of
1:7200 and contour intervals of 10 feet as well as 1:25,000 National Topographic Series maps. This profile, referred to previously, is shown in Figure 17.

Bed Material Samples

A total of 36 bed material samples were collected by the author between Vedder Crossing and Chilliwack Lake during the summer and fall of 1976. Most samples were measured either by using the photographic grid or tape method which have been outlined by Kellerhals (1973). Briefly, the two methods of sampling were as follows:

Photographic grid: A square grid with 50 mm spacings was placed over the area to be sampled and a photograph was then taken. The stone underlying each grid point was measured from the photograph.

Tape: A 50 foot tape survey tape was stretched along the sample site and stones underlying each foot mark on the tape were measured.

For each method the intermediate axis of each stone was measured and the results were presented as a cumulative frequency-by-number size distribution.

Also it should be noted that both of these methods describe only the surface bed material size distribution since only the top layer of stones was sampled. In general, much of the bed is armoured, with sand and fine gravel found to underlie the surface layer of cobbles.
In addition a smaller number of bulk sieve samples were collected in order to provide a comparison with the surface samples. Bulk samples were obtained by removing the surface armoured layer of gravel and then collecting approximately 20 pounds of the underlying bed material. The samples were analyzed by sieving and the results were presented as a cumulative frequency by weight size distribution.

A summary of the bed material data is presented in Table 9.

5.2 Analysis of River Processes

Even a brief inspection of air photographs shows the Chilliwack River undergoes several abrupt changes in channel characteristics between Chilliwack Lake and Vedder Crossing. The most apparent changes may be seen in the variation of channel pattern and bed material size along the river. It is believed that such variations may reflect changes in geomorphic setting, sediment supply or discharge regime - the major factors which tend to govern river morphology (Kellerhals, Church and Bray, 1976).

In order to simplify this discussion, the Chilliwack River has been subdivided into five reaches, each displaying roughly consistent channel characteristics. It is assumed that since the morphology is reasonably
constant in these reaches, the factors governing river behaviour are also relatively constant (Kellerhals, Church and Bray, 1976).

The five reaches that have been identified are as follows:

Chilliwack Lake - Chipmunk Creek
Chipmunk Creek - Slesse Creek
Slesse Creek - Tamahi Creek
Tamahi Creek - Ryder Creek
Ryder Creek - Vedder Crossing

The extent of each reach is shown on the long profile in Figure 17, while some of the most prominent morphological features have been illustrated on the series of maps shown on Figures 21.1 to 21.5. In addition, some of the key hydrologic, geomorphic and hydraulic data that have been determined for each reach are summarized in Tables 10 to 12 respectively.

**Chilliwack Lake to Chipmunk Creek**

Throughout this reach the Chilliwack River flows over a stable boulder bed which is characteristic of many lake outlet streams. The river flows in a single channel and has degraded into its former valley floor so that it is continuously confined by low wooded banks.

The channel pattern is very irregular in this reach with the river often deflected across the valley bottom wherever the channel impinges against the valley walls.
Bars and islands are generally very infrequent except in localized areas where the channel width increases abruptly. Although major tributaries such as Centre Creek, Nesakwatch Creek and Chipmunk Creek flow into the river, in this reach alluvial fans have not built out into the mainstream.

Between Chilliwack Lake and Chipmunk Creek the bed material consists predominately of angular boulders and more rounded cobbles (Table 9). Based on lithology and size it seems likely these boulders are derived from the debris which has dammed the western end of Chilliwack Lake. It is also likely that much of present channel consists of relatively stable "lag deposits" which are not transported except possibly in very extreme floods. Therefore, although the river capacity for sediment transport is very large due to the high slope, the river probably carries a relatively small bedload of gravel and cobbles. This is because the supply of sediment that can be transported under existing hydraulic conditions is quite low, being derived mainly from tributaries.

In summary, in this reach the Chilliwack River is flowing on a slope that has been imposed by the glacial history of the valley. Also most of the channel deposits which make up the river's bed material are not derived from present day channel processes but from early post glacial events. At present the channel is generally
stable in this reach with the major channel processes restricted to erosion of fan deposits supplied by tributary creeks. Bank erosion may also occur along the Chilliwack River where log-jams have created major obstructions to the flow.

**Chipmunk Creek - Slesse Creek**

Below Chipmunk Creek the Chilliwack River has dissected its former valley floor, leaving terraces up to 400 feet above the present river level (Figure 17). Close to Chipmunk Creek the river is nearly continuously confined by these terraces, however with increasing distance downstream the valley widens and a fragmentary valley flat can be recognized. At Chipmunk Creek, the river rises to its former valley floor level in a steep bedrock canyon. This canyon represents a stable "knickpoint" in the stream-profile and can be identified on the longitudinal profile and on Figure 21.2.

Throughout this reach the river displays a very irregular channel pattern, reflecting the influence of the valley walls which nearly continuously confines the stream. The river flows in a single channel throughout the reach and very few islands or bars are evident.

In general, the river flows in a cobble and boulder channel with heavily forested gravel and cobble banks. A random sampling of 25 boulders upstream of the Slesse Creek
confluence indicated stones up to 1100 mm (b-axis) are present in the channel while the average size was found to be about 500 mm.

The river slope was found to average around 0.018 between Chipmunk Creek and Slesse Creek and the channel top width ranged from about 70 feet - 90 feet. The only hydraulic data available was derived from the Water Survey of Canada gauging station located about 6800 feet below Chipmunk Creek and is summarized in Table 12 and Figure 20. Although these measurements are probably not representative of the entire reach, they at least provide a rough idea of the hydraulic conditions in the river. These results were used to estimate the threshold conditions for bed movement by means of Neill's competent velocity curves (Neill 1967, RTAC 1978) (See Table 13). Based on Neill's relation a flood of 4700 cfs (2 year return period) having a mean velocity of 7.5 ft/s and a mean depth of 8.5 ft should be able to transport stones up to about 50 mm in diameter. A more extreme flood of 6500 cfs should be competent to move stones up to about 80 mm. Since the bed material contains a considerable amount of material that is much larger than the calculated sizes (by as much as an order of magnitude) it seems likely that much of the bed will remain stable even during severe flood events. Therefore the channel deposits in this reach do not constitute a major source of material that is readily available for
transport as bedload. Consequently, it is expected that the bedload transport rate is relatively low.

The major processes within this reach appears to be widening of the valley floor by erosion of terraces which frequently confine the channel. This erosion has resulted in some fairly large landslides in the north and south valley walls upstream of Slesse Creek (Figure 21.2). This slide material consists predominately of sand and some fine gravel and would be rapidly removed by river erosion.

**Slesse Creek to Tamahi Creek**

Slesse Creek is the largest single tributary in the Chilliwack basin and has a major influence on the hydrology and morphology of the Chilliwack River.

One of the most noticeable effects of the increased discharge below Slesse Creek is the Chilliwack River's sudden increase in channel width. While the average channel width upstream of Slesse Creek ranges from 70 - 100 feet, between Slesse Creek and Tamahi Creek the width varies from about 130 - 150 feet.

In this reach the Chilliwack River begins to develop a more continuous valley flat which is composed of gravel and cobble floodplain deposits. However in some areas such as near Border Creek this valley flat disappears and the river is continuously confined by bedrock.
A short distance downstream of Slesse Creek the Chilliwack River also begins to develop meanders. This pattern varies from regular confined meanders downstream of Slesse Creek to very irregular, almost tortuous meanders above Slesse Creek. This variation in pattern seems to reflect differences in bank material properties - nearly Slesse Creek the river is frequently confined by the valley wall, whereas downstream of Border Creek the river flows against relatively erodible terrace deposits composed of silty-clay.

Frequent gravel diagonal bars and point bars are found in this reach, reflecting the input of sediment from Slesse Creek. The most prominent channel deposits can be found approximately two miles upstream of Tamahi Creek where very large fine gravel point bars have developed. These bars are illustrated in Figure 21.3.

Results of the bed material sampling program indicate the bed size decreases sharply in this reach (Table 9). An average of all samples on the large point bars above Tamahi Creek indicated a $D_{50}$ size of 70 mm while measurements of the confluence of Slesse Creek gave a corresponding size of 140 mm.

Estimates of hydraulic parameters measured at the Water Survey of Canada gauge below Slesse Creek (8MH55) are summarized in Table 12. These results are probably reasonably representative of the upper part of the reach.
near Slesse Creek but not of that part downstream of Borden Creek. A comparison of these measurements with the results from the gauge above Slesse Creek (8MH103) indicates that for the same flood return period the Chilliwack River below Slesse Creek flows within a wider, shallower channel at a higher velocity (hence higher froude number). Estimates of minimum stone size for incipient bed movement are summarized in Table 13. These calculations show stones up to about 80 mm can be moved during floods having a return period of two years while cobbles up to about 150 mm can be moved during more extreme floods with return periods of 20 years. These sizes appear roughly comparable to the size of the bed material measured throughout this reach. Therefore the bed probably becomes active fairly frequently - at least whenever flows exceed the two year flood level. This is quite different from the upstream reaches where much of the bed is composed of stable lag deposits which do not move except possibly during very extreme floods.

In terms of present day processes in this reach, the Chilliwack River appears to be continuing to widen its valley floor as shown by the erosion and slumping along the valley walls. Also in some areas, notably near Slesse Creek, old channel scars can be identified indicating lateral progression of the meanders has taken place in the past. The presence of frequent gravel bars in this reach
reflects the increased supply of sediment to the river from tributaries - with Slesse Creek the most important single source. The very extensive gravel point bars evident upstream of Tamahi Creek represent the only major channel deposits to be found in the upper Chilliwack River. These deposits can probably be considered as an important storage area for gravel-size sediment in the basin.

**Tamahi Creek to Ryder Creek**

Approximately 700 feet upstream of Tamahi Creek the Chilliwack River undergoes another very abrupt change in character. Near this point the river changes from a gravel bed meandering channel and begins to narrow, forming a series of steep rapids containing very large angular boulders up to 1 metre in size. This transition is illustrated on the photo in Figure 21.4 which shows standing waves and rapids just upstream of Tamahi Creek. These boulders may be found in the river channel for at least 1½ miles below Tamahi Creek although both the frequency and size of the boulders tends to decrease with distance downstream. By Osborne Road the channel appears once more to be composed of gravel and cobbles and boulder deposits can not be found.

Bed material samples were taken at the confluence of Tamahi Creek and downstream near Osborne Road. At Tamahi Creek the boulder deposits were sampled by measuring
the intermediate axis of 25 stones. The average boulder size was found to be about 0.9 m with the largest stone up to 1.85 m. In addition a standard tape grid sample was also made on the Tamahi Creek delta near its confluence with the Chilliwack River (Sample 64). This sample gave a $D_{50}$ size of 150 mm and a $D_{90}$ size of 220 mm. By Osborne Road the $D_{50}$ size had decreased to 120 mm and the $D_{90}$ size was 160 mm (Table 9).

The source of these large boulders is not evident, however there are a number of possibilities such as:

1. Moraine deposits
2. Debris carried down Tamahi Creek either by a large slide or by release of an ice dam.

The actual mechanism is probably not too important for the purposes of this study. The main point is that a large "plug" of boulders exists near Tamahi Creek and paves the bed of the river for a considerable distance downstream. These deposits also account for the increase in slope in this reach, which averages 0.0086.

The Chilliwack River channel appears to be quite stable in this reach due to the paving effect of the large boulders. However a comparison of airphotos indicated Tamahi Creek shifted about 250 feet during the flood of 1975. Surveys of the delta show that the old channel was completely filled-in with cobbles and gravel and a new channel was created. It is likely that several thousand
cubic yards of cobbles and gravel would have been dumped into the Chilliwack River during this flood.

Ryder Creek to Vedder Crossing

Downstream of Ryder Creek the Chilliwack River gradually transforms from a stable confined single channel and becomes much wider, developing a split or braided channel pattern. This change becomes most noticeable downstream of Liumchen Creek where a number of gravel bars first become evident in the channel. Opposite Cultus Lake the Chilliwack River flows in a shifting, laterally unstable channel with a valley flat nearly half a mile in width.

The south valley wall effectively confines the Chilliwack River until just west of Liumchen Creek. West of this point the valley wall turns to the south and eventually forms a steep bluff along the eastern shore of Cultus Lake. Therefore downstream of Liumchen Creek the river has been free to migrate laterally over a very wide area.

Detailed examination shows three terrace levels can be identified in this reach:

1. Glacial outwash deposits which confine the channel along the south bank above Liumchen Creek. These deposits extend up to about 500 feet above the present river level.
2. A small terrace about 50 feet above the present river level found along the north side of Cultus Lake. This probably corresponds to a former level of the Chilliwack River in post-glacial times prior to river down-cutting (see Chapter 4 - Geology).

3. A very widespread low terrace of the Chilliwack River approximately 10 feet above the present day floodplain. Many old channel scars visible between the existing channel and Cultus Lake suggest that this level was abandoned in very recent times.

The existing channel has a well developed valley flat that is forested with young conifers and cottonwoods and appears to be subject to frequent flooding.

A number of bed material samples were collected by the author in this reach (Table 9) and although considerable size variation was found between samples, no systematic downstream trends could be detected. Based on these samples, the average $D_{65}$ grain size in this reach was around 60 mm and the corresponding $D_{50}$ size was about 40 mm. All of these samples were collected from gravel bars (mainly diagonal and mid-channel bars) and are probably representative of the material that is being transported by the channel. Considerably coarser sediments could be seen to lie in the main channel however these were not sampled. The bank material was found to consist mainly of poorly sorted sand and gravels roughly similar in size to the
present bed material. The main difference between the bed and bank materials seemed to be in the greater proportion of finer gravels and sands (and greater range of sizes) in the banks.

The average water surface slope at low water between Vedder Crossing and Liumchen Creek was found to be about 0.0063, decreasing to 0.005 near Vedder Crossing.

Unfortunately the only cross sections available to the author in this reach were surveyed by the B.C. Water Resources Service in 1958 and again in 1963. Since considerable channel shifting has occurred since this time these sections are not representative of present channel conditions. However it was felt that this data could at least be used to provide a very rough estimate of bankfull flow in the reach. In addition since a water surface profile was obtained in 1963 during a moderately high flow (6880 cfs) it was felt that reasonable estimates could be made of the slope the channel roughness. Therefore the following analysis was carried out:

1. The water surface slope was computed and found to vary between 0.0052 and 0.0065. This agrees with the surveys carried out by the author in 1976 at low flow.

2. Using the surveyed water levels and the discharge of 6880 cfs Manning's "n" values were computed at each section and were found to vary between 0.0035 - 0.0042.

3. The cross-sections were then subdivided into main
channel and overbank sub-sections. Overbank sections were assigned roughness values of between 0.06 - 0.10 (based on survey descriptions and airphoto inspection) while a value of 0.038 was used for all the main channel sections.

4. The discharge was then computed from the Manning's equation \( Q = \frac{1.49 AR^{2/3}}{n} S^{1/2} \) for various river stages assuming uniform flow.

The calculations indicated that the bankfull discharge was between 9000 - 11000 cfs, which corresponds to a flood with a return period of around two years. These results are in general agreement with other studies on gravel bed rivers (Henderson, 1966; Bray, 1972). It was also found that once the bankfull level is exceeded the river stage-discharge relation becomes very flat, with only a slight increase in stage for even severe floods. This is because once bankfull stage is exceeded, the channel width becomes very large thereby increasing the channel area. As a result, it appears that the river stage does not exceed the level of the low terrace in this reach even during extreme floods. The results from these calculation are summarized in Table 12.

A comparison between the 1958 and 1963 cross sections showed that considerable channel shifting occurred around this time, with some tendency towards deposition (Figure 19). The average bed level change was computed at each section as follows:
1. The net area between the channel bottom in 1958 and 1963 was planimetered.

2. The average bed level change was then taken as the net area divided by the bottom width. The result indicated that three out of four sections show net aggradation had occurred in this period, with only the very narrow section near Vedder Crossing showing net degradation.

As far as the author is aware, no cross sections were surveyed in this reach following the 1974 flood. However, based on some field observations and airphoto comparisons, it appeared that many new gravel bars were formed as a result of this flood. Therefore, there is some evidence (although not conclusive) to indicate that the river may be now slowly aggrading in this reach.

The lateral instability of this reach is most marked between Vedder Crossing and Liumelen Creek and is characterized by sudden channel shifts termed "avulsions". These channel shifts were documented by comparing historical maps and air photographs.

The earliest map of this reach was dated 1886 and was obtained from the archives of the B.C. Electric Company. Although, the general river alignment appears to have been close to present day channel, the map did not show sufficient details to make any further conclusions.

Other old maps were found in the B.C. Electric
Company archives dating back to 1905, and 1910, drawn at scales of 1 in = 5880 feet and 1 inch = 1320 feet respectively. These maps showed the Chilliwack River had roughly the same alignment as at present, although the channel may have been wider. For example, the 1905 map showed the Chilliwack River in the vicinity of Sweltzer Creek had a width of 1200 feet, while the 1910 map showed that near Liumchen Creek the width varied between 2100 feet and 1000 feet. Although the present-day channel is comparable to the 1905 map near Sweltzen River, it is now roughly half the width shown on the 1910 map near Liumchen Creek.

The earliest airphotos obtained by the author were flown in 1940, with additional coverage following in 1952, 1958, 1966, 1969, 1971 and 1976. Some of these photos have been reproduced in Figure 22 in order to illustrate some of the channel changes that have occurred throughout this reach. In addition, maps were made from each of the photos at a common scale of 1 inch = 1300 feet and these are shown in Figures 24. The average active channel width was computed from each map over a distance of 10,000 feet extending from Vedder Crossing to Near Liumchen Creek. The average channel width was determined by planimetering the outline of the channel bottom in this reach and then dividing by the total reach length. Results of these computations are shown in Table 14.
Two noticeable changes have occurred since 1940.

1. Over the years the active channel width has narrowed appreciably in this reach, decreasing from 760 feet in 1940 to 560 feet in 1968. This trend was partially reversed by the flood in 1975 which caused considerable local bank erosion along the north side of the river. Most of this narrowing has resulted from apparent flood-plain re-construction on the south side of the river, with large areas formerly occupied by channels becoming very densely vegetated.

2. The channel pattern has changed from an essentially braided channel to a split channel with frequent wooded islands. This pattern change is most noticeable by comparing the airphotos of 1940 and 1971 (Figure 22).

The explanation for this apparent change in river characteristics is not known. However, these changes may reflect the changing pattern of flooding that has occurred over the last 80 years. It has been mentioned previously that there has been a number of severe floods in the first half of this century, followed by an unusually low incidence in the last 30 years. In fact, between 1951 and 1973 when most of the channel changes took place, the two largest floods to occur in this period had return periods of only about 5 years.

It is expected that a long duration without large floods could allow former channel areas to become re-vegetated
with heavy brush and willows and eventually become part of the floodplain or develop into wooded islands. As a result the flow would tend to become concentrated in two or three main channels causing the river to become somewhat incised and to develop a more split appearance. In addition when bankfull conditions were exceeded it is expected that at least some of the suspended load would be trapped by the new vegetation and deposited overbank. Such channel changes can certainly not be ruled out after comparing the cross sections surveyed in 1958 and 1963 (Figure 19) which seem to show overbank deposition going on in conjunction with thalweg deepening.

The maps shown in Figure 23 were used to prepare overlays in order to compute the areas of apparent floodplain reconstruction and erosion. These overlays are shown in Figure 24, while the results of the comparisons are summarized in Table 15.

It should be mentioned that the areas given as "floodplain reconstruction" should not be considered as areas of deposition. These areas were delineated simply by noting where vegetation had become established on the former channel surface. Therefore, this does not necessarily imply that actual sediment deposition has taken place in these areas. On the other hand, it appears that most of the areas of apparent erosion occurred in terrace or bank deposits so that large quantities of sediment would have
been removed during these channel shifts.

In general, the lateral shifting on the Chilliwack River has followed a systematic pattern, with apparent floodplain reconstruction on the south bank and erosion on the north bank. The maximum observed erosion rates estimated from the overlays ranged from 620 feet between 1958 and 1971 (averaging 48 feet/year) to 300 feet between 1971 and 1976 with most of this erosion occurring during the 1975 flood.

In terms of areas of land eroded, between 1940 and 1951 the average annual erosion rate was about 2.5 to 3.0 acres per year. This corresponds to a total of about 91 acres of land lost between Vedder Crossing and Liumchen Creek during this period. By comparison, between 1971 and 1976 about 47 acres of land was lost, with probably nearly all of this occurring during the 1975 flood.

It is possible to roughly estimate the volume of material eroded from this reach if an average bank height can be assigned to the eroded land areas. Based on an examination of presently eroding banks and on the cross sections surveyed in 1958 and 1963 it was decided a reasonable height would be about 10 feet.

The resulting sediment volumes shown in Table 15 should probably be considered order of magnitude estimates. This is because there are clearly large uncertainties involved both in preparing the original overlays and in
assigning a representative bank height. Regardless of these uncertainties, it is believed the estimates are reasonable and can be used to at least roughly indicate the amount of sediment that has been eroded by the river in this reach. It should also be noted that the volumes and areas shown in Table 15 were computed from overlays which extended only to Liumchen Creek. Therefore, any erosion occurring upstream of this point has not been included.

The results in Table 15 show that between 1940 and 1971 the erosion rate averaged about 50,000 cubic yards per year, and that approximately 637,000 cubic yards of material was eroded between 1971 - 1976. By comparison, surveys by the B.C. Water Resources Service indicated about 258,000 cubic yards of sediment was deposited between Vedder Crossing and the B.C. Electric Railway bridge during the 1975 flood. Therefore the amount of material eroded in the two mile reach above Vedder Crossing during 1975 was approximately twice the amount deposited onto the fan. This may indicate that some of the material was re-deposited upstream of Vedder Crossing or was transported downstream of the railway bridge into the Vedder Canal. Alternatively, this discrepancy may reflect the inaccuracies involved in preparing the overlays and in assigning a representative bank height. However, these results demonstrate that bank erosion immediately upstream of Vedder Crossing
constitutes one of the most important sources of sediment to the fan.

5.3 The Sediment Supply of the Chilliwack River

Although bank erosion in the reach above Vedder Crossing is clearly an important sediment source, there are other important sources in the basin.

Most of the sediment deposited on the fan is probably derived from three sources:

1. Tributary creeks
2. Landslides in terrace deposits
3. Bank erosion of channel and floodplain deposits

Based on field observations and airphoto interpretation it appears Slesse Creek is the target tributary source in the Chilliwack basin. The effect of this sediment influx on the Chilliwack River is very apparent, with the river's bed material size changing from boulders to cobbles and gravel and prominent point bars occurring. Other important creeks supplying sediment to the Chilliwack River probably include Nesakwatch Creek, Tamahi Creek and Liumchen Creek.

Several large landslides have occurred in the high terraces along the Chilliwack River upstream of Slesse Creek. These terraces are composed predominately of sand so this material would be rapidly eroded by the river and could probably be carried in suspension. Since the volume
of some slides has been in the order of hundreds of thousands of cubic yards, these events could constitute important sources of sand size material when they occur. It would appear simply from visual observations, that some of these slides have been aggravated by logging road construction.

As mentioned previously most active bank erosion occurs between Vedder Crossing and Liumchen Creek. Much of the river above Liumchen Creek is flowing on top of very coarse boulder deposits which appear to be stable even during major floods. Therefore, except for the reach immediately upstream of Vedder Crossing, the supply of sediments from bank and channel erosion is likely to be relatively small.

In determining the relative importance of these sediment sources, it is important to specify the time frame being considered.

In the short term (say a single flood event) it seems likely that bank erosion from the reach immediately upstream of Vedder Crossing is the single most important source of sediments for the fan. The simple calculations presented earlier clearly demonstrate that the volume of material eroded in this reach is comparable with the amount of deposition that has occurred downstream of Vedder Crossing.

It should also be noted that during a single flood event, transport of the gravel and cobble size bedload may
persist for a relatively short period of time, ranging from perhaps a few hours to several days per year. This can be confirmed by reviewing the calculations of threshold conditions for sediment movement which were presented earlier (Table 13). For example, based on Neill's competent velocity curves it was found that below Slesse Creek flows of 6500 cfs (corresponding to a two year flood) could move gravel up to about 80 mm. By examining the daily flow records in this reach it appears on average such flows may occur three to four days per year. Since bedload moves intermittently, at much slower spreads than the water velocity it seems doubtful that coarse material could move from the upper parts of the basin to the fan in a single flood event. More likely, coarse sediment probably moves a few miles in each flood, with sediment being deposited in localized reaches along the river. The extensive gravel bars that have developed at the meanders upstream of Tamahi Creek may correspond to one of these temporary sediment "sinks". This concept of localized sediment transport and deposition from reach to reach is illustrated in Figure 25.

There is little direct quantitative data to confirm this model of sediment movement. However, experiments have been conducted on other gravel rivers which provide some support. For example, Mosley (1978) carried out tracer studies on the Tamaki River in New Zealand and
concluded that the rate of movement of a slug of bed material was quite slow, with the peak travelling about 1.75 km/year. Of course the finer sand and silt sized material will move in suspension and may be swept through the basin in a single flood event.

Over a time frame lasting years or decades, sediment from the upper reaches of the Chilliwack River will move through the basin and be deposited below Vedder Crossing. This long term pattern sediment movement will determine whether the reach directly upstream of Vedder Crossing undergoes degradation or aggradation. For example if the supply of sediment from upstream does not balance the quantity of material removed from channel and bank erosion in this reach then degradation will take place.

In summary the actual pattern of sediment movement between the upper reaches of the Chilliwack River, the reach immediately upstream of Vedder Crossing and the fan below Vedden Crossing is obviously very complicated.

For example, it has been shown previously that when major floods are absent the bank erosion rates decrease, some of the channel deposits tent to become vegetated and the river pattern shows a tendency to develop a more incised, split channel. As shown in Table 15, during the period between 1952 and 1971 the volume of material eroded in the two mile reach above Vedder Crossing averaged about 30,000 cubic yards/year. During this period it is likely
that other sediment was supplied to the fan from upstream sources and from scour associated with the channel incision that occurred in this reach.

During major floods, such as in 1975, very large volumes of sediment are made available for transport due to bank erosion and channel shifting near Vedden Crossing. The sediment transport capacity of the river then determines how much of this material will be transported below Vedder Crossing onto the fan and how much will be re-deposited upstream. In addition some sediment from upstream sources may be transported through this reach onto the fan, and some may be deposited above Vedder Crossing in the channels that have been abandoned during the shifts. Since much of the erosion occurring in this reach appears to be aggravated by log jams it is impossible to estimate the quantity of material that will be supplied to the river during a given flood. However, the transport capacity of the river at Vedder Crossing, which governs the amount of sediment that will be transported onto the fan, can be estimated by sediment transport theory. Much of the remaining chapters of this study will be devoted to this subject.
CHAPTER VI

CHANNEL PROCESSES BELOW VEDDER CROSSING

The Vedder River has been studied by several agencies over the last two decades. Since much of the data collected during these past investigations has been used in this present study, a brief discussion of the data is warranted.

6.1 Data Available

Maps and Airphotos

Many of the historical maps and airphotos mentioned in Chapter V also show portions of the Vedder River. The earliest map of the river was surveyed between 1863 and 1902 and has been presented in Chapter II (Figure 3).

The airphoto coverage of the Vedder River is excellent, with the earliest photos dating back to 1930. The river has been reflown frequently since this time, in 1940, 1948, 1952, 1958, 1966, 1968, 1969, 1971, 1974, and 1976. Some of these photos have been summarized in Figure 4.

Two sets of detailed topographic maps of the surrounding floodplain were used in this study. The earliest map from Marr's 1964 report showed spot elevations to the nearest foot. In addition a 1976 orthophotograph was
obtained from the B.C. Water Resources Service, having a contour interval of 0.5 metres.

**Channel Surveys**

River cross sections have been surveyed by the B.C. Water Resources Service in 1958, 1959 and 1963. Twelve of these sections, situated between the outlet of the Vedder Canal and Vedder Crossing were obtained from Marr's 1964 report. In addition, surveys were carried out in 1975 and 1976; however, most of these were made at different locations from the earlier surveys. The data made available to the author consisted of:

- 12 sections surveyed in the late summer of 1975 and shortly after the December flooding, located between the railway bridge and Vedder Crossing.
- 20 sections surveyed after completion of the Phase I flood control program.

Additional cross sections were surveyed by the Water Survey of Canada between 1971 and 1975 as part of their sedimentation study on the river. Unfortunately, none of these sections were aligned with any of the B.C. Water Resources surveys. The surveys between 1971-1973 were obtained from preliminary reports by Tywoniuk (1971, 1972, 1973). In 1975, 16 additional surveys were made, all downstream of the railway bridge.
In order to avoid confusion, all of the sections have been given a mileage designation, measured upstream from the Trans-Canada highway crossing of the Vedder Canal near its confluence with the Fraser River. The location of the sections are shown on Figure 26 and all of the sections are summarized in Table 16. A few representative sections have been shown in Figure 27.

**Water Level Data**

The cross section surveys can be used to construct water surface profiles along the river. However most of these surveys were carried out at relatively low flows (around the long term mean), with the exception of a 1963 B.C. Water Resources water surface profile which was surveyed during a flow of 6880 cfs (Marr, 1964). The author carried out a number of slope surveys during flows which ranged between 4000 cfs and 6000 cfs. However these were made around the time when dredging was being carried out which could have affected the water profiles. The results of these surveys and the estimates from the agency cross sections are summarized in Table 17.

Aside from surveys, water levels have been recorded at three Water Survey of Canada gauging stations. These stations are

- Vedder Crossing (stage and discharge)
- below the railway bridge near Yarrow (stage 1952-1974)
- Sumas River near Sardis (stage 1951-1972)
Although these stations are not sufficient to determine a water surface profile along the river, they provide a considerable amount of data on flood levels and hydraulic processes.

**Bed Material Data**

Bed material data has been collected by Water Survey of Canada (W.S.C.) as part of their sediment investigation on the Vedder River and by the author. The Water Survey of Canada data was collected in 1973 and 1975 and consisted of 63 bulk samples taken between the Vedder Canal and Vedder Crossing.

Most samples ranged between 12-25 pounds and all were taken on W.S.C. cross section lines. The location of each sample on the cross section was specified in most cases.

In addition the author collected 21 samples in order to supplement the Water Survey of Canada data. Most of these were tape grid or photographic grid samples although a few bulk samples were also collected.

6.2 **Channel Pattern of the Vedder River**

In comparison to most rivers in B.C., the Vedder River is a very new stream - being only about one century old. Therefore throughout most of its life the Vedder
channel has probably been in a state of transition as it tried to adjust to the new flow regime and sediment load imposed on it.

The map of the Chilliwack area prepared around the turn of the century (Figure 3) shows the Chilliwack River split into three major channels a short distance below Vedder Crossing, with the Luck-a-Kuck and Chilliwack channels appearing to carry most of the flow. At this time the Vedder River had a relatively straight channel, frequently split by bars or islands, with a width varying from a maximum of about 400 feet near Vedder Crossing to between 75-100 feet near Sumas Lake. It is believed this map was prepared only a few years after the Chilliwack River began its main period of channel shifting in 1875.

Le Baron, in 1908, described the Vedder River as:

"having a width varying from 60 feet to 400 feet at low water and full of rapids and gravel banks. It is also much choked by fallen trees and logs which appear to have been uprooted when the river broke out into its present channel about 15 years ago. Hermon and Burwell, Civil Engineers state . . . the river is continually widening."

By 1930 the river had a very braided unstable channel with an active channel width varying from about 450 feet near the railway bridge to about 1600 feet opposite Webster Road (two miles below Vedder Crossing). The upper two miles of the channel appeared most unstable, while towards the railway bridge the channel appeared relatively incised.
This downstream decrease in width is most apparent on the June 1948 airphoto which taken at close to bankfull condition (Figure 4).

A comparison of the 1930 and 1948 photos shows the overall channel alignment had remained quite constant in this period. Therefore the channel widening referred to in Le Baron's 1908 description appears to have ceased approximately fifty years after the Chilliwack River avulsion. However localized bank erosion continued and is most evident between 1948 and 1951 where in some areas along the south bank the channel shifted up to 470 feet. It is believed most of this erosion occurred during the February 10th, 1951 flood.

Between 1952 and 1975 two main channel changes are apparent:

- extensive channelization of the river between the railway bridge and Ford Road;
- floodplain reconstruction between Webster Road and Vedder Crossing. Also, the river appears to have changed from a very unstable braided channel to a more incised split channel.

The channelization of the river has been described previously in Chapter II. This work, carried out in the late 1950's and 1960's resulted in a loss of channel area of about 75 acres. As a result the active channel width
decreased from 670 feet to 350 feet between the railway bridge and Ford Road.

The channel changes near Vedder Crossing appear to be mainly natural although some river training occurred in this area as well. This change in channel pattern may indicate the river had begun to "settle down" after a long period of channel instability following the initial channel shift in 1875. The long period of relatively moderate floods starting in the early 1950's and extending up to 1975 would probably have also contributed to this change in pattern. Therefore the existing channel pattern of the Vedder River can be described as being straight, split or slightly braided along its upper half becoming confined to a single channel through its channelized reach and the Vedder Canal.

Several different bar types could be distinguished along the river including large midchannel bars, side bars and diagonal bars. The diagonal bars form a regular series of "riffles" which extend through the channelized reach above the railway bridge (Figure 4). These riffles appear to be spaced roughly four channel widths apart (based on 1971 airphotos). Throughout the Vedder Canal a very striking sequence of alternating side bars can be seen during low flow conditions. The "wavelength" of these bars was found to average 1930 feet over the length of the
canal which corresponds to about 12 bankfull channel widths. Therefore the spacing between cross-over points is half of this amount. These alternating bars in the canal and diagonal bars in the constriction tend to produce a sinuous flow pattern during low flows and suggest the channel is attempting to develop meanders.

It is apparent that the channel pattern of the Vedder River has not remained constant over the past century, nor is it constant over the entire river length.

The upper half of the river from above the railway bridge to Vedder Crossing has displayed a laterally unstable channel with poorly defined banks and frequent islands and bars. This type of braided pattern is commonly associated with two conditions:

- streams transporting high sediment loads on steep slopes (Henderson, 1966). Such channels may develop because of the high shear on the banks which can cause channel widening and subsequent deposition in the centre of the stream.

- deltaic environments and alluvial fans where the sediment transport capacity decreases in a downstream direction (Church, 1972). Leopold and Wolman (1957) also associated braiding with channel aggradation.

In braided channels it is often difficult to distinguish
the bankfull level of the channel from the inactive floodplain and from low terraces. Terraces are remnants of former river deposition that have been subsequently abandoned by river down-cutting. As a result terraces may not be flooded even during extreme floods.

Inactive floodplains are often scarred with old abandoned channels and former bars, however they are often covered by dense vegetation. Inactive flood plain may become reoccupied during floods as a result of channel shifting. In the case of the Vedder River, nearly the entire surface of the alluvial fan could be considered inactive floodplain.

In braided rivers the active channel can be very wide and shallow. In this case the bankfull level normally corresponds to the top of the active bars. Table 8 lists some properties of the braided channels on the Vedder River. Most of this data was derived from the 1958 and 1963 cross section surveys since at this time, the channel was relatively unaltered by human activities. A few cross sections surveyed in 1975 near Peache Road were also included since this location has remained relatively undisturbed. In all cases only the major subchannels at each section were analysed. These results show the average bankfull depth was only about 3.1 feet. It was also found that the inactive floodplain, which is currently being developed and built on, is at nearly the same elevation
as the tops of the bars in the active channel. This feature is illustrated on some of the fan cross sections shown in Figure 28.

Downstream of the railway bridge the river flows in a single channel with few islands or bars. This reach has been so altered by channelization and dike construction that it is very difficult to determine the river's natural channel pattern, although early maps showed the river to be meandering.

Leopold and Wolman (1957) determined an empirical relation to discriminate between braiding and meandering on the basis of slope and discharge:

\[ S = 0.06 Q^{-0.44} \]

If the observed slope was found to be greater than the computed value, the channel was found to be braided, whereas if the measured slope was less than this value the channel was found to meander. Using a mean annual flood as a dominant discharge, the critical slope between braiding and meandering was estimated to be about 0.001. This would correspond to a location somewhere between the railway bridge and the head of the canal which agrees reasonably closely to the observed transition.

6.3 Channel Hydraulics

The river surveys reported previously show the slope
of the channel is relatively uniform from Vedder Crossing to near Webster Road and averages around 0.005. Downstream from this point the profile gradually flattens out, averaging around 0.0035 near Ford Road and 0.002 near the railway bridge.

The Vedder Canal is subject to backwater from the Fraser River so the slope in this reach is variable. To make matters more complicated, tidal influences can extend up the Fraser River past Chilliwack Mountain, so diurnal fluctuations can occur in the canal at some flows. In addition, the Sumas River enters the Vedder Canal and therefore has some influence on the river's stage.

As mentioned earlier geodetic water levels have been recorded near the outlet of the Vedder Canal between 1951 and 1972. The stage measurements in the canal were correlated with discharges recorded on the Chilliwack River at Vedder Crossing and the Fraser River at Hope (Figure 29). These results show that for a discharge of 10,000 cfs at Vedder Crossing, the stage in the canal can vary by over 15 feet, depending on the flows in the Fraser River. Furthermore, during the winter the Fraser River is generally low and insensitive to winter rain storms. Therefore during a winter flood on the Chilliwack River, a steep drawdown type profile should occur throughout the canal.
In the summer, when the Fraser River freshet occurs the high water levels at the canal outlet should cause a classic "M-1" type backwater profile in the canal and lower Vedder River.

The effect of these water level fluctuations on the slope through the canal can be roughly determined by comparing the water levels recorded at the railway bridge and the canal. Two extreme combinations in the period of record were considered:

- High flow on Chilliwack River (15,900 cfs)
- Low flow on Fraser River (110,000 cfs)
  average slope = 0.00058

- Low flow on Chilliwack River (5,170 cfs)
- High flow on Fraser River (408,000 cfs)
  average slope = 0.0001

The extent of the backwater in the Vedder Canal will depend on the flow in the Vedder River and in the Fraser River. The stage records near the railway bridge were scanned and it was determined that when the flow at Vedder Crossing was 5,000 cfs, the water levels at the railway guage were not affected by the Fraser River even when flows at Hope reached 400,000 cfs. Therefore except perhaps under very rare conditions, it can be concluded the Fraser River backwater does not extend past the railway
Hydraulic measurements have been carried out at only two sites along the Vedder River:
- at the gauging station near Vedder Crossing
- at the cableway just below the railway bridge near Yarrow.

All of this data was obtained from Water Survey of Canada and the average channel properties (top width, mean depth and mean velocity) were plotted against discharge as shown in Figure 30.

The data was found to plot as straight lines on log-log paper so that the hydraulic geometry could be expressed as power law relations. The derived expressions were:

\[
\begin{align*}
V &= 0.044 \ Q^{0.55} \\
W &= 185 \ Q^{0.559} \\
d &= 0.123 \ Q^{0.40} \\
V &= 0.183 \ Q^{0.430} \\
d &= 0.041 \ Q^{0.559} \\
W &= 134.3 \ Q^{0.011}
\end{align*}
\]

These results were derived from discharges ranging between 3,070 cfs and 12,400 cfs at the railway bridge and 1,770 cfs and 18,800 cfs at Vedder Crossing. The maximum velocity reported by Water Survey of Canada was 14 ft/sec.
at Vedder Crossing with a corresponding Froude number of about 0.75 and 6.8 ft/sec at the railway bridge, with a Froude number of 0.55.

The exponents of the power law equations indicates the variability of each parameter with discharge (Leopold, Wolman and Miller, 1964). At both sites the width is very insensitive to discharge while the velocity increases the most rapidly at the railway bridge and depth increases most rapidly at Vedder Crossing. These results can be used to indicate how the channel resistance changes with discharge. Since Manning's coefficient should be proportional to \( \frac{d^{2/3}}{v} \), the power law equations indicate:

\[
\begin{align*}
    n &\propto Q^{-0.28} \quad \text{(below the railway bridge)} \\
    n &\propto Q^{-0.06} \quad \text{(at Vedder Crossing)}
\end{align*}
\]

This shows the channel resistance decreases with increasing discharge near the railway bridge, but is nearly constant at Vedder Crossing. Values of Manning's coefficient estimated from the hydraulic measurements confirm these trends. At the railway bridge "n" values were found to decrease from about 0.034 at 5000 cfs to about 0.025 at 12,500 cfs. At Vedder Crossing the computed n values scattered between 0.04-0.045 and showed no consistent variation.

Channel resistance normally results from two factors:
- "skin" resistance due to the rough channel boundary
- "form" resistance due to bars, changes in channel
alignment or channel obstructions.

In gravel rivers form losses are normally considered to be small (Rouse, 1965). For hydraulically rough flows skin resistance can be estimated from the bed material composition by means of Strickler's empirical relation (Henderson, 1966).

\[ n = 0.034 \text{ D}_{90}^{1/6} \]

where \( D_{90} \) represents the bed material roughness height.

This expression indicates the minimum expected Manning's \( n \) values would be around 0.026 at the railway bridge and around 0.029 at Vedder Crossing.

If the expression for Manning's coefficient is substituted into Manning's equation \( V = \frac{1.49}{n} \frac{d^{2/3}}{s^{1/2}} \) a dimensionless resistance formula can be derived:

\[ \frac{V}{V^*} = 8.4 \left( \frac{d}{D_{90}} \right)^{1/6} \]

Two other resistance equations which can be expressed in a similar dimensionless form are Keulegan's logarithmic formula (Burkham and Dawdy, 1976) and Limerinos' modification of this formula (Limerinos, 1970). These are, respectively:

\[ \frac{V}{V^*} = 6.25 + 5.75 \log \left( \frac{d}{D_{90}} \right) \]

\[ \frac{V}{V^*} = 3.28 + 5.75 \log \left( \frac{R}{D_{84}} \right) \]

All three of these expressions have been plotted in
Figure 31, along with the measurements made below the railway bridge. The measured data scatters around the Limerinos equation, however it appears to plot at a steeper slope than any of the equations predict. These results show that for low depth/grain size ratios the measured channel resistance was higher than the predicted values, indicating that significant form losses were present. When the $\frac{d}{D_{90}}$ ratio exceeded about 15 the measured values began to approach the theoretical results indicating form losses were decreasing. This is quite reasonable since at high flows many of the bars and channel irregularities would be "drowned out".

Church (1972) showed that during very high flows the channel resistance could become much lower than the theoretical predictions would indicate. This was attributed partially to "live bed" conditions which apparently caused smoothing out of the bed. The Vedder River data was not collected over a sufficiently large flow range to make any conclusions on these observations.

In summary, near Yarrow the channel resistance seems to decrease at high flows as the form resistance is "drowned out". At flows around 12,000 cfs ($\frac{d}{D_{90}} \approx 15$) the resistance approaches values which correspond to skin resistance alone. At Vedder Crossing the resistance appears to remain reasonably constant over a wide flow range, probably
because the channel alignment is very irregular in this reach, causing standing waves and eddies to develop at high flows.

6.4 Bed Material Characteristics

Sediment sizes can vary widely on the Vedder River, both in the downstream direction and across the channel. Generally it was found that the coarsest sediments (gravel and cobbles) were found in the main channel of the river, forming a surface layer in which sand and finer gravels was virtually absent. This surface layer has been termed "pavement" by Bray (1972). When this surface layer was removed, large quantities of coarse sand and fine gravel was found underneath. On most of the bars a surface pavement was also evident although in general this was not as obvious as the main channel. On many bars large areas of fine gravels and sand could be found on the surface.

The large variability of sediment sizes in gravel rivers makes the collection of representative bed material samples very difficult. In addition, the natural paving which can be found indicates the gravel bed can consist of two separate populations - the surface layer and the underlying material (Kellerhals, 1971). If a simple bulk sample is collected then the two populations will be mixed together and the resulting sample will not be representative
of the pavement or of the underlying material.

Past studies have shown that the material found on bars is representative of the sediment being moved as bedload (Leopold, Wolman and Miller, 1964; Mosley, 1978). Therefore it seems reasonable to use bulk samples of the subsurface bed material to describe the composition of the bedload. Alternately, it has been generally accepted that the surface pavement is an important factor in determining channel roughness (Kellerhals, 1970; Church and Kellerhals, 1979). The only practical method to sample this surface layer is to use one of the grid methods described previously in Chapter V.

The procedures described above were used for all of the twenty one samples collected by the author. The Water Survey of Canada data were all bulk samples, however most of these were collected on bars where the surface paving was generally not prominent. Therefore these samples were used to establish the size of transported material.

In order to overcome some of the spatial variability in bed materials size, the river was sub-divided into six reaches and the samples within each reach were composited. These reaches were as follows:

Vedder Canal (mile 1.45-2.42)
head of canal (mile 2.42-3.52)
railway bridge (mile 3.52-4.43)
constriction (mile 4.43-5.61)
Ford Road to Peache Road (mile 5.61-6.56)
Vedder Crossing (mile 6.56-7.75)

The average surface and subsurface bed material composition of each reach is summarized in Table 19. These results show that the median size of the surface layer was generally around 1.5 to 3.0 times the size of the underlying material. In addition, values of Trask's Sorting Coefficient \(\frac{D_{75}}{D_{25}}\) which measure the spread of the size distribution, varied between 1.5-1.7 for the surface material and 2.25-3.3 for the underlying bed material. This shows that the surface layer is better sorted which reflects the loss of fine material in the pavement.

The average bed material composition summarized in Table 19 shows a trend towards decreasing grain size down the Vedder River. For example, the median bed size was found to decrease by half in the first 3 1/2 miles downstream from Vedder Crossing.

Downstream changes in grain size have been studied by many geomorphologists and engineers (Mackin, 1948; Yatsu, 1955; Schulits, 1941; Simons, 1971; Church and Kellershals, 1979). Initially, sediment gradation was attributed to abrasion. Following the work of Sternberg, Leliavsky (1955) presented data from the Rhine River which showed the weight of pebbles decreased exponentially with distance.
Using stone size instead of weight, this can be expressed as:

\[ D = D_0 e^{-a_d x} \]

where "a_d" is a size reduction coefficient

\[ x \]

is the distance along the river.

However, gradation may be also caused by selective transport of different bed material sizes along a river. Scheidegger (1961). This selective transport can occur in both aggradation and erosion processes. In the case of aggradational situations such as on alluvial fans, sediment sorting may occur when the transport capacity of the channel decreases with distance downstream. This can cause the coarse fraction of the sediment load to be selectively deposited near the head of the fan. Sediment sorting on aggrading streams has been described by Lustig (1963); Blissenbach (1952) and Church (1972). Sediment sorting can also occur in erosional situations where the finer material in the bed is selectively removed and the coarser immobile material is left behind.

It was decided to carry out a simple regression analysis on the coarse fraction of the bed material sample collected on the Vedder River. In order to reduce some of the variability of the samples all of the fine material (less than 8 mm) was removed from the samples. It was reasoned that this fine material should not show significant sorting over the length of the fan, since it can probably
be transported through the entire system. However in some areas large quantities of fine sediment can be temporarily deposited on bars. This sediment can introduce a large amount of scatter in the bed material samples unless the sampling sites are selected carefully to avoid this material.

Results of the analysis, based on a total of 58 samples showed the median sediment size could be expressed as:

\[ D_{50} = 65 e^{-0.193x} \quad r^2 = 0.63 \]
\[ x = \text{distance in miles} \]

As shown in Figure 32 the overall scatter is very large which indicates that although a trend is clearly present, there is a great deal of variability that is not accounted for by the regression. Some of this scatter is probably due to poor choice of sampling sites, and poor sampling techniques.

For comparison purposes, some data on downstream changes of bed material size reported by Simons (1971) has been summarized in Table 20. These results show the coefficient of size reduction \( a_d \) ranged between 0.15 miles\(^{-1}\) to 0.01 miles\(^{-1}\). In the case of most reasonably competent rock, abrasion can not account for large values of \( a_d \). For example Bradley et al (1972) concluded that for the Knik River in Alaska sorting processes accounted for 90 to 95\% of the reduction in gravel size, with the balance
attributable to abrasion.

Although the lithology of the Vedder River sediments was not studied in detail it was found that the gravels were composed primarily of, limestone, conglomerates and granodiorite. Most of the rock appeared competent. Since the coefficient of size reduction was even larger on the Vedder River than observed by Bradley on the Knik River it is concluded the selective deposition is the only significant factor causing the decrease in bed material size between Vedder Crossing and the Vedder Canal.
CHAPTER VII
SEDIMENT TRANSPORT

7.1 Suspended Load

Suspended load data has been collected by Water Survey of Canada routinely since 1965. These measurements have been made at the cableway just below the railway bridge.

Some statistics derived from the Water Survey of Canada measurements are shown in Table 21. The maximum recorded sediment concentration was measured on December 3, 1975 and reached 4,000 mg/l which corresponds to a load of 202,000 tons/day. By comparison, the average annual load between 1965-1977 has been only 146,000 tons/year.

There has been very few measurements to determine the size composition of the suspended load. However during the 1975 flood it was reported that the median grain size was about 0.05 mm and that 90% of the suspended load was finer than 0.25 mm.

The suspended load consists of two components - the suspended bed material load and the wash load. Wash load is that part of the sediment load consisting of grain sizes finer than those found in the streambed. The transport of this sediment is not governed by the hydraulic properties of the channel but by the sediment supply throughout the watershed. This is because the transport capacity of the
channel always exceeds the supply of this fine sediment. Einstein (1950) suggested that the limiting size for washload can be considered as the grain diameter for which 10% of the bed material is finer. This size fraction corresponds to between 0.4 mm-1.0 mm in the Vedder Canal and 0.5 mm-2.0 mm at the railway bridge. Since the composition of the suspended load was considerably finer than these sizes, virtually all of the suspended load can be considered as washload. Therefore although the quantity of the suspended load may be quite large it is simply washed through the Vedder River without having a major impact on the sedimentation processes.

7.2 Bedload

Bedload is that part of the sediment load that moves in contact with the channel bottom - by sliding, rolling or saltating. The Vedder River is one of the few rivers in the country where a concentrated effort has been made to collect bedload transport measurements. Other gravel rivers where bedload data has been collected include the Elbow River (Hollingshead, 1968, 1971) and the North Saskatchewan River (Samide, 1971). In addition, recently several studies have been carried out on some American gravel rivers including the Tanana River (Emmett et al, 1978) the Snake River and the Clearwater River (Emmett, 1976).
The bedload measurements on the Vedder River were collected at the cableway below the B.C. Electric railway bridge by the Water Survey of Canada between 1971 and 1975. This data, for the most part unpublished, was made available by the Sediment Survey Section of Environment Canada. Some additional data was extracted from the report by Jonys (1976). A total of 44 measurements were made, under flow conditions ranging from 3,370 cfs to 13,120 cfs. For each measurement usually several samples were collected across the river cross section in order to define the width of bedload movement.

Most of the measurements were made with either a basket sampler or a half size VUV sampler from the cableway. The basket sampler consisted of a rectangular frame with an open-ended 1/4 inch mesh basket fitted in to collect the bedload. Jonys (1976) reported the basket opening was two feet wide. It was assumed that the efficiency of the basket sampler was about 33% which is in agreement with estimates reported by Hollingshead (1971).

The half size VUV sampler is a pressure difference type sampler (Graf, 1971) with an opening of about 8 1/2 inches. In the VUV sampler sediment is collected is a metal pan so that the sand sized material should be retained. The efficiency of the VUV sampler has been found to equal 60% (Gibbs and Neill, 1973).
It is believed that the bedload transport is the main factor governing aggradation and degradation processes of the Vedder River. In the vast majority of gravel river sedimentation problems, information on bedload is determined indirectly - usually by relying on highly empirical theories that have often been based only on laboratory studies. Frequently, formulas have been derived for sand bed channels but have been applied to gravel streams without proper verification. Therefore, the bedload data collected on the Vedder River by Water Survey of Canada provides a unique opportunity to learn about the sediment transport processes on the river and to test out some of the predictive methods that are currently available.

The Yarrow cableway cross section where all of the bedload measurements were made is shown in Figure 27. In addition, the bedload data and corresponding hydraulic data collected at the time of the sampling is summarized in Table 22. A few observations on some features of the measurements are as follows.

Firstly, the bedload transport appeared to be very erratic and subject to wide fluctuations in magnitude. Jonys (1976) reports measurements made in 1974 at a single location where the transport rate fluctuated between 0.25 kg/m/minute (0.17 lb/ft/minute) to 14.11 kg/m/minute (9.46 lb/ft/minute) in a space of 12 minutes under approximately
steady flow conditions. Such large fluctuations in transport have also been reported by Graf (1971). These fluctuations mean that a large number of bedload samples must be collected to estimate the average transport rate. At present, there are no guidelines available to indicate how many samples are required to make a reliable estimate of the average transport rate.

Secondly, bedload movement did not usually occur over the entire channel but was restricted to a strip of about 125 feet in width. Jonys (1976) reported that most transport occurred along the north bank of the channel over a submerged bar and that very little movement occurred in the deepest part of the channel.

Analysis of the bedload size distribution showed that for flows under about 8,000 cfs the sediment load was composed predominately of sand. Under these conditions gravel-sized material (usually less than 4 mm) often comprised less than 5% of the total bedload. However, when the discharge exceeded about 8,000-9,000 cfs the bedload size increased very sharply and gravel made up the largest fraction of the sediment load. In order to illustrate this change in bedload grain size with discharge, the median particle size was plotted against discharge in Figure 33. In addition a few of the bedload size distribution curves have been plotted in Figure 34, along with the average
bed material distribution at the railway bridge. These curves show that when the flows exceeded about 8,000-9,000 cfs the bedload and bed material grain size distributions were very similar. This indicates that during high flows all of the grain sizes in the bed are active.

Using Shield's relation for incipient motion

\[
\frac{I}{(\gamma_s - \gamma)D} = 0.03
\]

the maximum particle size moved at a flow of 8,000 cfs would be about 70 mm, which corresponds to roughly the \(D_{80}\) size of the surface bed material. It appears that until this surface layer is set in motion the bedload consists mainly of sand moving over an immobile gravel bed. However, once the surface layer is "broken" a much larger range of gravel and sand sized material is made available for transport.

Figure 35 shows the relation between bedload transport rate and discharge. Although considerable scatter is evident on the log-log plot there appears to be a reasonably consistant relationship, with sediment load proportional to the fifth power of discharge. This illustrates the very non-linear nature of bedload transport and indicates how the majority of the annual transport may occur during a few days of high flows.

The highest flow at which bedload was measured was around 13,000 cfs. Therefore a rather large extrapolation
is required to apply this data to extreme flood conditions. As a result, it was decided to try to predict the bedload with available transport formulas and use the measurements to verify the results.

There has been very little success in estimating bedload from sediment transport theory. Blench (1969) stated that predicted bedload rates should be considered only as order of magnitude estimates. White, Milli and Crabbe (1975) compared several bedload formulas using over a thousand flume and field measurements. The formulas of Ackers and White, Engelund and Hansen and Rottner were found to be the most reliable equations. For these three formulas, better than half of the computed values were found to lie within 50% to 200% of the measured transport rates. However, the vast majority of data used to verify these formula related to sand bed streams. Church (1976) compared the Schocklitsch, Meyer-Peter, Meyer-Peter and Muller, Shields, Kalinske and Einstein formula using the bedload measurements reported by Hollingshead (1968) on the Elbow River. Church recommended that the performance of the Einstein equation could be improved by making a slight modification to some of the hydraulic calculations that are used in the original method. Basically, Church recommended that measured hydraulic data should be used to calculate bedload instead of relying on theoretical
procedures which attempt to compute the velocity and channel resistance. Church's unpublished report is one of the few studies to concentrate on the problem of predicting bedload in gravel rivers.

In order to determine which theories could give the best bedload predictions on the Vedder River, the author compared three formula that have claimed to be applicable to gravel rivers - the Einstein (1950) formula, Meyer-Peter and Muller's formula and Ackers and White's equation. These theories were verified using bedload measurements from five gravel rivers:

- Elbow River at Bragg Creek (Hollingshead, 1968, 1969)
- North Saskatchewan River (Samedi, 197 )
- Snake and Clearwater Rivers (Emmett, 1976)

A description of the five sites and results of the comparisons has been included in Appendix I.

The Einstein (1950) formula using the modification suggested by Church was found to give the closest agreement to the field measurements. However only about one third of the computed values lay within 50% to 200% of the measured data. Neither the Meyer-Peter and Muller formula nor the Ackers and White method could produce satisfactory results even when slight modifications were made to the
input data.

Figure 35 and 36 compare the three formula and the measured results on the Vedder River. The Ackers and White formula estimated threshold conditions satisfactorily however it tends to overestimate the transport at high flows by a factor of ten. The Meyer-Peter and Muller formula which is often considered to be suitable for gravel rivers gave very poor results on the Vedder River. When the original version of the formula was tried the bedload was overestimated by a factor of 50. After a modification was made to the estimation of threshold conditions (see Appendix I) the computed results intersected the measurements, however the transport rates were still underestimated at low flows and overestimated at high flows.

In view of the very poor performance of the Ackers and White and the Meyer-Peter and Muller formula, only the Einstein relation was considered applicable to the Vedder River. Unfortunately, even the Einstein relation was found to require careful useage. A sensitivity analysis showed that small changes in some of the input data could cause drastic changes in the computed transport rate, especially near threshold conditions (see Appendix I). The two most sensitive parameters were found to be the mean velocity and the effective bed roughness height (D_{65} grain size). Changing these values by only 10% could change
the sediment load by 50% to 250%. Therefore if reasonable bedload estimates are to be made field-determined hydraulic data (mean depth, mean velocity, width and slope) should be used instead of estimates based on Manning's equation or other formulas. Hollingshead (1970) also noted the importance of using actual hydraulic measurements.

The only other location on the Vedder River where routine hydraulic data has been collected is at the Vedder Crossing gauging station. Since this site also corresponds to the head of the fan it was considered important to estimate the sediment transport at this location in order to determine the sediment inflow to the fan.

The bedload transport at Vedder Crossing was calculated from the following data:

- hydraulic geometry measurements from gauge 8MH01 (Figure 30)
- surveyed slope = 0.006
- bed material samples collected from bars upstream of Vedder Crossing giving a D_{35} size of 32 mm.

It was decided to use bed material samples from the reach immediately upstream of Vedder Crossing in the calculations rather than the data collected at Vedder Crossing and reported in Table 19. It was felt that the finer samples collected on the bars above Vedder Crossing were more representative of the material being transported than the highly...
paved samples collected from the narrow, constricted reach. The coarser samples found below Vedder Crossing represent the sediment that the river has been incapable of transporting, while the finer sediments on the bars upstream of Vedder Crossing represents the material that is transported through this narrow gap onto the fan below.

The results of the calculations are summarized on Figure 37 along with the estimated bedload transport relation at the railway bridge. The threshold for sediment transport at Vedder Crossing was in the same range as at the railway bridge (roughly 5,000 cfs), however for flows exceeding 10,000 cfs the transport at Vedder Crossing was in the order of 20 to 50 times the transport near Yarrow.

It is possible to check the accuracy of the predicted Vedder Crossing bedload relation by attempting to reproduce the channel deposition resulting from the December, 1975 flood. As mentioned previously, B.C. Water Resources officials determined that 258,000 cubic yards of sediment was deposited between the railway bridge and Vedder Crossing during this flood (Tempest, 1976). Most of this sediment would have been transported during the period between December 1st and 10th when mean daily flows exceeded 5,000 cfs. Therefore the total amount of sediment deposited between Yarrow and Vedder Crossing was expressed as

\[ V = \sum_{1}^{10} (G_{vc} - G_{y}) \]

where \( G_{vc} = \) transport at Vedder Crossing
where \( G_y \) = transport at Yarrow

Assuming the bedload sediment has a unit weight of 100 lb/ft\(^3\) (Hollingshead, 1971), the total volume of sediment deposited in the reach was estimated at 155,000 cubic yards or about 60% of the measured quantity. Since the total sediment outflow at Yarrow was less than 10% of the inflow most of the error in this calculation must have been in underestimating the load at Vedder Crossing.

Computing the bedload to within a factor of two is probably all that can be expected, given the experience with the predictions at the railway bridge and on other gravel rivers where measurements have been obtained. The site at Vedder Crossing is especially difficult to carry out calculations because the reach is very non-uniform with the flow making a sharp bend and contracting very abruptly. As a result the slope may change during very high flows.

In view of the high sensitivity of the Einstein bedload equation it would be easy to improve the bedload prediction by adjusting one of the input parameters such as the slope, velocity or grain size. However, it was decided that the bedload-discharge relationship at Vedder Crossing could be estimated more directly by using the results of the 1975 flood surveys and the derived bedload relation at the railway bridge near Yarrow. It was assumed that the threshold conditions at Vedder Crossing were predicted
approximately correct by the Einstein equation. Therefore the endpoint of the bedload function shown on Figure 37 was fixed and the slope of the relation was adjusted by trial and error until the computed deposition on the fan exactly matched the surveyed quantity. This final adopted transport relation has also been shown on Figure 37.

Using this modified relationship, the transport at Vedder Crossing was estimated to be in the order of 20 to 100 times the transport below the railway bridge for flows exceeding 10,000 cfs. In fact, during the 1975 flood the total volume of sediment transported past the railway bridge and into the head of the Vedder Canal was probably only about 5% of the total sediment inflow at Vedder Crossing. Therefore the "trap efficiency" of the fan between Vedder Crossing and the railway bridge was probably close to 95%. It should be emphasized that this calculation is based mainly on field measurements:

- the bedload relationship at Yarrow which was determined by bedload measurements and extended by calculations
- the deposition computed from the 1975 flood surveys. Therefore it is believed this estimate is reasonably accurate assuming that the deposition reported by Tempest (1976) is correct.
8.1 The Process of Aggradation

Alluvial fans are distinctive land forms that are normally associated with sediment aggradation. The fan below Vedder Crossing is typical of many streams in British Columbia that flow out of the mountains onto a flat alluvial plain.

Usually sediment deposition on fans is attributed to a decrease in stream competence due to the characteristic decrease in slope which occurs along the channel (Lustig, 1965; Hooke, 1967; Malcovish, 1974). Deposition can also occur when the flow spills overbank and spreads over the fan surface.

Gessler (1971) carried out theoretical studies of aggradation and degradation processes. Gessler showed that raising or lowering of the streambed was related to the change in sediment transport rate along the river according to the sediment continuity equation:

\[
\frac{\partial G}{\partial x} = -c \frac{\partial z}{\partial t}
\]

where \( t = \text{time} \)

\( z = \text{bed level} \)

\( x = \text{distance along channel} \)

\( G = \text{sediment transport rate} \)
Gessler studies a simple case of a flume set at a slope where the bed material in the channel was just at the threshold of movement. Then a constant supply of sediment was fed into the upstream end of the channel. The resulting variation in bed level over time is summarized in Figure 38.

The aggradation developed as a wedge of sediment which eventually progressed downstream with time. Also the bed developed a concave profile which appears to be characteristic of aggrading conditions (Gessler, 1971). Eventually, as a result of the progressive build up of the bed, a new equilibrium slope was established which was steeper than the initial one. At this time all of the inflowing sediment was transported through the channel and the aggradation ceased.

The processes described by Gessler are comparable to deposition on an alluvial fan except for the fact that fans are free to shift laterally over a wide area. Therefore localized aggradation during a flood event can cause the flow to spill out of its channel and cause a sudden channel shift or avulsion. As a result, the locus of deposition may change over the entire fan surface.

Aggradation on fans can be interrupted for several reasons. For example, aggradation may cease when the
sediment supply from the upstream watershed is reduced. Ryder (1970) showed that many fans in south-central British Columbia are no longer actively aggrading due to climatic changes and to reduction in the availability of sediment.

Hooke (1967) noted that incision of the fanhead could occur on model fans when the locus of deposition shifted to a place that had not received sediment for several episodes of channel shifting. When a shift to one of these topographic lows took place, degradation occurred near the fanhead in response to the sudden increase in slope. This degradation lasted until the low area was built up to the level of the surrounding fan.

Malcovish (1974) carried out a number of model studies to determine the effect of constructing bridges and guide banks across fans. When guide banks were placed on the fan to restrict the lateral shifting of the channel, a change in the depositional pattern was noted. Initial downcutting occurred within the construction while a distinctive conical fan segment was formed below the bridge crossing. According to Malcovish, this zone of deposition served as a hinge point for further upstream aggradation, so that the initial degradation within the guide banks was eventually offset by deposition. In view of the considerable channelization work that has
been carried out on the lower Vedder River, Malcovish's observations are very relevant.

Although some of the processes governing aggradation are well understood, such as the sediment continuity relationship, there are still many difficulties involved in making quantitative predictions. The main difficulty lies in defining a sediment transport relationship along the channel. As was shown in the previous chapter, even with measured hydraulic geometry data, estimates of bed-load may be in error by at least a factor of two. Another major complication is that very limited study has gone into the unsteady nature of sediment movement and aggradation. For example, does most sediment deposition occur on the falling limb of a flood or is it coincident with the flood peak? Until advances are made in sediment transport theory, quantitative predictions of aggradation will be difficult to make and will rely to a large degree on systematic observations of channel behaviour.

8.2 Historical Deposition Rates on the Vedder River

Over the last few decades several different agencies have made estimates of the rate of deposition on the Vedder River. These groups have included the B.C. Water Resources Service (Marr, 1964; Tempest, 1976), Water Survey of Canada (W.S.C., 1971-1975) and the International Pacific Salmon Commission (I.P.S.C., 1977). The estimated
deposition rates that have been reported or can be com­puted from data supplied by these agencies is summarized in Table 23. These estimates are all for the period before the December 1975 flood.

On the basis of river surveys carried out in 1958 and 1963, Marr (1964) concluded that while considerable changes occurred at each section, there was no consistent pattern along the Vedder Canal or the river to establish an aggrading condition. Marr also carried out some bed­load transport calculations and concluded that the aver­age annual transport on the fan was about 139,000 cubic yards per year. Furthermore, it was concluded that all of this material was transported through the canal and deposited in the Fraser River. Marr's calculations were based on hydraulic conditions on the upper part of the fan and it was not established whether the canal had the capacity to transport this sediment load to the Fraser. Also, no mention was made of the sediment sorting which occurs along the river indicating that selective deposi­tion is taking place.

Water Survey of Canada resurveyed 11 cross sections located between the canal and Vedder Crossing several times between 1971 and 1975. The cross sections showed both scour and fill could occur from one year to the next, however, over the entire period net aggradation occurred at most locations. No estimate of the volume
of sedimentation was made, probably because the sections were spaced too far apart to achieve reasonably accurate results.

Tempest (1976) reported that about 258,000 cubic yards of sediment was deposited between the railway bridge and Vedder Crossing during the 1975 flood. Furthermore, it was estimated that about 235,000 cubic yards was deposited in this same area between 1972 and July 1975. In comparison, the International Pacific Salmon Commission (I.P.S.C.) estimated that 370,000 cubic yards was deposited between the railway bridge and Peache Road (about one mile below Vedder Crossing) between 1957 and 1976. This was based on 12 sections surveyed by the I.P.S.C. in 1957 and 13 sections surveyed by the B.C. Water Resources Service (B.C.W.R.S.) in January 1976. It was also stated that the average bed level rose 1.6 feet in 19 years. If the 1975/1976 deposition between the railway bridge and Peache Road reported by Tempest (1976) is subtracted from the corresponding 1957/1976 deposition, then the net deposition in this 2.6 mile reach between 1957 and July 1975 is only 192,000 cubic yards. This averages only about 10,000 cubic yards per year which is less than one-third of the rate estimated by Tempest between 1972-1975 in the 1.6 mile reach between the railway bridge and Ford Road.
The only direct comparison of cross sections below the railway bridge that can be made is from the eight 1958/1963 sections reported by Marr (1964) and the six 1971/1975 sections reported by the Water Survey of Canada. Comparison of this data showed the mean bed level rose 0.2 feet between 1958-1963 and 0.5 feet between 1971 and 1975. The corresponding volumes deposited in these periods cannot be estimated because of the very wide spacing between cross sections.

Based on the data available, it was concluded that all of the aggradation estimates prior to 1975 are very approximate. There has been no systematic long term observations on the river and most of the surveys carried out by the different agencies over the years have not been coordinated. As a result, only channel changes over relatively short periods of time can be established (1958-1963, 1971-1975). In the case of the I.P.S.C. estimates, it seems unlikely that the 1957 cross sections established by the I.P.S.C. would align exactly with the 1975-1976 sections established by the B.C.W.R.S. in view of the major changes in channel alignment that occurred over the years. Another problem with some of the estimates is that prior to 1964 a considerable amount of dredging took place on the lower river. Unfortunately, there appears to be no way to estimate the amount of sediment that was removed from the river around this
time. Finally, some estimates of deposition do not account for systematic changes in the deposition pattern which have occurred both along the river and over time. For example, the 1957-1975 estimate indicates only the net deposition between the railway bridge and Peache Road. However, as subsequent data will show, a large amount of degradation occurred during a portion of this period so that the total amount of sediment transported into this reach was much greater than the estimate would indicate.

Since it does not appear possible to directly compare the early cross sections, it was decided to plot profiles showing the mean bed elevations of each survey. The mean bed level was computed for all of the agency cross sections (except I.P.S.C., 1957) according to the following definition:

$$
\bar{E} = \frac{\sum E_i \Delta x}{X}
$$

where $E_i$ is the average bed elevation for the channel increment $\Delta x$.

$X$ is the active channel width.

The bed profiles computed from Marr's 1963 data and from the W.S.C. and B.C.W.R.S. 1975 data are plotted in Figure 39. Comparison of these profiles shows three main features:

- relatively little change occurred on the upper part of the fan between Peache Road and Vedder Crossing.
- degradation occurred between the railway bridge near Yarrow and Ford Road.
- aggradation occurred downstream of the railway bridge to near the head of the Vedder Canal.

The degradation in the 1.6 mile reach between the railway bridge and Ford Road ranged from between 2.0 feet and 4.5 feet. It is believed this degradation was caused by the extensive channelization that was carried out above the railway bridge between 1964 and 1968. Since the active channel width was reduced by as much as 50 per cent, the channel velocities could have been nearly doubled in this reach shortly after channelization. As a result the sediment transport capacity would have been increased in the narrow channel and would have exceeded the sediment inflow from the upper part of the fan.

Figure 40 compares a channel cross section surveyed near Hopedale Road in 1975 with a cross section surveyed in 1963 at the same location. This shows that when the channel width was reduced from about 700 feet to 350 feet, the mean bed level dropped about 4.6 feet while the elevation of the deepest portion of the channel (the thalweg) dropped about 9.0 feet.

A very rough order of magnitude estimate of the degradation in this reach is approximately 300,000 cubic yards, assuming the average channel width in 1975 was about 300 feet. The time required to develop this
degradation cannot be estimated precisely, however, the W.S.C. surveys between 1971 and 1975 show this reach was stable or possibly aggrading. Therefore, most degradation probably occurred soon after channelization began and ended by about 1970. Between 1964 and 1970 the largest flow reached only 15,400 cfs on June 2nd, 1968, and the daily discharge exceeded 10,000 cfs on only six days.

The aggradation below the railway bridge appears to be in the form of a wedge of sediment extending about 1.25 miles in length and having a maximum height of about 3.8 feet just below the bridge. This wedge of sediment creates an obvious "hump" in the streambed profile (Figure 39). A very rough calculation shows that the approximate volume of sediment deposited in this area was about 150,000 to 200,000 cubic yards. Since dredging is known to have occurred below the railway bridge in the mid-1960's, the actual amount of material deposited in this reach could have been considerably greater.

It is apparent that the material deposited below the bridge was derived from the degradation occurring in the channelized reach immediately upstream. This suggests the sediment was swept through the channelized reach and was deposited in the first area where the channel width expanded. As this wedge of sediment formed below the bridge and as the channel lowered in the constriction, the slope through these two reaches would have
gradually changed. This would have led to a decrease in sediment transport through the constriction, ceasing of degradation and eventual reestablishment of aggradation upstream of the bridge. This is similar to the laboratory observations reported by Malcovish (1974).

If the bedload transport relation established by the Water Survey of Canada's measurement program between 1971-1975 is compared with the quantity of sediment deposited below the railway bridge between 1963 and 1975, there is an obvious discrepancy. Using this bedload relation, there is no possible way for 150,000-200,000 cubic yards to be transported past the Yarrow cableway and deposited near the head of the canal. In fact under the flow conditions that existed between 1963 and 1975 the predicted deposition amount to about 10% of the estimated value. Although both the bedload measurements and the computed deposition volume may be subject to considerable errors, it is believed these are not sufficient to account for such a large difference. It is much more likely that the sediment transport capacity at the bridge gradually reduced as the slope and transport capacity in the constriction also changed. Also, when degradation was first going on in the constriction, the high velocity flow emerging from the bridge must have resembled a jet and this would also have tended to sweep some of the sediment considerably further downstream than might
ordinarily be predicted.

There is one final source of data that can provide additional information on historical channel changes on the Vedder River. These are the gauge records collected at Yarrow and Vedder Crossing which can be used to determine whether any systematic changes have occurred in the stage-discharge relations at the two sites. In order to determine whether systematic trends exist, the water level corresponding to a specific discharge was plotted over the period of the gauge record. The resulting plot of stage vs. time is termed a specific gauge record and according to Blench (1969) it is one of the most powerful single tests to determine whether a river is "in-regime."

Figure 41 shows the specific gauge plot at the Yarrow gauge. This was established from continuous stage measurements between 1952-1972 and miscellaneous measurements between 1973-1977. This plot shows that the river has been aggrading nearly continuously except for a short period between 1960-1964. The aggradation rate between 1964-1975 totalled about 3.6 feet or about 0.30 feet/year. This is nearly identical to the mean bed rise estimated from the 1963/1975 channel surveys.

It is believed that the apparent bed lowering indicated between 1960-1964 reflects the large amount of dredging and channel clearing work that was carried out
around this time (Table 1). When the dredging was stopped in 1964 and the channelization work commenced upstream of the railway bridge, the channel at the gauge location rose 2.0 feet in one year.

It is unfortunate that the Yarrow gauge had been discontinued at the time of the 1975 flood. However, the author surveyed the water level at the gauge site in 1976 and 1977 in order to determine the impact of the flood and the subsequent dredging operations. The gauge level was found to increase by about 0.4 feet between July 1975 and May 1976, which is not very different from the long term average rise. However, the Phase I limited dredging program had been carried out in between this time which could have reduced the water levels slightly. The large drop of about 3.3 feet between 1976-1977 reflects the massive Phase II dredging program that was carried out in 1976.

Considering that the Yarrow gauge provided the only continuous long term record of aggradation on the Vedder River, it seems very poor planning to have discontinued its operation.

The specific gauge plot at Vedder Crossing extends from between 1918-1929 and 1951-1975 (Figure 42). However, due to a shift in the gauge's location, the two sets of records cannot be related. The earliest part of the record shows the gauge level dropped nearly 4.0
feet in the five year period between 1918-1923. This change occurred only one year after the flood of record in 1917 and only about 25 years after the period of major channel shifting in 1894. It is not known whether the apparent bed-lowering resulted from construction activity which was carried out to prevent the river from shifting down one of its former channels or from natural degradation. Marr (1964) showed an early map near Vedder Crossing indicating that a stone revetment was in-place by 1916 along the north bank. Marr also mentioned that further work was carried out after the 1917 flood between 1921-1928. As shown by the cross sections in Figure 28, the channel near Vedder Crossing is close to being entrenched, with the bankfull depth in the order of 15 feet. In addition a low terrace approximately 10 feet above the surrounding flood plain can also be identified immediately upstream of Vedder Crossing. These features suggest that recent degradation has occurred near the head of the fan and for some distance upstream as well. Some degradation could probably be expected when the entire Chilliwack River flow was forced down Vedder Creek instead of being split into three channels as it was prior to the turn of the century. Therefore, in some respects, this early degradation would have developed under similar conditions as the degradation in the channelized reach above the railway bridge.
Between 1951 to the present the gauge at Vedder Crossing showed a slow alternating pattern of scour and fill, with aggradation occurring between 1951-1958 and degradation occurring between 1959-1968. It would be very interesting to know how the river was behaving upstream of Vedder Crossing throughout this period. Unfortunately, the only data available is three cross sections surveyed in 1958 and 1963 (Marr, 1964). All three sections showed net aggradation between 1958-1963 during the period when the fan head was slowly lowering. This may indicate that sediment was being trapped above Vedder Crossing during this period so that the actual supply to the fan was very low. Then, when a major flood such as in 1975 occurred, this sediment is washed through the narrow gap at Vedder Crossing and transported on the fan.

8.3 Aggradation During the 1975 Flood

The deposition resulting from the December 1975 flood has already been mentioned a number of times in this chapter and in earlier chapters as well. Tempest (1976) estimated the deposition between the railway bridge and Vedder Crossing was distributed as follows:

- railway bridge - Browne Rd. (1.1 mi) = 72,000 cu. yds.
- Browne Rd. – Ford Rd. (0.5 mi) = 52,000 cu. yds.
Ford Rd. - Peache Rd. (1.0 mi) = 58,000 cu. yds.
Peache Rd. - Vedder Crossing (1.1 mi) = 80,000 cu. yds.

This estimate was based on cross sections surveyed in July 1975 and January 1976. However, it appears that a certain amount of extrapolation was required to determine this deposition since the 1975 cross sections did not extend throughout the entire reach. For the purposes of this study, the numbers will be accepted as reasonably accurate estimates.

In order to determine how the aggradation varied along the river, the net area of deposition or scour and the mean bed level change were computed for each of the 13 cross sections provided by the B.C.W.R.S. These results indicated the average bed level rise was about 0.75 feet and ranged from virtually no change to about 2.0 feet.

Figure 43 shows a plot of the net channel area change, mean bed level change and channel width along the river.

The most interesting feature of this graph is the apparent relationship between channel deposition and bottom width. The greatest deposition consistently occurred in reaches where the width increased abruptly while the channel showed virtually no net change in some of the contractions. The zones where the deposition was greatest included:
- the very wide area of bars and islands near Peechel Rd.
- the braided reach near Ford Rd.
- near Hopedale Rd. where the channel width increases from about 200 feet to 330 feet.

All of these reaches could be characterized as flow expansions which show abrupt increases in width over relatively short distances.

The areas which remained relatively stable after the flood included:
- the short reach upstream of Ford Rd. where the channel flows in a single channel
- in the centre of the channelized reach where the width has been reduced to about 200 feet.

The results shown in Figure 43 suggests that deposition depended more on whether the flow was expanding or contracting than the absolute width. As a result, providing a narrow channel will not necessarily insure the river will remain self-scouring during a major flood.

When the channel changes surveyed by the Water Survey of Canada between 1971 and 1975 were plotted in a similar fashion as Figure 43, the results appeared exactly opposite to the pattern observed in 1975-1976. Between 1971-1975 most deposition occurred in the narrower reaches and relatively little change occurred in the wider areas. Therefore, the pattern of deposition
during the 1975 flood may not be consistent from year to year.

8.4 Prediction of Future Aggradation

The deposition that occurred during the 1975 flood caused considerable disruption to the Vedder channel and was partially responsible for the high water levels along the river. However, the 1975 flood was really a minor event having a return period of only about 10 years. If a 50 or 100 year flood occurred, the maximum instantaneous discharge could probably reach 40,000-50,000 cfs and the quantity of sediment deposited along the river could possibly be much larger than the 258,000 cubic yards that was estimated for the 1975 flood.

The computed bedload transport rates at the railway bridge and at Vedder Crossing derived in Chapter VII can be used to estimate the deposition resulting from various flood conditions. However, in order to compute the total deposition that may occur, the shape and duration of the flood hydrograph must be known as well as the flood peak.

The peak snowmelt and rainstorm floods were estimated for different probabilities of occurrence in Chapter III and were summarized in Table 5. The hydrographs corresponding to these floods were estimated as follows. A dimensionless snowmelt and rainstorm flood hydrograph
was developed from historical data by plotting:

\[
\frac{Q_t}{Q_p} \text{ vs. } t, \text{ where } Q_p = \text{peak flood} \\
Q_t = \text{flow on day "}t" 
\]

Discharges recorded in 1914, 1917, 1921, 1924, 1928 and 1975 were used to establish the rainstorm hydrographs. During these years the peak daily discharge ranged between 17,200 cfs and 27,000 cfs. The dimensionless snowmelt hydrographs were plotted from floods recorded in 1961, 1967, 1971, 1972 and 1976. During these years the maximum daily flows ranged from 8,800 cfs to 12,600 cfs.

The ordinates of the flood hydrographs were then determined as simple ratios of the peak mean daily flood discharge. The resulting snowmelt and rainstorm "design" hydrographs are shown in Figure 44.

The main difference between the snowmelt and rainstorm hydrographs is the much more gradual rise and recession of the snowmelt floods. As a result, for the same peak discharge, the snowmelt floods could transport considerably more sediment than a rainstorm flood because the duration of high flows is much greater. However, an extreme snowmelt flood is in the order of 12,000 cfs to 15,000 cfs while rainstorm floods can easily reach twice these flows. Therefore, over a long period of time, it is believed that the rainstorm floods will produce the dominant effect on sedimentation on the Vedder River.
The expected deposition between the railway bridge and Vedder Crossing was estimated for various flood frequencies on the basis of the sediment transport relations shown in Figure 37 and the hydrographs shown in Figure 44. The results, which are summarized in Table 24, confirm that for extreme events the winter rainstorm floods will produce much greater deposition than the snowmelt floods. The deposition predicted for a peak daily winter flood of 27,500 cfs (having a return period of 50 years) was close to 600,000 cubic yards or about 2.5 times the deposition that was reported in 1975. These figures represent minimum estimates since the sediment transport rates were all computed for mean daily flows and no attempt was made to account for the fact that the maximum instantaneous flow could be as much as 50% higher. However, even deposition of 600,000 cubic yards of sediment could produce local aggradation of several feet and could probably create considerable channel shifting and lateral instability.

In addition to knowing the deposition resulting from a single flood event, it is equally important to have an estimate of the average annual deposition rate. This has been estimated for two areas—downstream of the railway bridge and between the railway bridge and Vedder Crossing.
The transport into the reach below the railway bridge was estimated from the W.S.C. bedload measurements and theoretical calculations presented previously. The average annual transport was then estimated by means of a flow-duration curve which was determined from daily flow records at Vedder Crossing. Only flows exceeding 5,000 cfs were included in the analysis since the transport rate is very small below this value. The resulting calculation showed the transport below the bridge was less than 4,000 cubic yards per year. The sediment inflow at the railway bridge has been very low so it is no wonder that the grade of the Vedder Canal has not changed appreciably over the years (Marr, 1964; Sinclair, 1961).

This calculation does not include the large deposition that occurred below the railway bridge between 1963-1970, which was brought on by degradation in the constricted reach immediately upstream. It is believed this large deposition was a transient feature that is not representative of the long term transport rate. On the other hand, the bedload measurements between 1971-1975 were made after the upstream degradation had ceased which suggests a rough equilibrium condition had been achieved.
The average annual transport at Vedder Crossing was estimated from the same flow duration relation used in the previous calculation and the predicted bedload results shown in Figure 37. This calculation indicated an annual transport of about 100,000 cubic yards per year. Since predicted annual transport was only 4% of this estimate, the net deposition between the railway bridge and Vedder Crossing can also be taken as around 100,000 cubics yards/year.

It is possible to estimate the deposition in this reach from other sources in order to provide a comparison with the bedload calculations. These sources include:

- estimates of bank erosion upstream of Vedder Crossing which were presented in Chapter 5. The total erosion between 1940-1976 in the reach between Liumchen Creek and Vedder Crossing was about 2,200,000 cubic yards which averages 60,000 cubic yards/year.

- estimated deposition reported by I.P.S.C. (1977) between 1957-1976. The estimated 1963-1970 degradation in the constriction (300,000 cubic yards) should be added to the I.P.S.C. figure of 370,000 cubic yards net deposition in order to arrive at the total sediment inflow at Vedder Crossing. This figure totals 670,000 cubic yards between 1957-1976 or 46,000 cubic yards/year.
- estimated 1972-1976 net deposition of 493,000 cubic yards reported by Tempest (1976). Accounting for the sediment outflow at the railway bridge, the total inflow at Vedder Crossing averaged about 100,000 cubic yards/year.

A simple average of these four estimates indicates the average annual transport at Vedder Crossing is in the order of 76,000 cubic yards/year. Therefore, the average deposition between Vedder Crossing and the railway bridge is in the order of 72,000 cubic yards/year. Since the four estimates were made over different periods of time, computing a simple average is not strictly correct. However, the error introduced from this fact is probably small compared to the inaccuracies of the initial calculations.

The average deposition rate is probably most representative of the flow conditions existing over the last 20 years which has had unusually low floods compared to earlier years. Therefore, if data was available for a much longer period the annual deposition rate would probably be considerably higher - perhaps by even a factor of two. In addition it must be remembered that the deposition from a single extreme flood might exceed the average by a factor of ten.
CHAPTER IX
FLOOD CONTROL ON THE VEDDER RIVER

Providing flood control on streams that are actively aggrading is one of the most difficult problems in river engineering. This is because the channels can become very unstable during floods and subject to erratic shifts. Also, due to the progressive rise in the bedlevel, bank-full conditions are usually exceeded frequently and a large portion of the total flow may be carried by the floodplain.

9.1 Some Examples of Flood Control Practice

Some of the most interesting examples of river stabilization problems are found on some of the largest rivers in the world including the Mississippi River (Winkley, 1977), the Yellow River in China (Chou, 1976), the Kosi River in India (Gole et al, 1966) and the Brahmaputra River in Pakistan (Latif, 1969). Some of the historical channel shifts on these rivers have been immense (the Yellow River shifted 310 miles in 1194 A.D.) and have resulted in very high damages and loss of life.

However, these rivers are so large and of such different character than the Vedder River that the methods developed to provide flood control are not very comparable.
Therefore, it was decided to try to find some case histories of stabilization problems on streams that could be considered comparable to the Vedder River. The four examples that will be described include:

- Snake River near Jackson Hole, Wyoming (Jones, 1966)
- Rhine River near Lake Constance (Einstein, 1973)
- Waimakariri River, New Zealand (Henderson, 1960)
- Southern California alluvial fans (Wong and Robles, 1971)

Some of the general characteristics of these rivers are summarized in Table 25.

9.1.1 Snake River, Wyoming

According to Jones (1966), the Snake River is a braided, gravel stream that is actively aggrading. The maximum recorded flood is 40,000 cfs and the two year flood is about 20,000 cfs. The channel slope was reported to be 19 feet/mile or about 0.0036.

Prior to river training, extensive channel changes endangered large areas of ranch land. The method of bank stabilization was to construct earth and gravel levees protected by riprap which extended up to the level of annual flood discharge (15,000 cfs). The top of the levees extended about 3 feet above the project design flood which was 45,000 cfs. The distance between levees
was established at 1,000 feet, which was the width estimated to maintain the channel without aggradation or degradation. At the time of this report, the project had only been completed two years so its long term performance could not be evaluated.

9.1.2 Rhine River, Switzerland

Einstein (1973) described a very extensive investigation of sediment aggradation and river training that was carried out on the Rhine River above Lake Constance in the 1930's. The river is described as a mountain stream with a gravel bed. The river appears to have a steep slope of about 0.01 and the maximum discharge was reported to be 3,000 m$^3$/s (106,000 cfs). Extensive bedload measurements were made on the river and the annual transport was reported to be 90,000 tonnes/year (99,000 tons/year) which is probably in the same order as the Vedder River. The main problem on the Rhine River was channel aggradation which was brought on by extensive dike construction further upstream. This channelization caused the coarser sediment to be transported to the reach below and be deposited. The solution to this problem was to reduce the bottom width of the aggrading reach in order to increase its transport capacity. According to Einstein, as a result of a continuous dredging program and channel narrowing, the river has remained stable
over the years. No mention was made whether this channelization work created aggradation problems further downstream on the river.

9.1.3 Waimakariri River, New Zealand

The Waimakariri River flows in a mountainous valley until reaching the Canterbury Plains where it flows for about 36 miles before reaching the ocean. In this lower reach it changes its channel pattern from highly braided to split and eventually becomes a single meandering channel. In addition, the slope and bed material size also decreases downstream with the river bed changing from gravel to sand and the slope dropping from 0.005 to 0.00038.

In general terms the Waimakariri River flood control scheme relied on three components--channel excavation to control aggradation, a main channel which was supposed to carry the sediment to the sea and have a capacity for low floods (less than 2 years) and set-back dikes located a considerable distance from the channel to contain the overbank flows.

Henderson's study was intended to evaluate the long term methods of providing flood control on the river and to recommend possible alternative solutions. The main recommendations included:

1. construct bank protection in the upper reaches of
the river to reduce erosion and thereby decrease the supply of sediment to the aggrading reach.

2. remove local constrictions in the flow which caused increased water levels and contributed to downstream aggradation.

3. maintain channel dredging at a level that equalled or exceeded the average annual sediment transport on the lower part of the river.

4. carry out limited river training in the aggrading reach in order to improve its transport capacity.

5. After the above steps were completed, a careful monitoring program was suggested to see whether aggradation was increasing or decreasing. If the bedlevels persisted to rise after a 5 to 10 year period, further studies were recommended to determine whether additional channelization or dredging should be carried out.

9.1.4 California Coastal Streams

Wong and Robles (1971) describe some of the extensive facilities that have been constructed in southern California to control flooding from debris-laden mountain streams that flow onto the valley areas and cause widespread destruction. The main facilities that have been built include debris basins at the headworks of flood control channels, flood control basins to reduce flood
peaks and concrete lined channels to carry water and sediment. The debris basins were designed to provide storage for gravel bedload and had capacities in the order of 100,000-200,000 cubic yards. They were constructed by excavating a bowl shaped pit into the surface of the alluvial fan and placing an embankment around the downstream rim of the pit which ties into high ground. The extensive structural flood control works in California have been built at a cost of many millions of dollars over a period of 30 years. Much of this work could probably only be justified because of the large population in this area and the high flood damages involved. Therefore, most of the flood control measures are probably not applicable to the Vedder River.

9.2 Flood Control on the Vedder River

On the Vedder River the main requirements of flood control should be:

1. prevent the river from undergoing a radical shift down one of its former channels or developing a completely new route to the Fraser River.

2. provide flood protection to Yarrow and some of the land upstream of the railway bridge.

3. provide an optimum channel alignment that will minimize the effects of aggradation and protect
against bank erosion and channel shifting.

4. adopt flood control schemes that are compatible with salmon spawning requirements and with possible salmon enhancement projects.

Following the 1975 flood, three main flood control developments were carried out. These included:

- removal of approximately one million cubic yards of gravel between the head of the canal and Peache Road (I.P.S.C., 1977). As a result, the bottom elevation of the channel was reduced by several feet.

- bank protection in the channelized reach upstream of the railway bridge was raised to provide capacity for a 24,000 cfs flood with 2 feet of freeboard (I.P.S.C., 1977).

- approval was received for earthfill set-back dikes which would be located upstream of the railway bridge and protect against a 200 year flood.

When set-back dikes are constructed, the potential for serious flooding on the Vedder River should be reduced. If the dikes extend far enough upstream and tie into high ground, then the chance of the river making a sudden shift across its fan should be greatly reduced. At present, the most likely location for such a shift is the old Atchelitz channel, with flood water breaking out of the Vedder channel downstream of the Army base, below
Vedder Crossing.

The long term effectiveness of the massive dredging and bank raising along the Vedder River is more questionable. Although a large quantity of sediment was removed, unless regular dredging is carried out, the channel will gradually fill in and return to the conditions existing in 1975. The specific gauge record near Yarrow shown in Figure 42 indicates how rapidly the channel can fill in once dredging is discontinued.

Vincent (1968) carried out field and model studies to determine the effect of deepening the channel of the Danube River on upstream river levels and sediment transport processes. The studies showed that the bedload transport into the dredged reach increased sharply above the normal rate that had been measured prior to dredging. As a result, the hole filled in rapidly and degradation was induced in the upstream reach due to an increase in slope.

A similar situation could also occur on the Vedder River, with rapid aggradation developing first below the railway bridge and then progressing upstream into the constricted reach. The time required for this filling will depend on the sequence of flows that occur over the next few years. If flood conditions remain close to the long term average, then the time required could probably be only 10-15 years. If a sequence of very high flows
occurred, possibly less than 5 years would be required.

It is clear that regular maintenance will be re-
quired on the Vedder River even when set-back dikes are
constructed. As a result, it is fair to examine possible
alternatives to the existing channel alignment.

In the previous section, four very different methods
of providing flood control on aggrading gravel rivers were
described. These methods included:

- widely spaced armoured levees (Snake River, Wyoming)
- river narrowing and upstream dredging (Rhine River)
- upstream bank protection to reduce sediment supply,
  set-back dikes to contain overbank flows, river
  training and dredging to control sediment aggrada-
  tion (Waimakariri River)
- construction of debris basins, extensive channel-
  ization (southern California alluvial fans).

Although each river has its own unique character-
istics which determine the type of flood control works
that will be most appropriate, it is interesting to note
that most of the schemes listed above have in part or
in whole been suggested for the Vedder River. Therefore,
at this time some alternative methods of providing flood
control will be reviewed.
9.2.1 Upstream Sediment Control

Reducing the supply of the gravel bedload above Vedder Crossing could, in the long term, provide an effective method of reducing sedimentation on the Vedder River. As discussed in Chapter 5, probably the most important source of this sediment is produced from bank erosion in the laterally unstable reach between Vedder Crossing and Liumchen Creek.

The I.P.S.C. suggested that controlling erosion in this reach would be beneficial in reducing aggradation on the Vedder River. It was also concluded that log jams and debris contributed to this erosion (I.P.S.C., 1977).

Unfortunately, controlling bank erosion could become very expensive even if this work were confined to the most unstable reach which extends about 10,000 feet above Vedder Crossing. If a continuous revetment were constructed, probably 50,000-75,000 cubic yards of heavy rip-rap would be required which could cost well over one million dollars. This scheme would not be entirely effective since most of the large islands and bars would still be subject to erosion. Probably a cheaper approach would be to construct a number of short spurs or "plugs." Their purpose would be to prevent the river from re-occupying old abandoned side channels by encouraging them to silt up and become stabilized with vegetation. Special
care would have to be taken to insure that these structures did not encourage new erosion or increase channel scour.

It is unlikely that any erosion control scheme could reduce the sediment supply to the Vedder River entirely. However, even a reduction of 20-30% could prove to be very important during a major flood. Probably the best approach to take would be to carry out a limited program to begin with—for example, complete measures to protect the road running along the north bank (Figure 22), then wait a few years and see how the protection is performing. At this time detailed surveys would be made downstream of Vedder Crossing to determine whether the rate of aggradation was being reduced.

It is unlikely that authorities would consider any erosion control measures at the present time because of the expense involved and because it is difficult to quantify the effectiveness. However, as residential development intensifies above Vedder Crossing, the justification for erosion control should increase.

9.2.2 Sediment Traps

Large excavations could be made in the channel to help trap sediment and localize sediment deposition. Based on the pattern of aggradation that has been observed on the river, three locations already appear to
function in this manner and could be developed relatively easily. These areas are:

- the wide braided reach near Peache Road
- the wide area near Ford Road
- immediately downstream of the railway bridge.

All three of these sites are areas where the width increases abruptly, causing a sudden flow expansion. An excavation 2,000 feet long by 600 feet wide by 5 feet deep might be able to trap about 200,000 cubic yards of gravel. Therefore, the sediment traps would be most effective in controlling the average annual sediment deposition but during a major flood they would probably be completely overwhelmed.

One disadvantage of these traps is that dredging would be carried out in areas that might be prime spawning areas for salmon (for example Ford Road). However, it is possible that much of the excavation could be carried out on dry bars in these wide, braided reaches which could minimize disturbances. Also, it is possible that if permanent sediment traps were constructed, some form of overhead cable excavation system could be developed.

There appears to be no point in constructing a large trap above Vedder Crossing, as there are no suitable locations unless major changes were made to the channel. The major goal of sediment control in this area should be channel stabilization, not removal.
9.2.3 Wide Vedder River with Armoured Dikes

This scheme would leave the river above Ford Road in its existing form, with the addition of rip-rapped dikes to contain overbank flows. Below Ford Road the existing bank protection would be removed and the channel would be allowed to develop within a width of about 1,000 feet between the dikes. A possible alignment of these dikes is shown in Figure 45 which is reproduced from Peters, (1978).

The advantage of this scheme would be primarily an improvement to the spawning and rearing areas for the Vedder River fisheries. Peters (1978) showed this could significantly increase the production of chum salmon from the lower river.

A second advantage of a widened Vedder River is that the storage area available for sediment deposition would be greatly increased. If the river was returned to its 1958 alignment between the railway bridge and Ford Road, the bottom area would be nearly doubled. Therefore, during a major flood the bed level rise due to sedimentation might be less than in the existing alignment.

There are several disadvantages to this scheme. First, the armoured dikes would have to carry out two different functions—prevention of major channel shifting and containment of overbank flows. As a result, they could end up not handling either role very effectively.
If the dikes were spaced about 1,000 feet apart, the river would probably eventually return to a very braided channel pattern as shown on the 1930 airphoto in Figure 4. As a result, the river would become very shallow with large quantities of sediment added to the channel from bank erosion. This in turn would probably contribute to further aggradation during major floods and erratic alignment shifts. An example of this type of shifting occurring during the February, 1951 flood when a large amount of erosion occurred along the south bank near Browne Road (Figure 4).

Also, in order to achieve some form of river training, the armoured dikes would probably have to be situated closer to the river than the set-back dike scheme. As a result, there would be comparatively less overbank area to carry the flow over the floodplain. In this case the "Wide River" scheme may actually not increase the channel conveyance very much.

If a wide channel scheme was adopted, there would not be much advantage in trying to construct sediment traps—in this configuration the entire channel is the sediment trap. Therefore, dredging would be required over the entire reach. This would probably be considered advantageous to the fisheries since dredging could probably be concentrated on the tops of dry gravel bars and in-channel excavation could be avoided.
9.2.4 River Training with Set-Back Dikes

This scheme is roughly equivalent to the plan adopted by the B.C. Water Resources Service. The set-back dikes would be constructed of earth-fill and would be spaced roughly 1,000-1,500 feet apart.

The river training would be accomplished by a continuous revetment along the main low-water channel. It is expected that the existing channel alignment upstream of Ford Road would be maintained. If further river narrowing was carried out on the upper half of the fan, there is no doubt as to the outcome. Aggradation would increase drastically on the lower reach near Yarrow which could threaten the stability of the channel.

The main advantage of this scheme is that flood control would not rely on a single facility as in the "Wide River" scheme. The purpose of the set-back dikes would be to control overbank flow on the floodplain and to direct this back into the main channel below the railway bridge. Therefore, the dikes could be designed so that during very extreme floods a large part of the flow could be carried by the floodplain.

The revetment would limit bank erosion and reduce the influx of sediment on the upper part of the fan. On the lower half of the fan, the main object of the bank protection would be to help maintain a stable channel that could transport its average annual sediment load without
excessive aggradation.

The main disadvantage of this scheme is that the spawning habitat on the lower river would suffer from most of the problems that already exist. These problems include limited area available for spawning, destruction of eggs due to channel scour during floods, poor rearing areas due to high velocities and possible disruption from in-channel dredging. Some of these effects could be partially overcome by rehabilitating some of the old side channels on the south bank which are located between the bank protection and the set-back dikes.

A second disadvantage of this scheme would be that reasonably frequent maintenance would be required to the bank protection. Also, sediment removal from below the railway bridge, Ford Road and Peache Road would probably be a very important component of this flood control scheme. Therefore, if localized, intensive dredging was prevented, it would be difficult to make this approach work.

9.3 Designing a Flood Control Scheme for the Vedder River

The general concept of providing bank protection to stabilize the Vedder channel and set-back dikes to contain the overbank flow appears to be the soundest method of providing long term flood control on the Vedder
River. Unfortunately, this may not represent the best alternative as far as the fisheries resources are concerned. On the other hand, this scheme does not necessarily ensure that the river must be turned into a channelized ditch, nor does it rule out some possible salmon enhancement alternatives. Therefore, a general plan for developing this flood control scheme will be discussed.

The main questions that need to be answered include:

1. What should the main channel alignment be?
2. How should dredging be incorporated into the plan?
3. What should be the capacity of the main channel?

In an early proposal presented by Marr (1964), it was suggested that a channel could be constructed so that the entire sediment inflow could be transported through the system and deposited in the Fraser River without producing aggradation. The sediment transport calculations at Yarrow and Vedder Crossing along with the analysis of sediment sorting on the fan has shown this to be entirely impossible. Therefore, in order to control aggradation, sediment removal will be required.

The depositional pattern observed during the 1975 flood showed that deposition tended to be localized in the abrupt flow expansions while most of the contractions remained more stable. This feature suggests a possible
channel alignment that could tend to localize the sediment deposition in particular sites while allowing a significant portion of the channel to remain stable. Since most deposition would occur in the wide braided reaches, sediment removal could probably be accomplished by excavating dry bars which would minimize fisheries objections.

A possible schematic alignment is illustrated in Figure 46. Above Ford Road the existing channel would not be greatly modified except for providing for sediment removal near Peache Road. No river narrowing would be made in this reach. The two other deposition zones would be developed near Ford Road and below the railway bridge. As a result, the Vedder River would be developed into an alternating sequence of transport reaches and deposition reaches spaced roughly one mile apart. In order to prevent aggradation developing into the transport reaches, over the long term, the annual sediment load deposited on the fan would have to be removed by dredging. Based on calculations presented earlier, this would probably average about 70,000 cubic yards per year.

It is expected that under average flood conditions, the alternating channel pattern would stay stable for many years. However, during extreme floods the channel could be changed extensively so that a new pattern would have to be established afterwards.
Once a channel alignment is selected, the flood capacity of the main channel must be decided. If overtopping was allowed to occur very frequently, then such a large portion of the flow would be carried by the floodplain that the transport capacity of the channel would be greatly reduced. Alternately, if overtopping is never allowed, very high channel velocities would occur along with scour and possible bank erosion.

In Henderson's study of the Waimakariri River, it was recommended that the best approach for establishing the bankfull capacity of the main channel was to find out the frequency of overbank flooding on natural rivers. Based on data collected on New Zealand, European and American rivers, Henderson adopted a design flow with an annual return period of about 1.5 years. Bray (1972) carried out a similar study using data from 72 Alberta rivers and concluded bankfull conditions usually had a return period of between 2 and 10 years. Furthermore, it was concluded that a return period of 2 years provided the best estimate of a "dominant" or channel forming discharge (Bray, 1975).

A two year maximum instantaneous flood on the Vedder River has a discharge of about 15,000 cfs. Therefore, this figure was adopted for establishing the capacity of the main channel.
The final decision is to estimate the width of the main channel below Ford Road. Theoretically the inflowing sediment load at Peache Road could be computed and the channel could then be designed to match this rate. However, the inaccuracies involved in carrying out this calculation are probably so large that it is not worth trying.

An alternative approach is to treat the channel as if it were a natural, self-formed river that has adjusted to its bed material, sediment inflow and discharge. This assumption would be completely invalid on an aggrading alluvial fan but in this hypothetical alternating transport and deposition channel this assumption may not be too incorrect.

Parker (1979) considered the case of self-formed rivers with gravel bed and banks transporting low sediment loads and derived a number of rational regime equations to compute the hydraulic geometry of these channels. The equations developed by Parker are summarized in Table 27.

For a bankfull discharge of 15,000 cfs and a representative bed material size of 45 mm, these equations indicate the stable bed width should be about 320 feet and the maximum channel depth would be about 6.5 feet. This is slightly wider than some parts of the 1976 channel which reached about 250 feet in some places.
It is believed this approach would have a reasonable chance of success during "average" flood conditions. During a catastrophic flood which might exceed 40,000-50,000 cfs would probably have relatively little effect on the flows and it is likely that channel widening might occur and some of the old side-channels would be re-occupied. Therefore under these flow conditions this scheme would not act much differently than the wide river scheme. However, care would have to be taken to insure that the set-back dikes were situated far enough away from the side-channels so that erosion did not take place.
CHAPTER X
CONCLUSIONS

Flood Hydrology

The Chilliwack River is typical of many mountainous streams in southwestern British Columbia. Two separate types of floods can occur—spring snowmelt and winter rainstorm floods. A frequency analysis was carried out for each flood-type and the results were then combined to determine the annual flood frequency. It was found that for extreme floods the winter rainstorm events are dominant.

Analysis of the occurrence of flooding over the last 70 years showed that the incidence of major floods has been unusually low since 1950. This was attributed to short term climatic cycles.

The Regime of the Chilliwack River

Above Vedder Crossing the Chilliwack River flows in a glaciated valley that has been subsequently modified by river processes. For much of its length, the river flows on an imposed slope through boulder deposits. These "lag" deposits become mobile very infrequently. Throughout most of this reach the river is frequently confined by terraces or valley walls.
In the two mile reach between Liumchen Creek and Vedder Crossing, the river has been free to develop a wide alluvial plain, and the channel pattern has alternated between being braided or split with large wooded islands. This reach is very unstable laterally and during floods bank erosion provides the main source of sediment that is transported onto the fan below Vedder Crossing.

Channel Changes on the Vedder River

The Vedder channel is only about one century old, and since the initial channel shift around the turn of the century, it has been in a constant state of transition. Shortly after the shift, degradation appears to have taken place near the head of the fan while rapid channel widening occurred further downstream.

Major changes were induced by channelization on the lower part of the fan in the mid-1960's. This channelization initially caused degradation above the railway bridge and rapid aggradation in the reach immediately downstream. Eventually aggradation spread upstream into the channelized reach.

Sediment Transport on the Vedder River

Comparison with bedload measurements collected on the Vedder River and other gravel rivers showed the
original Einstein bedload equation, with slight modification, could provide reasonably accurate estimates of sediment transport. About one-third of the estimates fell within one-half to two times the measured results. It is believed the Einstein relation works better than other formulas because its empirical coefficients were derived from flume studies with gravel sediment. Also, it is the only relation that computes bedload by size fraction rather than depending on a representative \( D_{35} \) or \( D_{50} \) size.

The results of calculations showed that for the channel geometry existing between 1971-1975, nearly all of the sediment inflow through Vedder Crossing was deposited above the railway bridge.

**Flood Control on the Vedder River**

Based on the bedload transport calculations and on the observed sediment sorting that is apparent, there is no chance of constructing a channel that will "sluice" all of the sediment inflow at Vedder Crossing through the Vedder River and into the Fraser River. Therefore, to achieve long-term stabilization of the river, either upstream sediment control or dredging below Vedder Crossing will be required, regardless of the channel alignment of the lower river.

It was concluded that providing set-back dikes to control overbank flooding and river training to stabilize
the Vedder Channel could provide long-term flood protection if combined with sediment control or sediment removal.

It was also concluded that a main channel alignment could probably be established which would tend to localize deposition in wide reaches along the river. Therefore, sediment could be removed from these zones, while the channel in-between could remain relatively stable.

It was suggested that the main channel should be able to contain moderate floods having return periods of about two years (maximum instantaneous). This would correspond roughly to a five year maximum daily flood. For more extreme floods, the flow should be allowed to spill overbank and be carried by the side channels and flood plain inside the set-back dikes.
BIBLIOGRAPHY


Le Baron, J. F., "Report on the Reclamation of the Sumas Prairies and Drainage of Sumas Lake in the District of New Westminster", 1908, B.C. Water Resources Service Library, Victoria, B.C.


Meyer-Peter, E., Muller, P., "Formula for Bed Load Transport", International Association for Hydraulic Research, 1948.


TABLES
<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Work Carried Out</th>
<th>Approximate Cost</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1921</td>
<td>Vedder Crossing</td>
<td>construction of a rock crib near Vedder Crossing</td>
<td>$ -</td>
<td>Marr, 1964</td>
</tr>
<tr>
<td>1922-1924</td>
<td>Vedder River below Yarrow</td>
<td>construction of Vedder Canal, north and south Vedder dykes, Sumas dam and drainage works</td>
<td>$3,400,000</td>
<td>Sinclair, 1961</td>
</tr>
<tr>
<td>1930</td>
<td>Downstream of B.C. Railway bridge</td>
<td>cutoff meander bend</td>
<td>$ -</td>
<td>Sinclair, 1961</td>
</tr>
<tr>
<td>1951</td>
<td>Near B.C. Railway bridge</td>
<td>channel clearing</td>
<td>$ 7,338</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1952</td>
<td>One mile west of Vedder Crossing</td>
<td>bank protection</td>
<td>$ 1,722</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1952</td>
<td>Near Browne Road</td>
<td>gravel removal</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1953</td>
<td>Vedder River</td>
<td>channel clearing</td>
<td>$ 2,756</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1954</td>
<td>Browne Road to Lickman Road</td>
<td>channel clearing</td>
<td>$ 4,537</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1955</td>
<td>Keith Wilson Road - Peache Road</td>
<td>channel improvement, bank protection</td>
<td>$ 13,152</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1956</td>
<td>Browne Road - Vedder Crossing</td>
<td>wing dam, bank protection</td>
<td>$ 14,852</td>
<td>Township of Chilliwack</td>
</tr>
</tbody>
</table>
### TABLE 1 (cont'd)

<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Work Carried Out</th>
<th>Approximate Cost</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1957</td>
<td>Vedder River</td>
<td>- gravel removal</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1958</td>
<td>Lickman-Webster Road</td>
<td>- bank protection</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1961</td>
<td>Peache Road - Lickman Road</td>
<td>- 2,000 feet of bank protection</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td></td>
<td>Near Hopedale Road</td>
<td>- bank protection</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td></td>
<td>Browne Road</td>
<td>- bank protection</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td></td>
<td>Above Vedder Crossing</td>
<td>- channel clearing</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1962</td>
<td>B.C. Railway bridge to Ford Road</td>
<td>- bank protection and dyke construction</td>
<td>$ 14,800</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1963</td>
<td>B.C. Railway bridge to Ford Road</td>
<td>- bank protection and dyke construction</td>
<td>$ 25,726</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1964</td>
<td>Browne Road</td>
<td>- dyking and gravel removal</td>
<td>$ 40,000</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1965</td>
<td>Near B.C. Railway bridge</td>
<td>- bank protection</td>
<td>$ 3,630</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1967</td>
<td>Hopedale Road</td>
<td>- dyke extended from railway bridge to Hopedale Road</td>
<td>$ 10,135</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1968</td>
<td>Near B.C. Railway bridge</td>
<td>- bank protection</td>
<td>$ 15,000</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1969</td>
<td>Near B.C. Railway bridge</td>
<td>- bank protection</td>
<td>$ 15,000</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>Year</td>
<td>Location</td>
<td>Work Carried Out</td>
<td>Approximate Cost</td>
<td>Reference</td>
</tr>
<tr>
<td>------</td>
<td>--------------</td>
<td>--------------------------------------------</td>
<td>------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td>1974</td>
<td>Peache Road</td>
<td>- riprap construction</td>
<td>$ -</td>
<td>Township of Chilliwack</td>
</tr>
<tr>
<td>1975</td>
<td>Vedder River</td>
<td>- channel clearing of debris</td>
<td>$ 100,000</td>
<td>B.C. Water Resources Service, 1976</td>
</tr>
<tr>
<td>1976</td>
<td>Vedder River</td>
<td>Phase I Flood Control Works</td>
<td>$ 473,000</td>
<td>B.C. Water Resources Service, 1976</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- scalping of gravel bars</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>- raising banks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1976</td>
<td>Vedder River</td>
<td>Phase II Flood Control Works</td>
<td>$ 682,000</td>
<td>Peters, 1978</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- dredging of 750,000 cubic yards of gravel</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gauge Location</td>
<td>Gauge #</td>
<td>Drainage Area (mi$^2$)</td>
<td>Long Term Mean (cfs)</td>
<td>Basin Yield (in/yr)</td>
</tr>
<tr>
<td>------------------------</td>
<td>----------</td>
<td>------------------------</td>
<td>----------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>Vedder Crossing</td>
<td>8MH1</td>
<td>474</td>
<td>2,440</td>
<td>69.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Below Sless Creek</td>
<td>8MH55</td>
<td>323</td>
<td>1,500</td>
<td>65.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Above Sless Creek</td>
<td>8MH103</td>
<td>249</td>
<td>1,350</td>
<td>73.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chilliwack Lake Outlet</td>
<td>8MH16</td>
<td>127</td>
<td>679</td>
<td>72.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## TABLE 3

### MAJOR FLOODS ON CHILLIWACK RIVER

<table>
<thead>
<tr>
<th>Year</th>
<th>Date</th>
<th>Discharge at Vedder Crossing (cfs)</th>
<th>Max. One Day Precipitation (inches)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1875</td>
<td>unknown</td>
<td>unknown</td>
<td>unknown</td>
<td>channel shifted down Vedder Creek</td>
</tr>
<tr>
<td>1898</td>
<td>unknown</td>
<td>unknown</td>
<td>unknown</td>
<td>major shifting reported</td>
</tr>
<tr>
<td>1906</td>
<td>Oct</td>
<td>19,000</td>
<td>unknown</td>
<td>reported by Le Baron (1908)</td>
</tr>
<tr>
<td>1917</td>
<td>Dec 29</td>
<td>27,000</td>
<td>unknown</td>
<td>bridge at Vedder Crossing destroyed</td>
</tr>
<tr>
<td>1921</td>
<td>Dec 12</td>
<td>19,000</td>
<td>unknown</td>
<td>no damage reported</td>
</tr>
<tr>
<td>1924</td>
<td>Feb 12</td>
<td>17,600</td>
<td>unknown</td>
<td>no damage reported</td>
</tr>
<tr>
<td>1928</td>
<td>Jan 12</td>
<td>17,200</td>
<td>unknown</td>
<td>no damage reported</td>
</tr>
<tr>
<td>1932</td>
<td>unknown</td>
<td>unknown</td>
<td>1.1/3.11 (4) Nov 12</td>
<td>railway embankment overtopped (Marr, 1964)</td>
</tr>
<tr>
<td>1935</td>
<td>Jan 26</td>
<td>unknown</td>
<td>3.50 Jan 26</td>
<td>extensive flooding to Sumas reported</td>
</tr>
<tr>
<td>1948</td>
<td>Jun 7</td>
<td>G.H. = 12.2 ft (2) snowmelt flood</td>
<td>snowmelt flood</td>
<td>railway embankment overtopped (Marr, 1964)</td>
</tr>
<tr>
<td>1949</td>
<td>Nov 27</td>
<td>G.H. = 12.9 ft (2)</td>
<td>1.47 Nov 27</td>
<td>no damage reported</td>
</tr>
<tr>
<td>1951</td>
<td>Feb 10</td>
<td>G.H. = 13.5 ft (2)</td>
<td>4.55 Feb 3</td>
<td>bridge at Vedder Crossing destroyed</td>
</tr>
<tr>
<td>1955</td>
<td>Nov 3</td>
<td>15,400</td>
<td>4.56 Nov 2</td>
<td>no damage reported</td>
</tr>
</tbody>
</table>
### TABLE 3

**MAJOR FLOODS ON CHILLIWACK RIVER**

*(CONTINUED)*

<table>
<thead>
<tr>
<th>Year</th>
<th>Date</th>
<th>Discharge at Vedder Crossing (cfs)</th>
<th>Max. One Day Precipitation (inches)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1968</td>
<td>Jun 2</td>
<td>15,900</td>
<td>2.77</td>
<td>no damage reported</td>
</tr>
<tr>
<td></td>
<td>Jun 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1975</td>
<td>Dec 3</td>
<td>18,700</td>
<td>4.05</td>
<td>railway embankment failed, Yarrow flooded</td>
</tr>
<tr>
<td></td>
<td>Dec 1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:**

(1) All discharges are mean daily flows.

(2) Gauge heights shown for 1948, 1949 and 1951 were recorded at Vedder Crossing.

(3) All precipitation values except 1932 recorded at Chilliwack.

(4) Precipitation values shown for 1932: 1.10 inches at Chilliwack

3.11 inches at Cultus Lake
<table>
<thead>
<tr>
<th>Year</th>
<th>Winter Flood (cfs)</th>
<th>Date</th>
<th>Spring Flood (cfs)</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1976/77</td>
<td>10,200</td>
<td>18/01/77</td>
<td>5,730</td>
<td>07/06/77</td>
</tr>
<tr>
<td>1975/76</td>
<td>18,700</td>
<td>03/12/75</td>
<td>9,490</td>
<td>19/06/76</td>
</tr>
<tr>
<td>1974/75</td>
<td>5,780</td>
<td>21/12/74</td>
<td>9,380</td>
<td>05/06/75</td>
</tr>
<tr>
<td>1973/74</td>
<td>8,440</td>
<td>16/01/74</td>
<td>12,800</td>
<td>19/06/74</td>
</tr>
<tr>
<td>1972/73</td>
<td>7,840</td>
<td>26/12/72</td>
<td>6,620</td>
<td>18/05/73</td>
</tr>
<tr>
<td>1971/72</td>
<td>7,590</td>
<td>17/03/72</td>
<td>12,700</td>
<td>09/06/72</td>
</tr>
<tr>
<td>1970/71</td>
<td>12,600</td>
<td>31/01/71</td>
<td>8,800</td>
<td>13/05/71</td>
</tr>
<tr>
<td>1969/70</td>
<td>5,240</td>
<td>23/09/69</td>
<td>7,680</td>
<td>03/06/70</td>
</tr>
<tr>
<td>1968/69</td>
<td>8,800</td>
<td>17/09/68</td>
<td>9,540</td>
<td>24/05/69</td>
</tr>
<tr>
<td>1967/68</td>
<td>12,000</td>
<td>31/10/67</td>
<td>15,400</td>
<td>02/06/68</td>
</tr>
<tr>
<td>1966/67</td>
<td>11,600</td>
<td>16/12/66</td>
<td>10,400</td>
<td>20/06/67</td>
</tr>
<tr>
<td>1965/66</td>
<td>5,240</td>
<td>04/11/65</td>
<td>6,510</td>
<td>17/06/66</td>
</tr>
<tr>
<td>1964/65</td>
<td>7,030</td>
<td>02/10/64</td>
<td>6,200</td>
<td>11/06/65</td>
</tr>
<tr>
<td>1963/64</td>
<td>13,000</td>
<td>26/11/63</td>
<td>8,860</td>
<td>10/06/64</td>
</tr>
<tr>
<td>1962/63</td>
<td>7,660</td>
<td>20/11/62</td>
<td>4,870</td>
<td>12/06/63</td>
</tr>
<tr>
<td>1961/62</td>
<td>9,280</td>
<td>03/01/62</td>
<td>5,740</td>
<td>25/06/62</td>
</tr>
<tr>
<td>1960/61</td>
<td>9,310</td>
<td>15/01/61</td>
<td>9,240</td>
<td>18/06/61</td>
</tr>
<tr>
<td>1959/60</td>
<td>8,930</td>
<td>24/11/59</td>
<td>6,570</td>
<td>02/06/60</td>
</tr>
<tr>
<td>1958/59</td>
<td>8,810</td>
<td>10/10/58</td>
<td>9,970</td>
<td>29/04/59</td>
</tr>
<tr>
<td>1957/58</td>
<td>4,760</td>
<td>17/01/58</td>
<td>8,150</td>
<td>28/05/58</td>
</tr>
<tr>
<td>1956/57</td>
<td>9,300</td>
<td>20/10/56</td>
<td>6,970</td>
<td>08/05/57</td>
</tr>
<tr>
<td>1955/56</td>
<td>15,900</td>
<td>03/11/55</td>
<td>9,700</td>
<td>10/06/56</td>
</tr>
<tr>
<td>1954/55</td>
<td>10,600</td>
<td>22/11/54</td>
<td>9,720</td>
<td>12/06/55</td>
</tr>
<tr>
<td>1953/54</td>
<td>12,400</td>
<td>31/10/53</td>
<td>9,800</td>
<td>02/07/54</td>
</tr>
<tr>
<td>1952/53</td>
<td>5,930</td>
<td>12/01/53</td>
<td>7,190</td>
<td>13/06/53</td>
</tr>
<tr>
<td>1951/52</td>
<td>3,730</td>
<td>20/10/51</td>
<td>6,770</td>
<td>17/05/52</td>
</tr>
<tr>
<td>1950/51</td>
<td>20,000</td>
<td>10/02/51</td>
<td>6,320</td>
<td>23/05/51</td>
</tr>
<tr>
<td>Year</td>
<td>Winter Flood (cfs)</td>
<td>Date</td>
<td>Spring Flood (cfs)</td>
<td>Date</td>
</tr>
<tr>
<td>----------</td>
<td>--------------------</td>
<td>----------</td>
<td>--------------------</td>
<td>----------</td>
</tr>
<tr>
<td>1929/30</td>
<td>10,800</td>
<td>10/02/30</td>
<td>6,330</td>
<td>10/06/30</td>
</tr>
<tr>
<td>1928/29</td>
<td>8,090</td>
<td>09/10/29</td>
<td>6,160</td>
<td>22/05/29</td>
</tr>
<tr>
<td>1927/28</td>
<td>17,200</td>
<td>12/01/28</td>
<td>10,080</td>
<td>23/05/28</td>
</tr>
<tr>
<td>1926/27</td>
<td>4,430</td>
<td>10/12/26</td>
<td>11,000</td>
<td>07/06/27</td>
</tr>
<tr>
<td>1925/26</td>
<td>6,790</td>
<td>11/12/25</td>
<td>3,720</td>
<td>27/05/26</td>
</tr>
<tr>
<td>1924/25</td>
<td>16,900</td>
<td>12/12/24</td>
<td>11,600</td>
<td>20/05/25</td>
</tr>
<tr>
<td>1923/24</td>
<td>17,600</td>
<td>12/02/24</td>
<td>10,700</td>
<td>12/05/24</td>
</tr>
<tr>
<td>1922/23</td>
<td>10,700</td>
<td>24/12/22</td>
<td>9,340</td>
<td>09/06/23</td>
</tr>
<tr>
<td>1921/22</td>
<td>19,000</td>
<td>12/12/21</td>
<td>12,100</td>
<td>03/06/22</td>
</tr>
<tr>
<td>1920/21</td>
<td>10,600</td>
<td>05/10/20</td>
<td>10,400</td>
<td>07/06/21</td>
</tr>
<tr>
<td>1918/19</td>
<td>-</td>
<td>-</td>
<td>10,100</td>
<td>23/05/19</td>
</tr>
<tr>
<td>1917/18</td>
<td>27,000</td>
<td>29/12/17</td>
<td>8,170</td>
<td>09/06/18</td>
</tr>
<tr>
<td>1916/17</td>
<td>4,870</td>
<td>09/11/16</td>
<td>8,000</td>
<td>16/06/17</td>
</tr>
<tr>
<td>1915/16</td>
<td>9,350</td>
<td>28/10/15</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1914/15</td>
<td>5,000</td>
<td>11/11/14</td>
<td>8,700</td>
<td>03/04/15</td>
</tr>
<tr>
<td>1913/14</td>
<td>20,000</td>
<td>06/01/14</td>
<td>5,800</td>
<td>15/05/14</td>
</tr>
<tr>
<td>1912/13</td>
<td>10,000</td>
<td>17/02/13</td>
<td>12,200</td>
<td>02/06/13</td>
</tr>
<tr>
<td>1911/12</td>
<td>3,650</td>
<td>25/01/12</td>
<td>10,500</td>
<td>20/06/12</td>
</tr>
<tr>
<td>1906/07</td>
<td>19,000</td>
<td>unknown</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: Winter flood period September 1 - March 31
Spring flood period April 1 - July 31
All discharges are mean daily flows
<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Probability of Exceedance (%)</th>
<th>Mean Snowmelt (cfs)</th>
<th>Daily Discharge Rainstorm (cfs)</th>
<th>Annual (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1</td>
<td>15,000</td>
<td>32,000</td>
<td>32,000</td>
</tr>
<tr>
<td>50</td>
<td>2</td>
<td>14,200</td>
<td>27,500</td>
<td>27,500</td>
</tr>
<tr>
<td>20</td>
<td>5</td>
<td>13,000</td>
<td>22,500</td>
<td>22,500</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>12,000</td>
<td>18,700</td>
<td>18,800</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>10,800</td>
<td>15,000</td>
<td>15,100</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>8,600</td>
<td>9,800</td>
<td>11,200</td>
</tr>
<tr>
<td>1.25</td>
<td>80</td>
<td>6,700</td>
<td>6,400</td>
<td>8,600</td>
</tr>
</tbody>
</table>

Note: Snowmelt floods considered to occur between April 1 - July 31.

Rainstorm floods considered to occur between August 1 - March 31.

Annual flood frequency determined by combining the two seasonal relations.
TABLE 6
MAXIMUM INSTANTANEOUS FLOOD FREQUENCY ESTIMATES
Chilliwack River at Vedder Crossing

<table>
<thead>
<tr>
<th>Return Period (Years)</th>
<th>Probability of Flow Exceedance (%)</th>
<th>Snowmelt Flood</th>
<th>Rainstorm Flood</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>15,000</td>
<td>22,500</td>
</tr>
<tr>
<td>50</td>
<td>2</td>
<td>14,200</td>
<td>21,300</td>
</tr>
<tr>
<td>20</td>
<td>5</td>
<td>13,000</td>
<td>19,500</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>12,000</td>
<td>18,000</td>
</tr>
<tr>
<td>5</td>
<td>20</td>
<td>10,800</td>
<td>16,200</td>
</tr>
<tr>
<td>2</td>
<td>50</td>
<td>8,600</td>
<td>12,900</td>
</tr>
<tr>
<td>1.25</td>
<td>80</td>
<td>6,700</td>
<td>10,100</td>
</tr>
</tbody>
</table>
TABLE 7

FREQUENCY OF HISTORICAL FLOODS

<table>
<thead>
<tr>
<th>Year</th>
<th>Date</th>
<th>Mean Daily Discharge (cfs)</th>
<th>Annual Probability of Exceedance (%)</th>
<th>Annual Return Period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1975</td>
<td>Dec 3</td>
<td>18,700</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>1955</td>
<td>Nov 3</td>
<td>15,900</td>
<td>17</td>
<td>6</td>
</tr>
<tr>
<td>1928</td>
<td>Jan 12</td>
<td>17,200</td>
<td>13</td>
<td>7.7</td>
</tr>
<tr>
<td>1924</td>
<td>Feb 12</td>
<td>17,600</td>
<td>12.5</td>
<td>8</td>
</tr>
<tr>
<td>1921</td>
<td>Dec 12</td>
<td>19,000</td>
<td>9.5</td>
<td>10.5</td>
</tr>
<tr>
<td>1917</td>
<td>Dec 29</td>
<td>27,000</td>
<td>2.5</td>
<td>50.40</td>
</tr>
<tr>
<td>1914</td>
<td>Jan 6</td>
<td>20,000</td>
<td>8</td>
<td>12.5</td>
</tr>
</tbody>
</table>
### TABLE 8
SUMMARY OF MAPS/AIRPHOTOS ABOVE VEDDER CROSSING

<table>
<thead>
<tr>
<th>Extent</th>
<th>Date</th>
<th>Scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vedder Crossing - Liumchen Cr.</td>
<td>1886</td>
<td>map</td>
</tr>
<tr>
<td>Vedder Crossing - Liumchen Cr.</td>
<td>1905</td>
<td>1:70560 map</td>
</tr>
<tr>
<td>Liumchen Cr. - Tamahi Cr.</td>
<td>1910</td>
<td>1:15840</td>
</tr>
<tr>
<td>Vedder Crossing - Tamahi Cr.</td>
<td>July 15, 1940</td>
<td>1:20676 BC 209 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Tamahi Cr.</td>
<td>Sept. 1952</td>
<td>1:39600 BC 1622 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Tamahi Cr.</td>
<td>Oct. 24, 1958</td>
<td>1:6000 BC 5005 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Tamahi Cr.</td>
<td>1961</td>
<td>1:25000</td>
</tr>
<tr>
<td>Vedder Crossing - Chilliwack Lake</td>
<td>1964</td>
<td>1:50000 map</td>
</tr>
<tr>
<td>Vedder Crossing - Chilliwack Lake</td>
<td>Sept. 4, 1966</td>
<td>1:38520 BC 5217 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Tamahi Cr.</td>
<td>May 18, 1968</td>
<td>1:17124 BC 7057 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Liumchen Cr.</td>
<td>March 11, 1969</td>
<td>1:11556 BC 5318 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Liumchen Cr.</td>
<td>March 19, 1971</td>
<td>1:33432 BC 5406 / Photo</td>
</tr>
<tr>
<td>Vedder Crossing - Liumchen Cr.</td>
<td>June 19, 1976</td>
<td>1:19400 BC 5714 / Photo</td>
</tr>
</tbody>
</table>
### TABLE 9
BEDMATERIAL DATA ABOVE VEDDER CROSSING

<table>
<thead>
<tr>
<th>Distance From Vedder Crossing Bridge (feet)</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>Location</th>
<th>Grain Size Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>D&lt;sub&gt;90&lt;/sub&gt;</td>
</tr>
<tr>
<td>2 + 50</td>
<td>29</td>
<td>P</td>
<td>Right side bar near Vedder Crossing</td>
<td>85</td>
</tr>
<tr>
<td>2 + 50</td>
<td>30</td>
<td>P</td>
<td>Right side bar near Vedder Crossing</td>
<td>30</td>
</tr>
<tr>
<td>7 + 50</td>
<td>22</td>
<td>P</td>
<td>Right side bar near Vedder Crossing</td>
<td>48</td>
</tr>
<tr>
<td>8 + 50</td>
<td>23</td>
<td>P</td>
<td>Large island near Vedder Crossing</td>
<td>77</td>
</tr>
<tr>
<td>11 + 00</td>
<td>26</td>
<td>P</td>
<td>Large island near Vedder Crossing</td>
<td>68</td>
</tr>
<tr>
<td>12 + 00</td>
<td>25</td>
<td>P</td>
<td>Large island near Vedder Crossing</td>
<td>47</td>
</tr>
<tr>
<td>15 + 00</td>
<td>24</td>
<td>P</td>
<td>Left bank near Sweltzer Creek</td>
<td>55</td>
</tr>
<tr>
<td>19 + 00</td>
<td>5</td>
<td>P</td>
<td>Right bank side bar</td>
<td>60</td>
</tr>
<tr>
<td>31 + 00</td>
<td>6</td>
<td>P</td>
<td>Mid-channel bar</td>
<td>65</td>
</tr>
<tr>
<td>31 + 00</td>
<td>8</td>
<td>B</td>
<td>Right bank side bar</td>
<td>55</td>
</tr>
<tr>
<td>34 + 00</td>
<td>7</td>
<td>P</td>
<td>Mid-channel bar</td>
<td>100</td>
</tr>
<tr>
<td>55 + 50</td>
<td>32</td>
<td>P</td>
<td>Mid-channel bar</td>
<td>63</td>
</tr>
<tr>
<td>55 + 50</td>
<td>31</td>
<td>P</td>
<td>Mid-channel bar</td>
<td>78</td>
</tr>
<tr>
<td>73 + 50</td>
<td>37</td>
<td>P</td>
<td>Diagonal near mobile home development</td>
<td>37</td>
</tr>
<tr>
<td>78 + 50</td>
<td>42</td>
<td>P</td>
<td>D/S end of side bar</td>
<td>26</td>
</tr>
<tr>
<td>78 + 50</td>
<td>42A</td>
<td>B</td>
<td>D/Send of side bar</td>
<td>30</td>
</tr>
<tr>
<td>81 + 00</td>
<td>41</td>
<td>P</td>
<td>Middle of side bar</td>
<td>45</td>
</tr>
<tr>
<td>81 + 00</td>
<td>40</td>
<td>P</td>
<td>Middle of side bar</td>
<td>65</td>
</tr>
<tr>
<td>81 + 00</td>
<td>39</td>
<td>P</td>
<td>Upstream end of side bar</td>
<td>95</td>
</tr>
<tr>
<td>104 + 00</td>
<td>34</td>
<td>P</td>
<td>Small mid-channel bar near Liumchen Creek</td>
<td>47</td>
</tr>
<tr>
<td>125 + 00</td>
<td>36</td>
<td>P</td>
<td>Right side bar near Liumchen Creek</td>
<td>87</td>
</tr>
<tr>
<td>260 + 50</td>
<td>59</td>
<td>T</td>
<td>Right bank Osborne Rd.</td>
<td>160</td>
</tr>
<tr>
<td>265 + 00</td>
<td>64</td>
<td>T</td>
<td>Tamahi Creek confluence</td>
<td>220</td>
</tr>
<tr>
<td>445 + 00</td>
<td>48</td>
<td>T</td>
<td>Point bar 1.2 miles above Tamahi Creek</td>
<td>188</td>
</tr>
<tr>
<td>445 + 00</td>
<td>48A</td>
<td>T</td>
<td>Point bar 1.2 miles above Tamahi Creek</td>
<td>157</td>
</tr>
<tr>
<td>449 + 00</td>
<td>45</td>
<td>P</td>
<td>U/S end of point bar 1.25 miles U/S Tamahi Creek</td>
<td>87</td>
</tr>
<tr>
<td>540 + 00</td>
<td>59</td>
<td>T</td>
<td>At Borden Creek confluence</td>
<td>200</td>
</tr>
<tr>
<td>744 + 00</td>
<td>60</td>
<td>T</td>
<td>At Slesse Creek confluence</td>
<td>500</td>
</tr>
<tr>
<td>744 + 00</td>
<td>61</td>
<td>T</td>
<td>Slesse Creek confluence</td>
<td>200</td>
</tr>
<tr>
<td>1268 + 00</td>
<td>62</td>
<td>T</td>
<td>Near Centre Creek</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>63</td>
<td>T</td>
<td>Near Post Creek</td>
<td>400</td>
</tr>
</tbody>
</table>

**Notation:**

1. **Sample type**  
   - P = Photographic grid  
   - T = Tape grid  
   - B = Bulk sample

2. D/S = downstream  
   U/S = upstream
## TABLE 10
SUMMARY OF HYDROLOGIC DATA ALONG CHILLIWACK RIVER

<table>
<thead>
<tr>
<th>Reach</th>
<th>Length (mi)</th>
<th>Drainage Area at D/S end of Reach (mi²)</th>
<th>Major Tributaries in Reach</th>
<th>Gauging Station in Reach</th>
<th>Drainage Area at Gauge (mi²)</th>
<th>Period of Record</th>
<th>Long Term Mean Flow (cfs)</th>
<th>2 yr</th>
<th>5 yr</th>
<th>10 yr</th>
<th>20 yr</th>
<th>Maximum Recorded Flow (cfs)</th>
<th>Minimum Recorded Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lake Outlet to Chipmunk Cr.</td>
<td>10.7</td>
<td>234</td>
<td>Post Cr. Centre Cr. Nesakwatch Cr.</td>
<td>8MH16</td>
<td>127</td>
<td>1923-1978</td>
<td>680</td>
<td>2300</td>
<td>2900</td>
<td>3200</td>
<td>3400</td>
<td>104</td>
<td>10/06/72</td>
</tr>
<tr>
<td>Chipmunk Cr. to Slesse Cr.</td>
<td>4.0</td>
<td>262</td>
<td>Chipmunk Cr.</td>
<td>8MH103</td>
<td>249</td>
<td>1963-1978</td>
<td>1350</td>
<td>4700</td>
<td>5600</td>
<td>6100</td>
<td>6500</td>
<td>10000</td>
<td>04/12/75</td>
</tr>
<tr>
<td>Slesse Cr. to Tamahi Cr.</td>
<td>7.7</td>
<td>355</td>
<td>Slesse Cr. Borden Cr.</td>
<td>8MH55</td>
<td>323</td>
<td>1966-1962</td>
<td>1710</td>
<td>6500</td>
<td>7800</td>
<td>8600</td>
<td>9500</td>
<td>9000</td>
<td>02/12/58</td>
</tr>
<tr>
<td>Tamahi Cr. to Ryder Cr.</td>
<td>2.1</td>
<td>415</td>
<td>Tamahi Cr.</td>
<td>None</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ryder Cr. to Vedder Crossing</td>
<td>4.5</td>
<td>474</td>
<td>Ryder Cr. Liunchen Cr. Sweltzer R.</td>
<td>8MH01</td>
<td>474</td>
<td>1911-1978</td>
<td>2440</td>
<td>11200</td>
<td>15100</td>
<td>18800</td>
<td>22500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reach</td>
<td>Valley Description</td>
<td>Width of Valley Floor (mi)</td>
<td>Description of Valley Flat</td>
<td>Relation of Channel to Valley</td>
<td>Channel Bed</td>
<td>Channel Banks</td>
<td>Per Cent Alluvial Left Bank/Right Bank</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------------------</td>
<td>----------------------------</td>
<td>----------------------------</td>
<td>-----------------------------</td>
<td>-------------------------------</td>
<td>-----------------</td>
<td>-------------------</td>
<td>----------------------------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lake Outlet to Chipmunk Cr.</td>
<td>broad glaciated mountain valley</td>
<td>0.5</td>
<td>indefinite</td>
<td>has degraded into valley train</td>
<td>continuously confined</td>
<td>angular boulders and cobbles</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chipmunk Cr. to Slesse Cr.</td>
<td>extensive valley train on valley floor</td>
<td>0.2</td>
<td>indefinite</td>
<td>entrenched and confined</td>
<td>shallow deposits overlying gravel</td>
<td>bedrock or cobbles and gravel</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slesse Cr. to Tamahi Cr.</td>
<td>deep stream-cut in mountain valley</td>
<td>0.2</td>
<td>fragmentary</td>
<td>frequently confined by bedrock or gravel</td>
<td>shallow gravel</td>
<td>bedrock or clay cobbles gravel</td>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tamahi Cr. to Ryder Cr.</td>
<td>stream cut</td>
<td>0.3</td>
<td>fragmentary narrow forested</td>
<td>channel flows over boulder debris</td>
<td>angular boulders, cobbles gravel</td>
<td>boulders or gravel, cobbles gravel</td>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ryder Cr. to Vedder Crossing</td>
<td>wide alluvial plain on south bank</td>
<td>0.7</td>
<td>continuous and broad forested</td>
<td>occasionally confined along north bank</td>
<td>predominantly gravel</td>
<td>predominantly gravel</td>
<td>90</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reach</td>
<td>Channel Pattern</td>
<td>Islands</td>
<td>Bars</td>
<td>Lateral Activity</td>
<td>Average Channel Slope</td>
<td>Discharge (cfs)</td>
<td>Top Width (ft)</td>
<td>Mean Depth (ft)</td>
<td>Mean Velocity (ft/sec)</td>
<td>Number of Cross Sections</td>
<td>Data Source</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------------</td>
<td>-----------------</td>
<td>---------</td>
<td>----------</td>
<td>------------------</td>
<td>------------------------</td>
<td>-----------------</td>
<td>----------------</td>
<td>------------------</td>
<td>--------------------------</td>
<td>--------------------------</td>
<td>-------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lake Outlet to Chipmunk Cr</td>
<td>irregular slightly sinuous single channel</td>
<td>few</td>
<td>few</td>
<td>stable</td>
<td>0.017</td>
<td>1350</td>
<td>68</td>
<td>6.0</td>
<td>3.3</td>
<td>gauge rating curve 1</td>
<td>B.C. W.R.S.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chipmunk Cr to Slesse Cr</td>
<td>irregular single channel</td>
<td>none</td>
<td>none</td>
<td>stable</td>
<td>0.015</td>
<td>4700</td>
<td>73</td>
<td>8.5</td>
<td>7.5</td>
<td>gauge rating curve 1</td>
<td>B.C. W.R.S.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slesse Cr to Tamahi Cr</td>
<td>confined meanders single channel</td>
<td>few</td>
<td>prominent point bars</td>
<td>erosion causing slumps in valley wall</td>
<td>0.011</td>
<td>6500</td>
<td>115</td>
<td>6.6</td>
<td>8.5</td>
<td>gauge rating curve 1</td>
<td>B.C. W.R.S.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tamahi Cr to Ryder Cr</td>
<td>irregular single channel</td>
<td>few</td>
<td>diagonal bars riffles</td>
<td>stable</td>
<td>0.020</td>
<td>7800</td>
<td>116</td>
<td>7.1</td>
<td>9.5</td>
<td>gauge rating curve 1</td>
<td>B.C. W.R.S.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ryder Cr to Vedder Crossing</td>
<td>split/ braided</td>
<td>frequent wooded islands</td>
<td>frequent mid-channel irregular shifts, avulsions</td>
<td>frequent</td>
<td>0.0063</td>
<td>2440</td>
<td>217</td>
<td>2.1</td>
<td>5.4</td>
<td>B.C. W.R.S. sections 4</td>
<td>B.C. W.R.S.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 13

INCIPIENT MOTION FOR BED MATERIAL ALONG CHILLIWACK RIVER

<table>
<thead>
<tr>
<th>Location</th>
<th>Flow Condition</th>
<th>Discharge (cfs)</th>
<th>Mean Velocity (ft/s)</th>
<th>Mean Depth (ft)</th>
<th>Froude Number</th>
<th>Maximum Particle Size Moved (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chilliwack River above</td>
<td>mean daily</td>
<td>1350</td>
<td>3.3</td>
<td>6.0</td>
<td>0.24</td>
<td>5</td>
</tr>
<tr>
<td>Slesse Cr. gauge 8MH103</td>
<td>2 yr flood</td>
<td>4700</td>
<td>7.5</td>
<td>8.5</td>
<td>0.45</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>5 yr flood</td>
<td>5600</td>
<td>8.1</td>
<td>9.0</td>
<td>0.48</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td>10 yr flood</td>
<td>6100</td>
<td>8.6</td>
<td>9.2</td>
<td>0.50</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>20 yr flood</td>
<td>6500</td>
<td>8.9</td>
<td>9.5</td>
<td>0.51</td>
<td>80</td>
</tr>
<tr>
<td>Chilliwack River below</td>
<td>mean daily</td>
<td>1710</td>
<td>3.4</td>
<td>4.8</td>
<td>0.27</td>
<td>6</td>
</tr>
<tr>
<td>Slesse Cr. gauge 8MH55</td>
<td>2 yr flood</td>
<td>6500</td>
<td>8.5</td>
<td>6.6</td>
<td>0.58</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>5 yr flood</td>
<td>7800</td>
<td>9.5</td>
<td>7.1</td>
<td>0.63</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>10 yr flood</td>
<td>8600</td>
<td>10.2</td>
<td>7.2</td>
<td>0.65</td>
<td>96</td>
</tr>
<tr>
<td></td>
<td>20 yr flood</td>
<td>9500</td>
<td>11.0</td>
<td>7.32</td>
<td>0.72</td>
<td>103</td>
</tr>
</tbody>
</table>

Note: Slope at gauge 8MH103 assumed 0.015

Slope at gauge 8MH55 assumed 0.017

Incipient motion computed using critical velocity relation of Neill (1976, 1978)
### TABLE 14

**SUMMARY OF WIDTH CHANGES ABOVE VEDDER CROSSING**

<table>
<thead>
<tr>
<th>Year</th>
<th>Average Active Channel Width (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1940</td>
<td>790</td>
</tr>
<tr>
<td>1952</td>
<td>660</td>
</tr>
<tr>
<td>1958</td>
<td>660</td>
</tr>
<tr>
<td>1968</td>
<td>560</td>
</tr>
<tr>
<td>1971</td>
<td>430</td>
</tr>
<tr>
<td>1976</td>
<td>610</td>
</tr>
</tbody>
</table>

*Note: Channel width measured between Vedder Crossing and Liumchen Creek*
TABLE 15
SUMMARY OF CHANNEL CHANGES AND
SEDIMENT EROSION ABOVE VEDDER CROSSING

<table>
<thead>
<tr>
<th>Year</th>
<th>Approximate Floodingplain Reconstruction (Acres)</th>
<th>Apparent Erosion (Acres)</th>
<th>Sediment Volume Eroded (cubic yards)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1940-1952</td>
<td>92</td>
<td>39</td>
<td>634,000</td>
</tr>
<tr>
<td>1952-1958</td>
<td>21</td>
<td>22</td>
<td>356,000</td>
</tr>
<tr>
<td>1958-1971</td>
<td>91</td>
<td>34</td>
<td>541,000</td>
</tr>
<tr>
<td>1971-1976</td>
<td>Nil</td>
<td>47</td>
<td>637,000</td>
</tr>
</tbody>
</table>

Note: (1) measured between Vedder Crossing and Liumchen Creek
(2) bank height of eroded material assumed to be 10 feet
### Table 16

**SUMMARY OF CROSS SECTION DATA**

**BELOW VEDDER CROSSING**

<table>
<thead>
<tr>
<th>Section Designation</th>
<th>Surveyed by</th>
<th>Original Designation</th>
<th>Date of Survey</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.39</td>
<td>BCWRS</td>
<td>1</td>
<td>1958, 59, 63</td>
<td>below Hwy 1 bridge</td>
</tr>
<tr>
<td>0.98</td>
<td>BCWRS</td>
<td>2</td>
<td>1958, 59, 63</td>
<td>Vedder Canal</td>
</tr>
<tr>
<td>1.45</td>
<td>BCWRS</td>
<td>3</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>1.57</td>
<td>WSC</td>
<td>11</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>2.00</td>
<td>WSC</td>
<td>4</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>2.05</td>
<td>BCWRS</td>
<td>28</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.13</td>
<td>BCWRS</td>
<td>27</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.22</td>
<td>BCWRS</td>
<td>26</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.32</td>
<td>BCWRS</td>
<td>5</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>2.34</td>
<td>WSC</td>
<td>9</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>2.42</td>
<td>WSC</td>
<td>25</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.51</td>
<td>WSC</td>
<td>24</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.61</td>
<td>WSC</td>
<td>23</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.70</td>
<td>WSC</td>
<td>22</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.70</td>
<td>BCWRS</td>
<td>6</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>2.77</td>
<td>WSC</td>
<td>8</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>2.84</td>
<td>WSC</td>
<td>21</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>2.92</td>
<td>WSC</td>
<td>20</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.02</td>
<td>WSC</td>
<td>19</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.14</td>
<td>WSC</td>
<td>18</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.24</td>
<td>WSC</td>
<td>17</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.30</td>
<td>BCWRS</td>
<td>7</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>3.36</td>
<td>WSC</td>
<td>16</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.44</td>
<td>WSC</td>
<td>7</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>3.52</td>
<td>WSC</td>
<td>15</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.61</td>
<td>WSC</td>
<td>14</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>3.69</td>
<td>WSC</td>
<td>13</td>
<td>1975</td>
<td></td>
</tr>
<tr>
<td>Section Designation</td>
<td>Surveyed by</td>
<td>Original Designation</td>
<td>Date of Survey</td>
<td>Location</td>
</tr>
<tr>
<td>---------------------</td>
<td>-------------</td>
<td>---------------------</td>
<td>----------------</td>
<td>----------</td>
</tr>
<tr>
<td>3.84</td>
<td>WSC</td>
<td>12</td>
<td>1975</td>
<td>Cableway</td>
</tr>
<tr>
<td>3.84</td>
<td>BCWRS</td>
<td>8</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>3.91</td>
<td>WSC</td>
<td>6</td>
<td>1971, 72, 73</td>
<td>Railway Bridge</td>
</tr>
<tr>
<td>4.00</td>
<td>BCWRS</td>
<td>5</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>4.07</td>
<td>BCWRS</td>
<td>6</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>4.14</td>
<td>BCWRS</td>
<td>7</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>4.23</td>
<td>BCWRS</td>
<td>8</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>4.43</td>
<td>BCWRS</td>
<td>9</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>4.43</td>
<td>WSC</td>
<td>5</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>4.50</td>
<td>BCWRS</td>
<td>10</td>
<td>1975, 76</td>
<td>Hopedale Road</td>
</tr>
<tr>
<td>4.76</td>
<td>VCWRS</td>
<td>11</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>4.83</td>
<td>BCWRS</td>
<td>9</td>
<td>1958, 63</td>
<td></td>
</tr>
<tr>
<td>4.96</td>
<td>BCWRS</td>
<td>12</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>5.00</td>
<td>WSC</td>
<td>4</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>5.13</td>
<td>BCWRS</td>
<td>13</td>
<td>1975, 76</td>
<td>Browne Road</td>
</tr>
<tr>
<td>5.32</td>
<td>BCWRS</td>
<td>14</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>5.52</td>
<td>BCWRS</td>
<td>15</td>
<td>1975, 76</td>
<td>Ford Road</td>
</tr>
<tr>
<td>5.61</td>
<td>BCWRS</td>
<td>10</td>
<td>1958, 63</td>
<td></td>
</tr>
<tr>
<td>5.61</td>
<td>WSC</td>
<td>3</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>5.75</td>
<td>BCWRS</td>
<td>16</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>5.88</td>
<td>BCWRS</td>
<td>17</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>6.20</td>
<td>BCWRS</td>
<td>18</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>6.43</td>
<td>BCWRS</td>
<td>19</td>
<td>1975, 76</td>
<td></td>
</tr>
<tr>
<td>6.56</td>
<td>WSC</td>
<td>2</td>
<td>1971, 72, 73</td>
<td></td>
</tr>
<tr>
<td>6.61</td>
<td>BCWRS</td>
<td>20</td>
<td>1975, 76</td>
<td>Peache Road</td>
</tr>
<tr>
<td>6.68</td>
<td>BCWRS</td>
<td>11</td>
<td>1958, 59, 63</td>
<td></td>
</tr>
<tr>
<td>6.95</td>
<td>BCWRS</td>
<td>21</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>7.40</td>
<td>BCWRS</td>
<td>23</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>7.58</td>
<td>BCWRS</td>
<td>24</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>7.70</td>
<td>BCWRS</td>
<td>25</td>
<td>1976</td>
<td></td>
</tr>
<tr>
<td>7.70</td>
<td>WSC</td>
<td>1 1971, 72, 73</td>
<td>195</td>
<td>Vedder Crossing Road bridge</td>
</tr>
</tbody>
</table>
Table 17

WATER SURFACE SLOPE SURVEYS

<table>
<thead>
<tr>
<th>Reach</th>
<th>Mileage</th>
<th>Nov. 1963</th>
<th>April 1971</th>
<th>May 11, 1977</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>6880</td>
<td>1900</td>
<td>4000</td>
</tr>
<tr>
<td>Vedder Crossing</td>
<td>7.75</td>
<td>0.0042</td>
<td>0.0049</td>
<td>0.006</td>
</tr>
<tr>
<td>Webster Road</td>
<td>6.12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ford Road</td>
<td>5.61</td>
<td>0.0048</td>
<td>0.0032</td>
<td></td>
</tr>
<tr>
<td>Browne Road</td>
<td>5.02</td>
<td>0.0036</td>
<td>0.0043</td>
<td>0.0031</td>
</tr>
<tr>
<td>Railway Bridge</td>
<td>4.00</td>
<td>0.0026</td>
<td>0.0016</td>
<td>0.00226</td>
</tr>
<tr>
<td>Head of Canal</td>
<td>2.70</td>
<td>0.0016</td>
<td>0.0020</td>
<td></td>
</tr>
<tr>
<td>Vedder Canal</td>
<td>0.39</td>
<td>0.00032</td>
<td>0.0009</td>
<td></td>
</tr>
</tbody>
</table>
TABLE 18
BANKFULL PROPERTIES OF BRAIDED SUB-CHANNELS
ALONG THE VEDDER RIVER

<table>
<thead>
<tr>
<th>River</th>
<th>Mileage</th>
<th>Inactive Floodplain</th>
<th>Bankfull Level</th>
<th>Mean Bed Level</th>
<th>Mean Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4.83</td>
<td>41</td>
<td>40.1</td>
<td>36.9</td>
<td>3.6</td>
</tr>
<tr>
<td></td>
<td>5.68</td>
<td>55</td>
<td>53.5</td>
<td>51.4</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>5.88</td>
<td>60</td>
<td>60</td>
<td>57.5</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>6.20</td>
<td>69</td>
<td>69</td>
<td>64.6</td>
<td>4.4</td>
</tr>
<tr>
<td></td>
<td>6.43</td>
<td>71</td>
<td>70</td>
<td>67.8</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>6.61</td>
<td>77</td>
<td>76</td>
<td>72.8</td>
<td>3.2</td>
</tr>
<tr>
<td></td>
<td>6.68</td>
<td>82</td>
<td>80</td>
<td>76.5</td>
<td>3.5</td>
</tr>
<tr>
<td>Reach</td>
<td>Number of Samples Used</td>
<td>D&lt;sub&gt;90&lt;/sub&gt;</td>
<td>D&lt;sub&gt;65&lt;/sub&gt;</td>
<td>D&lt;sub&gt;50&lt;/sub&gt;</td>
<td>D&lt;sub&gt;35&lt;/sub&gt;</td>
</tr>
<tr>
<td>---------------------</td>
<td>------------------------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
<td>--------------</td>
</tr>
<tr>
<td>Vedder Canal</td>
<td>12 bars</td>
<td>45</td>
<td>15</td>
<td>8.8</td>
<td>4.5</td>
</tr>
<tr>
<td>1.45-2.42</td>
<td>12 pavement</td>
<td>54</td>
<td>32</td>
<td>21</td>
<td>15</td>
</tr>
<tr>
<td>Canal Head</td>
<td>14 bars</td>
<td>35</td>
<td>16</td>
<td>9.5</td>
<td>5.0</td>
</tr>
<tr>
<td>2.51-3.52</td>
<td>4 pavement</td>
<td>67</td>
<td>43</td>
<td>32</td>
<td>25</td>
</tr>
<tr>
<td>Railway Bridge</td>
<td>10 bars</td>
<td>54</td>
<td>31</td>
<td>21</td>
<td>13</td>
</tr>
<tr>
<td>3.61-4.43</td>
<td>7 pavement</td>
<td>80</td>
<td>60</td>
<td>46</td>
<td>38</td>
</tr>
<tr>
<td>Constriction</td>
<td>6 bars</td>
<td>94</td>
<td>50</td>
<td>34</td>
<td>20</td>
</tr>
<tr>
<td>5.0-5.61</td>
<td>6 pavement</td>
<td>100</td>
<td>60</td>
<td>45</td>
<td>34</td>
</tr>
<tr>
<td>Ford Rd-Peache Rd</td>
<td>4 bars</td>
<td>88</td>
<td>41</td>
<td>24</td>
<td>12</td>
</tr>
<tr>
<td>5.70-6.56</td>
<td>5 pavement</td>
<td>100</td>
<td>54</td>
<td>48</td>
<td>37</td>
</tr>
<tr>
<td>Vedder Crossing</td>
<td>8 bars</td>
<td>95</td>
<td>70</td>
<td>52</td>
<td>32</td>
</tr>
<tr>
<td>7.1-7.75</td>
<td>9 pavement</td>
<td>120</td>
<td>77</td>
<td>63</td>
<td>49</td>
</tr>
</tbody>
</table>
### TABLE 20

**DOWNSTREAM BED MATERIAL CHANGES ON GRAVEL RIVERS**

<table>
<thead>
<tr>
<th>River</th>
<th>Range of Particle Sizes (mm)</th>
<th>Coefficient of Size Reduction ( (a_d) ) ((\text{miles}^{-1}))</th>
<th>Distance Required to Reduce Grain Size by 50% ((\text{miles}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peace</td>
<td>--</td>
<td>0.008-0.010</td>
<td>86.6-69</td>
</tr>
<tr>
<td>Rhine</td>
<td>50-160</td>
<td>0.018</td>
<td>32.5</td>
</tr>
<tr>
<td>Mur</td>
<td>34-83</td>
<td>0.031</td>
<td>22.5</td>
</tr>
<tr>
<td>Iller</td>
<td>50-140</td>
<td>0.0135</td>
<td>51.3</td>
</tr>
<tr>
<td>Kinu</td>
<td>20-70</td>
<td>0.0407</td>
<td>17.0</td>
</tr>
<tr>
<td>Watrase</td>
<td>30-80</td>
<td>0.067</td>
<td>10.3</td>
</tr>
<tr>
<td>Tenryu</td>
<td>15-50</td>
<td>0.0856</td>
<td>8.1</td>
</tr>
<tr>
<td>Kiso</td>
<td>35-70</td>
<td>0.0520</td>
<td>13.3</td>
</tr>
<tr>
<td>Nagara</td>
<td>25-40</td>
<td>0.0778</td>
<td>9.7</td>
</tr>
<tr>
<td>Sho</td>
<td>20-50</td>
<td>0.0464</td>
<td>14.9</td>
</tr>
<tr>
<td>Abe</td>
<td>15-90</td>
<td>0.1150</td>
<td>6.0</td>
</tr>
<tr>
<td>Makita</td>
<td>100-200</td>
<td>0.180</td>
<td>3.9</td>
</tr>
<tr>
<td>Knik</td>
<td>30-400</td>
<td>0.130</td>
<td>5.3</td>
</tr>
<tr>
<td>Vedder</td>
<td></td>
<td>0.193</td>
<td>3.6</td>
</tr>
</tbody>
</table>

\[
D = D_0 e^{-adx}
\]

All data except Knik, Peace, and Vedder Rivers are reported by Simons (1971).

Knik River data reported by Bradley et al (1972)

Peace River data reported by Church and Kellerhals, 1978
<table>
<thead>
<tr>
<th>Period of Recorded Record</th>
<th>Maximum Load (tons/day)</th>
<th>Maximum Concentration (mg/l)</th>
<th>Minimum Concentration (mg/l)</th>
<th>Average Annual Load (tons/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1965-77</td>
<td>4,000</td>
<td>202,000</td>
<td>0</td>
<td>146,000</td>
</tr>
</tbody>
</table>
### TABLE 22

**SUMMARY OF BEDLOAD MEASUREMENTS**

**COLLECTED ON VEDDER RIVER**

<table>
<thead>
<tr>
<th>Date</th>
<th>Discharge</th>
<th>Mean Velocity (ft/sec)</th>
<th>Mean Depth (ft)</th>
<th>Sampler Type</th>
<th>Bedload (Tons/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>June 1, 1971</td>
<td>5,400</td>
<td>5.42</td>
<td>3.91</td>
<td>basket</td>
<td>0</td>
</tr>
<tr>
<td>June 2, 1971</td>
<td>5,690</td>
<td>5.11*</td>
<td>3.91</td>
<td>basket</td>
<td>0</td>
</tr>
<tr>
<td>June 7, 1971</td>
<td>6,710</td>
<td>5.89*</td>
<td>4.17</td>
<td>VUV</td>
<td>41.4</td>
</tr>
<tr>
<td>June 22, 1971 7,640</td>
<td>6.01*</td>
<td>4.40</td>
<td>VUV</td>
<td>54.3</td>
<td></td>
</tr>
<tr>
<td>June 23, 1971</td>
<td>8,040</td>
<td>6.20*</td>
<td>4.49</td>
<td>basket</td>
<td>9.3</td>
</tr>
<tr>
<td>June 24, 1971</td>
<td>8,500</td>
<td>5.73*</td>
<td>4.24</td>
<td>VUV</td>
<td>29.1</td>
</tr>
<tr>
<td>June 25, 1971</td>
<td>6,970</td>
<td>5.73*</td>
<td>4.24</td>
<td>VUV</td>
<td>43.5</td>
</tr>
<tr>
<td>May 26, 1972 5,210</td>
<td>5.34</td>
<td>VUV</td>
<td>9.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 29, 1972 11,220</td>
<td>7.85</td>
<td>basket</td>
<td>1062</td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 30, 1972 12,480</td>
<td>7.52</td>
<td>basket</td>
<td>453</td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 31, 1972 11,640</td>
<td>7.23</td>
<td>basket</td>
<td>27.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 31, 1972 11,300</td>
<td>7.02</td>
<td>VUV</td>
<td>33.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 1, 1972 9,630</td>
<td>6.46</td>
<td>basket</td>
<td>2.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 1, 1972 9,630</td>
<td>6.46</td>
<td>VUV</td>
<td>18.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 2, 1972 8,250</td>
<td>5.98</td>
<td>VUV</td>
<td>22.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 5, 1972 8,330</td>
<td>6.04</td>
<td>VUV</td>
<td>24.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 6, 1972 9,400</td>
<td>6.48</td>
<td>VUV</td>
<td>VUV</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 6, 1972 9,350</td>
<td>6.45</td>
<td>VUV</td>
<td>15.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 7, 1972 10,800</td>
<td>6.92</td>
<td>VUV</td>
<td>98.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 8, 1972 11,020</td>
<td>6.89</td>
<td>basket</td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 9, 1972 13,120</td>
<td>7.72</td>
<td>basket</td>
<td>287</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 13, 1972 7,400</td>
<td>5.29</td>
<td>VUV</td>
<td>22.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 15, 1972 6,200</td>
<td>4.92</td>
<td>VUV</td>
<td>29.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>June 15, 1972 6,200</td>
<td>4.92</td>
<td>VUV</td>
<td>6.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td>Discharge</td>
<td>Mean Velocity (ft/sec)</td>
<td>Mean Depth (ft)</td>
<td>Sampler Type</td>
<td>Bedload (Tons/day)</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------</td>
<td>------------------------</td>
<td>----------------</td>
<td>--------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>June 23, 1972</td>
<td>5,860</td>
<td>4.92</td>
<td>4.3</td>
<td>VUV</td>
<td>16.5</td>
</tr>
<tr>
<td>June 30, 1972</td>
<td>7,360</td>
<td>5.53</td>
<td>4.3</td>
<td>VUV</td>
<td>6.3</td>
</tr>
<tr>
<td>May 17, 1973</td>
<td>5,400</td>
<td>5.09</td>
<td>3.9</td>
<td>basket</td>
<td>0</td>
</tr>
<tr>
<td>May 17, 1973</td>
<td>5,730</td>
<td>5.26</td>
<td>3.9</td>
<td>VUV</td>
<td>20.5</td>
</tr>
<tr>
<td>May 17, 1973</td>
<td>5,900</td>
<td>5.36</td>
<td>3.9</td>
<td>basket</td>
<td>3.1</td>
</tr>
<tr>
<td>May 18, 1973</td>
<td>6,080</td>
<td>5.43</td>
<td>3.9</td>
<td>basket</td>
<td>1.1</td>
</tr>
<tr>
<td>May 18, 1973</td>
<td>6,010</td>
<td>5.41</td>
<td>3.9</td>
<td>VUV</td>
<td>31.5</td>
</tr>
<tr>
<td>May 23, 1973</td>
<td>3,370</td>
<td>3.95</td>
<td>3.1</td>
<td>VUV</td>
<td>8.3</td>
</tr>
<tr>
<td>May 24, 1973</td>
<td>5,440</td>
<td>5.13</td>
<td>3.8</td>
<td>VUV</td>
<td>20.8</td>
</tr>
<tr>
<td>May 24, 1973</td>
<td>6,500</td>
<td>5.60</td>
<td>4.0</td>
<td>basket</td>
<td>0</td>
</tr>
<tr>
<td>May 25, 1973</td>
<td>4,900</td>
<td>4.85</td>
<td>3.6</td>
<td>VUV</td>
<td>16.2</td>
</tr>
<tr>
<td>June 22, 1973</td>
<td>5,000</td>
<td>4.90</td>
<td>3.7</td>
<td>VUV</td>
<td>17.0</td>
</tr>
<tr>
<td>June 3, 1974</td>
<td>7,560</td>
<td></td>
<td></td>
<td>basket</td>
<td>43.5</td>
</tr>
<tr>
<td>June 4, 1974</td>
<td>7,700</td>
<td></td>
<td></td>
<td>basket</td>
<td>53.9</td>
</tr>
<tr>
<td>June 12, 1974</td>
<td>9,080</td>
<td></td>
<td></td>
<td>basket</td>
<td>280</td>
</tr>
<tr>
<td>June 13, 1974</td>
<td>10,600</td>
<td></td>
<td></td>
<td>basket</td>
<td>790.6</td>
</tr>
<tr>
<td>June 14, 1974</td>
<td>11,700</td>
<td></td>
<td></td>
<td>basket</td>
<td>286.9</td>
</tr>
<tr>
<td>June 15, 1974</td>
<td>12,600</td>
<td></td>
<td></td>
<td>basket</td>
<td>1292.6</td>
</tr>
</tbody>
</table>

Note: All bedload data obtained from Water Survey of Canada, Sediment Survey Section

*estimated by interpolation of Stage-discharge measurements
<table>
<thead>
<tr>
<th>Reach</th>
<th>Length (mi)</th>
<th>Period</th>
<th>Total Deposition (yd³)</th>
<th>Average Annual Deposition (yd³/yr)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>railway bridge - Peache Road</td>
<td>2.6</td>
<td>1957 - July 1975</td>
<td>192,000</td>
<td>10,100</td>
<td>I.P.S.C. (1)</td>
</tr>
<tr>
<td>Peache Road - Vedder Crossing</td>
<td>1.1</td>
<td>1957 - July 1975</td>
<td>130,000</td>
<td>6,800</td>
<td>I.P.S.C. (1)</td>
</tr>
<tr>
<td>railway bridge - Ford Road</td>
<td>1.6</td>
<td>1972 - July 1975</td>
<td>135,000</td>
<td>33,800</td>
<td>B.C.W.R.S.</td>
</tr>
<tr>
<td>Ford Road - Vedder Crossing</td>
<td>2.1</td>
<td>1972 - July 1975</td>
<td>100,000</td>
<td>25,000</td>
<td>B.C.W.R.S.</td>
</tr>
<tr>
<td>railway bridge - Peache Road</td>
<td>2.6</td>
<td>1958 - 1963</td>
<td>17,000</td>
<td>3,400</td>
<td>Marr (1964)</td>
</tr>
<tr>
<td>Peache Road - Vedder Crossing</td>
<td>1.1</td>
<td>1958 - 1963</td>
<td>-</td>
<td>-</td>
<td>Marr (1964)</td>
</tr>
<tr>
<td>railway bridge - Peache Road</td>
<td>2.6</td>
<td>1971 - 1975</td>
<td>80,000</td>
<td>16,000</td>
<td>W.S.C. (2)</td>
</tr>
<tr>
<td>Peache Road - Vedder Crossing</td>
<td>1.1</td>
<td>1971 - 1975</td>
<td>84,000</td>
<td>16,800</td>
<td>W.S.C. (2)</td>
</tr>
</tbody>
</table>

Notes:
(2) Based on net channel area changes at 6 sections reported by W.S.C. Estimates are very approximate and for comparison purposes only.
<table>
<thead>
<tr>
<th>Probability of Exceedance in Period Indicated</th>
<th>Peak Daily Discharge (cfs)</th>
<th>Total Bedload Transport (tons)</th>
<th>Total Deposition (tons)</th>
<th>Total Deposition (cu. yds.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>8,600</td>
<td>49,000</td>
<td>300</td>
<td>48,700</td>
</tr>
<tr>
<td>0.10</td>
<td>12,000</td>
<td>158,000</td>
<td>3,000</td>
<td>155,000</td>
</tr>
<tr>
<td>0.05</td>
<td>13,000</td>
<td>209,000</td>
<td>5,000</td>
<td>204,000</td>
</tr>
<tr>
<td>0.02</td>
<td>14,200</td>
<td>290,000</td>
<td>8,000</td>
<td>282,000</td>
</tr>
</tbody>
</table>

Spring Snowmelt Floods

<table>
<thead>
<tr>
<th>Probability of Exceedance in Period Indicated</th>
<th>Peak Daily Discharge (cfs)</th>
<th>Total Bedload Transport (tons)</th>
<th>Total Deposition (tons)</th>
<th>Total Deposition (cu. yds.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.50</td>
<td>9,800</td>
<td>30,000</td>
<td>400</td>
<td>29,600</td>
</tr>
<tr>
<td>0.10</td>
<td>18,700</td>
<td>254,000</td>
<td>12,000</td>
<td>242,000</td>
</tr>
<tr>
<td>0.05</td>
<td>22,500</td>
<td>451,000</td>
<td>29,000</td>
<td>422,000</td>
</tr>
<tr>
<td>0.02</td>
<td>27,500</td>
<td>855,000</td>
<td>61,000</td>
<td>794,000</td>
</tr>
</tbody>
</table>

Winter Rainstorm Floods

Note: Spring snowmelt period: April 1 - July 31
Winter rainstorm period: Sept. 1 - Mar. 31

Flood hydrographs used to compute bedload transport shown in Figure
<table>
<thead>
<tr>
<th>River</th>
<th>Design Discharge (cfs)</th>
<th>Dso Size (mm)</th>
<th>Channel Width (ft)</th>
<th>Channel Pattern</th>
<th>Problems</th>
<th>Control Scheme</th>
</tr>
</thead>
<tbody>
<tr>
<td>Snake River Wyoming</td>
<td>45,000</td>
<td>0.0036</td>
<td>20</td>
<td>braided</td>
<td>aggradation, bank erosion</td>
<td>levees</td>
</tr>
<tr>
<td>Rhine River Switzerland</td>
<td>106,000</td>
<td>≈ 0.01</td>
<td>?</td>
<td>single channel</td>
<td>aggradation</td>
<td>upstream dredging, constriction</td>
</tr>
<tr>
<td>Waimakariri River New Zealand</td>
<td>167,000</td>
<td>0.0024</td>
<td>9-38</td>
<td>split/braided</td>
<td>aggradation</td>
<td>river training, dredging, stop banks</td>
</tr>
<tr>
<td>Equation</td>
<td>Description</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$H_c/D_{50} = 0.253 \overline{Q}^{0.415}$</td>
<td>Centreline channel depth</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$S = 0.223 \overline{Q}^{-0.410}$</td>
<td>Slope</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$B/D_{50} = 4.4 \overline{Q}^{0.5}$</td>
<td>Top width</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Where $\overline{Q} = Q/ (1.65g D_{50} D_{50}^2)$

$H_c$ = Centreline channel depth
$S$ = Slope
$B$ = Top width
$Q$ = Dominant discharge
FIGURES
FIGURE 1. Site Location
FIGURE 2. Study Area Below Vedder Crossing
FIGURE 4. Channel Pattern of the Vedder River 1930-1976
FIGURE 4. Channel Pattern of the Vedder River 1930-1976
March 19 1971  Scale: 1 lin=1325 ft

June 19 1976  Scale: 1 lin=1680 ft

FIGURE 4. Channel Pattern of the Vedder River 1930-1976
FIGURE 5. Flood Paths Along Vedder River in 1951, 1975
FIGURE 6. Chilliwack River Basin
FIGURE 7. Monthly Met Data for the Chilliwack Basin
At Vedder Crossing
Vedder River near Yarrow
Below Slesse Creek
Above Slesse Creek
Chilliwack Lake Outlet
Slesse Creek

Legend:
- discharge
- stage only

FIGURE 8. Summary of Stream Gauging Operations Along Chilliwack River
FIGURE 9. Monthly Flows Along Chilliwack River
FIGURE 10. Historical Occurrence of Past Floods
Winter Rainstorm Flood December 1975

Max Inst = 27,800 cfs
Max Daily = 18,700

Summer Snowmelt Flood June 1972

Max Inst = 15,100 cfs
Max Daily = 12,700 cfs

Figure 11. Comparison of Two Rainstorm and Snowmelt Floods
FIGURE 12. Correlation Between Nooksack River and Chilliwack River Floods
FIGURE 13. Predicted and Recorded 1975 Flows
FIGURE 14. Predicted Flood Hydrographs for 1932 and 1951
FIGURE 15. Time Series of Annual Floods at Vedder Crossing

Note: All flows are mean daily discharges recorded at Vedder Crossing

average of 1911-1975

ungauged
FIGURE 16. Flood Frequency Analysis at Vedder Crossing

Note: relations derived from mean daily flows

Mean Daily Discharge (cfs)

Probability of Exceedance (%)
FIGURE 17. Stream Profile of Chilliwack River

Legend:
- Terrace level
- Bedrock
- Tributary creek
- Chilliwack Lake
- Vedder Crossing
- Liurnchen Cr.
- Ryder Cr.
- Tamahi Cr.
- Slesse Cr.
- Chipmunk Cr.
- Nesakwatch Cr.
- Centre Cr.
- Post Cr.
Figure 18. Fan Profiles Measured from Vedder Crossing
FIGURE 19. Channel Cross-sections Above Vedder Crossing

1.2 mi above Vedder Crossing

Distance (ft)

Elevation (ft)

Note: all data from Marr (1964)

2.5 mi above Vedder Crossing

Distance (ft)

Elevation (ft)

3.6 mi above Vedder Crossing

Distance (ft)

Elevation (ft)
Figure 20. Hydraulic Measurements at Water Survey of Canada Gauges above Vedder Crossing
Knick point

Chilliwack Lake to Chipmunk Cr.

FIGURE 21.1 Channel Reach Descriptions
Chipmunk Cr. to Slesse Cr.

FIGURE 21.2 Channel Reach Descriptions
FIGURE 21.3 Channel Reach Descriptions
FIGURE 21.4 Channel Reach Descriptions

Ryder Cr.

Valley Walls

Scale
1"=4167 ft

Tamahi Cr.

Boulder deposits in below Tamahi Cr.

Tamahi Cr. to Ryder Cr.
FIGURE 21.5 Channel Reach Descriptions

View upstream from near Vedder Crossing
July 15, 1940  Scale lin=1723 ft (Approx)

Vedder Crossing

May 18, 1968  Scale lin=1430 ft (Approx)
Discharge=4230 cfs

June 19, 1976  Scale lin=1617 ft (Approx)
Discharge=9490 cfs

Figure 22  Comparative Airphotos - Vedder Crossing to Liumchen Creek
FIGURE 23.1. Channel Pattern Upstream of Vedder Crossing

September 1952
FIGURE 23.2. Channel Pattern Upstream of Vedder Crossing
FIGURE 23.3. Channel Pattern Upstream of Vedder Crossing
FIGURE 24.1. Channel Changes Near Vedder Crossing 1940-1958
FIGURE 24.2. Channel Changes Near Vedder Crossing 1958-1971
FIGURE 24.3. Channel Changes Near Vedder Crossing 1971-1976
FIGURE 25. Conceptual Bedload Movement Above Vedder Crossing
FIGURE 26. Approximate Location of Vedder River Cross-sections
FIGURE 27.1 Cross-sections Along Vedder River
FIGURE 27.2 Cross-sections Along Vedder River

- Mile 4.23 constriction above rail bridge
  - June 1975
  - Jan. 1976

- Mile 4.43 Hopedale Rd

- Mile 4.96 (near Browne Rd)

- Mile 4.76 (Between Hopedale-Browne Rd)
Mile 7.40 (near Vedder Crossing)

Mile 7.58 (near Vedder Crossing)

Mile 7.70 (at Vedder Crossing)

Note: all sections surveyed by B.C.W.R.S. 1976

FIGURE 27.3 Cross-sections Along Vedder River
FIGURE 28. Sections Showing Floodplain and Channel Topography
Figure 29. Effect of Fraser and Vedder River Flows on the Stage in the Vedder Canal
FIGURE 30. Hydraulic Geometry at Yarrow and Vedder Crossing
Resistance Equations

1. Manning-Strickler $\frac{V}{V_*} = 8.4 \left(\frac{d}{D_{90}}\right)^{1/6}$

2. Keulegan $\frac{V}{V_*} = 6.25 + 5.75 \log\left(\frac{d}{D_{90}}\right)$

3. Liméritos $\frac{V}{V_*} = 3.28 + 5.75 \log\left(\frac{d}{D_{90}}\right)$

FIGURE 31. Comparison of Resistance Formulas Using Yarrow Data
FIGURE 32. Downstream Changes in Bedmaterial Size Along Vedder River

Median Bedmaterial Size (mm)

Vedder Canal

note: all material less than 8mm removed from samples

Railway Bridge

Vedder Crossing
Figure 33. Bedload Grain Size Measured near Yarrow

Note: All data derived from W.S.C. measurements 1971-75
FIGURE 34. Bedload Size Distribution at Different Flow Conditions
measurements collected by W.S.C 1971-1974

predicted by Einstein formula

FIGURE 35. Bedload Transport at Yarrow
FIGURE 36. Comparison of Measured and Computed Bedload at Yarrow
FIGURE 37. Bedload Estimates at Vedder Crossing
Based on analysis by Gessler (1971)

FIGURE 38. Theoretical Aggradation Profiles
FIGURE 39. Comparison of Mean Bedlevels Along Vedder River
FIGURE 40. Effect of Channelization on Channel Geometry
FIGURE 41. Specific Gauge Record Near Yarrow

Note: gauge height recorded near Yarrow plotted for constant discharge values.

Q=7500 cfs

Q=5000 cfs

Period of dredging

Phase II dredging
FIGURE 42. Specific Gauge Record at Vedder Crossing
FIGURE 43. Variation in Deposition Along Vedder River Due to 1975 Flood
Figure 44. Derived Hydrographs for Snowmelt and Rainstorm Floods
FIGURE 45. Wide Vedder River Flood Control Scheme

- Strengthen and upgrade existing dike
- Remove existing bank protection
- Channel allowed to shift within these limits
- 1958 channel

1000 ft scale
Set-Back Dike

revetment to contain moderate floods

Excavate annually

old side channels

railway bridge

provide flood relief here

Set-Back Dike

Browne Rd

Ford Rd

FIGURE 46 Possible Set-Back Dike and River Training Alignment
APPENDIX 1
APPENDIX 1

PERFORMANCE OF BEDLOAD FORMULA ON GRAVEL RIVERS

The performance of available bedload formulae were tested using field measurements from five gravel rivers. Several past studies have evaluated the features of transport formula using flume data and field measurements from sand bed streams (ASCE Sedimentation Manual, 1975; Raudkivi, 1967; White, Milli and Crabbe, 1975). However except for reports by Church (1976) and Hollingshead (1968, 1971) very little effort has been made to test the usefulness of these formula on gravel rivers. Rather than trying to exhaustively test every formula available, only methods that have proven reasonably reliable in the past or have been developed from conditions representative of gravel rivers were used. In this study three bedload equations were selected for testing:

Meyer Peter & Muller (1948)
Einstein (1950)
Ackers-White (1972)

Field measurements of bedload transport in gravel rivers are still relatively scarce due to the technical difficulties involved and the high cost of sampling. Therefore, conclusions drawn from this study may need to be modified as more data and better sampling techniques become available. Although considerably more data may
exist, only data from the following gravel rivers were used for evaluating the bedload formulae:

Elbow River near Bragg Cr. Alberta (Hollingshead 1968, 1971)
N. Saskatchewan River at Nordegg, Alberta (Samedi, 1971)
Snake River near Anatone Washington (Emmett 1976)
Clearwater River near Spalding Idaho (Emmett 1976)

The methods used to collect bedload, hydraulic and bed material data are summarized in Table A1. Since the bedload measuring techniques as well as the type of input data in the transport equations varied from site to site a brief description of the methods used at each river are included.

**Bedload Measurements:**

Bedload measurements on the Elbow River, N. Saskatchewan River, and Vedder River were made using both 1/4" and 1/2" mesh basket samplers. From the limited data available an overall trap efficiency of 0.31 was applied to the measured discharge rates. This factor was estimated by Hollingshead (1971) after comparing the bedload deposited in an excavated pit with material caught in a sampler. Trap efficiency will vary with discharge and sediment size, however no information is available yet to account for these effects. On the Vedder River, a half size VUV sampler
was used near threshold levels along with the basket sampler. The results showed that the basket samplers were not suitable for measuring bedload at low flows due to the loss of fines through the coarse mesh.

Bedload samplers used on the Snake and Clearwater Rivers were of the Helley-Smith type and are described further by Emmett (1976). The Helley-Smith samples have not been fully calibrated however, preliminary field testing by Emmett (1975) indicated the efficiency may approach 100%.

The practical problems involved in measuring bedload are outlined clearly by Samedi (1971) and Hollingshead (1968). Besides the purely technical problems involved, the difficulty in assessing a sampler efficiency factor and the inherent unsteady nature of sediment transport introduces considerable uncertainty into any bedload measurement. Therefore, in this study the difference between measured and predicted bedload rates should not be considered as the error associated with the formula. In fact, probably both the measurements and the predictions vary considerably from the "true" transport rate - if such a value has any real meaning.

**Hydraulic Data:**

The quantity and type of hydraulic data reported from each site varied considerably. Data collected from
the Elbow River and North Saskatchewan River were reduced to mean hydraulic geometry relations from a composite cross-section representative of the test reach. For example, at Elbow River twenty cross-sections were surveyed within a length of 3400 feet at several stage intervals to obtain average channel properties.

In contrast Emmett (1976) reported mean hydraulic conditions at one section recorded during sampling on the Snake and Clearwater River. In this case, bedload results were computed using the actual recorded data measured at a single station. The possibility that a single cross-section may not be representative of the river reach makes this type of data less desirable for use in predicting bedload.

**Bed Material Data**

The final input required for all bedload calculations is the bed materials grain size distribution. The choice of sampling procedure and sample location may drastically alter the sediment transport rate predicted from formula. This is especially true for gravel rivers where large spatial variations in grain size, channel paving and imbrication often occur. Unfortunately, except for data from Vedder River and Elbow River only a single representative bed material size distribution curve were available. At Elbow River 10 pit samples excavated
at least to the largest exposed particle were used to establish the bed material available for transport (Hollingshead 1971). At Vedder River 16 pit samples were chosen in the vicinity of the test reach to define the bed material. Additional surface samples were taken by using a photographic grid method (Kellerhals & Bray 1971) to establish the bed roughness size.

Prediction Results

Einstein Equation (1950)

The original 1950 Einstein procedure was used in this study with two minor variations. In the original method the mean velocity, hydraulic radius and discharge were computed directly using a resistance equation proposed by Einstein and Barbarossa (1952). The reduction in bed shear due to form losses was considered by dividing the hydraulic radius \( r \) into two components:

\[ r' \text{: hydraulic radius due to bed roughness} \]
\[ r'' \text{: hydraulic radius due to form losses} \]

so that the shear stress contributing to bedload movement is:

\[ \tau' = \gamma r'S_f \].

When Einstein's predicted hydraulic parameters were compared with actual field measurements the results showed considerable discrepancies on both the Vedder and Elbow Rivers. In both cases Einstein's method overestimated
the mean depth and underestimated the mean velocity. Since
the flow resistance was overestimated the predicted bed-
load rates were greatly reduced. In this study, known
values of mean velocity, hydraulic radius and wetted peri-
meter were used as input in the bedload equation to elimi-
nate this source of error. Also, the hydraulic radius
due to grain roughness \( r' \) was computed directly from
the Manning Strichler equation.

\[
\frac{V}{V_s} = 8.40 \left( \frac{r'}{K_s} \right)^{1/6}
\]

instead of from the "bar resistance graph" proposed by
Einstein. The Manning-Strichler equation was developed
from lab experiments by Nikuradse and from field experi-
ments in gravel streams by Strichler. It is applicable
only for the case of fully rough turbulent flow - a con-
dition normally encountered in gravel rivers.

Results from the Einstein equation appear to be quite
good considering the large scatter often displayed by the
field measurements. Predictions on the Elbow River,
Vedder River and Snake River tend to agree with the general
trend of the measurements throughout the range of flows en-
countered during sampling. On the N. Saskatchewan River
threshold conditions were estimated closely however trans-
port rates were underestimated at higher flows. The poorest
performance of the Einstein equation was on the Clearwater
River where threshold conditions were grossly overestimated.
For example, at a discharge of 60,000 cfs measurements indicated a transport rate of approximately 250 tons/day while the estimated transport was only 25 tons/day. With increased discharge the measured and predicted transport rates tend to converge however agreement was still poor.

The ratio of predicted to measured transport rates were computed using all data except for measurements on the Clearwater River. Only about one third of the measurements fell within one half to two times the predicted transport rates. This ratio represents a very pessimistic estimate of the Einstein equation's performance because the field measurements displayed a large amount of scatter. It is quite likely that a considerable portion of this scatter can be attributed to measurement errors and inaccuracies and to fluctuations in bedload transport associated with turbulence. A better estimation of the equations' reliability was made by plotting the measured transport rates against discharge on log-log paper. Predicted bedload rates were then compared with "best fit" straight line relations derived from the measurements. In this case, using the Elbow River data the discrepancy ratio \( \frac{\text{Predicted Transport}}{\text{Measured Transport}} \) varied from 1.0 to 1.25 while the Snake River results showed a variation between 1.0 and 2.5. Poorest correlation occurred on the Clearwater River where
the discrepancy ratio varied from 0.01 to 0.45.

_Meyer Peter & Muller_

This formula was developed from flume experiments using sediment as coarse as 28.65 mm. The final formula suggested by Meyer-Peter & Muller (1948) was:

\[ \frac{k_s^{3/2}}{r} \tau_\text{critical} - 0.047(\gamma_s - \gamma)D_{50} = 0.25(p^{1/8})g_s^{2/3} \]

The term \( \tau_\text{critical} \) represents the shear stress on the bed while \( 0.047(\gamma_s - \gamma)D_{50} \) is the critical stress required to initiate particle movement. Unlike earlier equations based on excess shear such as the duBoys equation, the Meyer-Peter & Muller formula accounts for the reduction in bed shear due to form losses by using a resistance coefficient \( k_{gb}^{3/2} \). This term is the ratio of total resistance to grain resistance and is calculated using Strichler's formula and Manning's coefficient.

\[ k_b = 1.49/n \]
\[ k_g = 48 \frac{D_{90}^{1/6}}{D_90} \]

In general the Meyer-Peter & Muller formula tended to significantly overestimate threshold conditions, resulting in very poor agreement with measurements at lower flows. On the Vedder River threshold conditions were estimated to occur at 9200 cfs while measurements showed significant
sediment transport commenced near 600 cfs. Similarly, one the Snake River, threshold was estimated to be near 76,000 cfs while measurements indicated a value of 35,000 cfs was more likely. The Meyer-Peter & Muller formula calculates threshold conditions from Shields formula

$$\frac{T}{(\gamma_s - \gamma)D_{50}} = 0.047.$$ Reducing this value from 0.047 to 0.03 significantly improved the predictions near threshold however insufficient data is available to test this further.

At higher flows the formula may tend to overestimate bedload. This was especially true on the Elbow and Snake River where the estimated bedload exceeded the measured values by over five times.

**Ackers-White**

According to White et al (1975) the Ackers-White formula is one of the most reliable bedload equations available at present. For application to gravel rivers the formula simplifies to:

$$G_s = 0.025 \frac{D}{s} Q \frac{\gamma (F_{gr} - 1.0)^{1.5}}{1.0}$$

where $F_{gr} = \frac{V}{\sqrt{g} D_x (s-1)} \frac{\sqrt{32 \log (\frac{10d}{D_x})}}{D_x}$.

Originally the authors suggested using $D_{35}$ as the representative grain size in the transport equation, however better agreement with measurements occurred when the $D_{50}$
size was used. Unfortunately results from the Ackers-White equation were still quite disappointing. Although threshold conditions were estimated well the formula generally overestimated bedload rates at higher flows. This was especially noticeable on the Elbow River, Vedder River and possibly the N. Saskatchewan River. Comparing the predicted bedload rates with best fit measured relations indicated the discrepancy ratio varied from 32 to 56 using Vedder River data to between 1.9 and 3.6 on the Snake River. The reason for such large differences may be due to the formula's failure to account for form losses adequately. Although past studies have found the Ackers-White formula reliable it does not appear to be applicable for gravel rivers when mean hydraulic geometry are used as the input data.

Limitations of Bedload Theories in Gravel Rivers

Although bedload formula often have been derived from different theories, many formula share very similar deficiencies. For example, all formulae except the Einstein equation use a representative grain size such as $D_{35}$ or $D_{50}$ to describe the bed material composition. In the case of sand bed streams such an assumption may be reasonable, however it does not seem valid in the case of gravel rivers. Besides the much greater size range displayed in gravel rivers, variations in the shape of the grain size distribution
curve may also be pronounced.

Another common feature of many bedload equations including the Meyer-Peter & Muller and Ackers-White method is that they can be reduced to the form:

\[ G_s = k_1 (X_1 - X_2)^N \]

This type of relation is very susceptible to errors when \( X_1 \to X_2 \); which normally corresponds to threshold conditions. For example, in the Meyer Peter & Muller formula bedload transport is proportional to:

\[ k_1 \left( \frac{\tau_{rb}}{\tau_g} \right)^{3/2} - 0.047(\gamma_s - \gamma)D_{50} \]

The factor \( 0.047(\gamma_s - \gamma)D_{50} \) was taken from Shield's relation \( \left( \frac{\tau}{(\gamma_s - \gamma)D} \right) = 0.047 \) to describe the beginning of particle movement.

Considering the complicating factors of bed imbrication, and particle shape it is likely the value of 0.047 may vary considerably in actual rivers. However, any small error in this factor will be multiplied considerably in the equation.

One major factor not considered by present day theory is the variation in bed composition over a flood. To date all theories assume the initial bed material size remains constant. In gravel rivers this is not likely to occur. At low flows the coarse fraction of the bed does not move. However, during a flood the coarser material
will eventually begin to move, the armoured layer will break up and finer sediment will be entrained. This could cause an increase in transport as the courser gravel becomes buried by the finer material.

Summary and Conclusions:
1. The Einstein equation was found to be the most reliable formula available for predicting bedload on the five gravel rivers examined. When individual measurements were compared with predicted values only one third of the measurements fell with one half to two times the predicted values. When "best fit" straight line relations were established from the measurements considerably better agreement was found.
2. The Ackers-White appears to be reliable near threshold conditions but overestimated the bedload rate at high flows on all five rivers examined.
3. The Meyer-Peter & Muller formula did not perform well near threshold conditions in this test. However, at higher flows it seemed to converge towards the measured results.
4. The performance of any equation is related to the quality of the input data. At present, it appears better to use mean hydraulic geometry which is representative of the river reach rather than data from a single cross section.
### Table A.1
**Summary Description of Measurement Sites**

<table>
<thead>
<tr>
<th>River</th>
<th>Data Source</th>
<th>Mean Annual Flood (cfs)</th>
<th>Flows During Sampling (cfs)</th>
<th>Channel Slope</th>
<th>Bed Material Size ($D_{50}$ mm)</th>
<th>Number of Measurements per Sample/Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elbow River</td>
<td>Hollingshead (1968, 1971)</td>
<td>2000</td>
<td>1370 - 3850</td>
<td>0.0075</td>
<td>25.0</td>
<td>24 basket</td>
</tr>
<tr>
<td>N. Saskatchewan River</td>
<td>Samide (1971)</td>
<td>15000</td>
<td>6169 - 10358</td>
<td>0.00158</td>
<td>17.0</td>
<td>15 basket</td>
</tr>
<tr>
<td>Vedder River</td>
<td>WSC (1971, 1972, 1973)</td>
<td>10000</td>
<td>5400 - 12300</td>
<td>0.00195</td>
<td>21.5</td>
<td>35 basket and $\frac{1}{2}$ size VUV</td>
</tr>
<tr>
<td>Snake River</td>
<td>Emmett (1976)</td>
<td>-</td>
<td>32000 - 133000</td>
<td>0.00066-0.0011</td>
<td>32.0</td>
<td>28 Helley-Smith</td>
</tr>
<tr>
<td>Clearwater</td>
<td>Emmett (1976)</td>
<td>-</td>
<td>21984 - 123930</td>
<td>0.00068-0.00011</td>
<td>32.0</td>
<td>29 Helley-Smith</td>
</tr>
</tbody>
</table>
Vedder River near Yarrow
data provided by W.S.C.

FIGURE A.1 Comparison of Bedload Formulas on Vedder River

Legend:
- Einstein (1950)
- Meyer Peter&Muller
- Ackers&White
- Regression from measured data
Elbow River near Bragg Cr.
data from Hollingshead (1968)

FIGURE A.2 Comparison of Bedload Formulas on Elbow River
North Saskatchewan River at Nordegg
data from Samedi (1971)

FIGURE A.3 Comparison of Bedload Formulas on N. Saskatchewan River
Clearwater River at Spalding Washington
data from Emmett (1976)

FIGURE A.4 Comparison of Bedload Formulas on Clearwater River
Snake River near Anatone Washington
data from Emmett (1976)

Legend:
- Einstein (1950)
- Meyer-Peter & Muller
- Ackers & White
- Regression from measured data

FIGURE A.5 Comparison of Bedload Formulas on Snake River