

box 679A

Report

**DEWDNEY-ALOUETTE REGIONAL DISTRICT
MISSION, B.C.
HATZIC PRAIRIE DRAINAGE STUDY
VOLUME 1**

August 1992



ASSOCIATED
ENGINEERING



August 19, 1992
File: VP97-5-4-130

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32836 Fletcher Avenue
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**Re: HATZIC PRAIRIE DRAINAGE STUDY
FINAL REPORT**

Associated
Engineering
(B.C.) Ltd.

Dear Mr. Gairns:

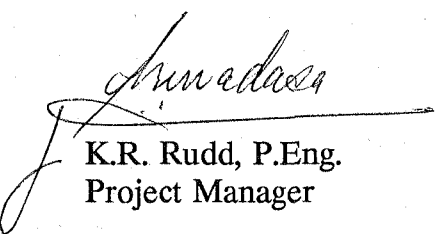
Please find attached twenty-five (25) copies of the Final Report for this project.

This report includes review comments on the Draft Report by the Advisory Committee which were discussed on June 9, 1992 meeting. It also incorporates an Executive Summary which summarizes proposed mitigation and improvement works and estimated costs.

We trust this report will provide an opportunity for the Regional District and other local institutions to identify a package of improvements for the Hatzic Prairie area and seek funding from senior Governments and funding agencies.

Respectfully submitted,

ASSOCIATED ENGINEERING (B.C.) LTD.



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ACKNOWLEDGEMENTS

This study of drainage and sedimentation problems which exist in the Hatzic Prairie catchment area, was commissioned by the Dewdney-Alouette Regional District (DARD) on June 1, 1991. The study and preparation of the report have required nine months to complete. Throughout this period meetings have been held on a regular basis with an advisory committee which consisted of representatives from a number of interested groups in the project area. The local knowledge, concern and constructive input provided by this committee and the staff of the Dewdney-Alouette Regional District have contributed significantly to the completion of this report.

We wish to thank the following committee members for their contribution:

Alderman D. Adair, District of Mission
Director B. Appleby, Electoral Area B, DARD
Director A. Neufeld, Electoral Area E, DARD
D. Bogstie, Hatzic Prairie-McConnell Creek Ratepayers Association
J. Connor, Sheltered Cove Improvement District
P. England, Building Inspector, DARD
J. Kinneard, Sheltered Cove Improvement District
R. Kinneard, Dewdney-Area Improvement District
A. Oliver, Dewdney Area Improvement District
D. Riecken, District of Mission
D. Russbueldt, Hatzic Island Ratepayers Association
K. Wilson, Water Management Branch, BC Environment

PROJECT TEAM

This report has been prepared on behalf of the Dewdney Alouette Regional District by Associated Engineering (B.C.) Ltd. assisted by a team of specialist subconsultants. This consultant team was assembled to address the tasks identified within the study Terms of Reference which required expertise from several engineering disciplines.

The consultant team and their respective areas of involvement were:

- **Associated Engineering (B.C.) Ltd.**

Prime consultant responsible for project management, design of mitigation works and preparation of cost estimates.

- **Northwest Hydraulic Consultants Ltd.**

Hydrology, hydraulics, sedimentation and forestry.

- **Golder Associates**

Geotechnical engineering.

- **ECL Envirowest Consultants Ltd.**

Environmental issues.

EXECUTIVE SUMMARY

The drainage catchment area for the Hatzic Lake watershed comprises approximately 9000 hectares or 90 square kilometres. Within the watershed a number of agencies have jurisdiction over overlapping sections of land. The major agencies consist of the District of Mission which occupies approximately 1,900 hectares, Crown Land which occupies approximately 2,650 hectares and the Dewdney Alouette Improvement District which cover approximately 3,200 hectares. No single authority has responsibility for operating and maintaining the drainage systems from top to bottom of the catchment, although the whole area lies within the Dewdney Alouette Regional District.

The area has been subjected to several flooding problems including land slides in the upper Pattison Creek resulting in heavy sediment deposition within the lower watercourses and wide spread flooding. On several occasions the Province has complete remedial work on an emergency basis under the Provincial Emergency Program. The record of expenditures to repair flood damage are incomplete, however the available records indicate the cost of remedial work during the past 25 years exceeds \$1,400,000.

Flooding in agricultural areas begins at approximately an 3 m elevation. The Everglades Resort on Hatzic Island begins to experience flooding when the water elevation reaches 3.35 m and buildings in low lying areas flood when the water level exceeds 3.85 m. During recent events lake levels have risen to elevations in excess of 4.3 m and has resulted in significant damage and distress to the low lying areas.

Approximately 4.4% of the total watershed has been logged. Logging in the area has not caused a significant increase in the magnitude of peak flows in the lower river system, however, it appears that logging is associated with debris slides and surface erosion of soil which has increased the sediment load in Lagace Creek. Deposition of sediment in Lagace Creek and some of the secondary creeks has raised the invert of the channel and contributed to some of the past flooding events.

The capacity of existing drainage works including floodboxes and pump station adjacent to the Fraser River and culverts under the Canadian Pacific Railway (CPR) tracks have been reviewed. The capacity of the CPR culverts is compatible with the capacity of the existing Fraser River floodboxes when the flow through these culverts is not restricted by debris.

A number of mitigation works have been reviewed, the impact of these improvement works has been assessed by modelling historic storm events which have caused flooding to determine what benefit may accrue should the improvement works be implemented.

The results of the modelling indicated the following:

- There would be no appreciable benefit to the drainage system capacity by dredging the lake. However, this would enhance the recreational use of the lake in summer.
- There would be no appreciable benefit during extreme flooding events to increase the capacity of the existing floodboxes, as the floodboxes are also closed for much of the time on the rising leg of the storm runoff hydrograph due to high water levels in the Fraser River. However, additional floodboxes will release the flood waters faster as the Fraser River level drops.
- The CPR culverts would only require upgrading if additional floodbox capacity were added.
- The greatest benefit would result from increasing the pumping capacity from 6 cubic metres per sec. (cms) to 14.4 cms. Additional benefits may be achieved by lowering the water level in the Lower Hatzic Slough and lake levels down to elevation 1.8 metres compared with the current setting of 2.44 metres. This lower lake level would however have a negative impact on recreational use of Hatzic Lake.

Additional conclusions reached include:

- ARDSA criteria for agricultural drainage would not be met by enlarging the existing drainage channels on Hatzic Prairie.
- Sedimentation is a significant problem, and will require annual main-tenance if the channel capacities are to be maintained at present levels.
- Dyking of Lagace Creek/Hatzic Slough north of the lake would result in higher water levels in Hatzic Lake during a storm, and would not eliminate flooding during extreme events.

A number of mitigation options and local drainage problems have been reviewed. A summary of items reviewed and estimated costs for each follows:

.1 Pump Station/Floodbox Structure

Pump Station Cost	\$1,908,300 - Triplex Submersible
Annual O & M Cost	\$44,100
Floodbox Capital Cost	\$ 815,500 if combined with pump station \$1,105,000 if built as a separate structure

.2 CPR Crossing Upgrade

Bridge Alternative
Capital Cost \$1,105,000

Culvert Alternative \$1,000,000 if no ground stabilization req'd*
capital cost \$1,750,000 if ground stabilization req'd*

* Soils investigation required to determine whether ground stabilization is required or not.

.3 Debris Control Structure at Hatzic Lake Outlet

Capital Cost \$93,000 to \$116,500

Not required if CPR crossing upgraded to a bridge

.4 Debris Control Structure at Pump Station

Capital Cost \$27,000

Not required if a bridge is not built at CPR crossing

.5 Dredging Hatzic Lake

1,700, 000 cubic metres of material to remove

Capital Cost \$12,000,000 to \$21,000,000
(depending on degree of trucking req'd)

.6 Lagace Creek Dyking/Local Pump Stations

Dyke crest established to resist the 50 year return period flow with 0.6 m freeboard

Capital Cost: \$9,862,000
(assumes use of dredged sediment from Lagace Creek)

.7 Raising Roads Above Flood Levels

Capital Cost \$2,360,000 to \$3,694,000
(depending on whether dredged materials from
Lagace Creek are used)

.8 Minor Watershed Improvements to Reduce Potential Sediment Supply

Capital Cost \$55,000
(Pattison Creek Watershed)

.9 Diversion of Pattison Creek to Amber Lake to Trap Sediment

Capital Cost \$2,250,000

Average Annual Maintenance \$30,000

(Filling of Lake may not be approved by Environmental Authorities)

.10 Annual Excavation of Sediment Along Lagace Creek

On-going Maintenance Cost \$2,025,000
(over 10 year period)

.11 Create Sediment Trap In Lagace Creek & Excavate Annually

Capital Cost \$366,000
(to build trap)

On-going Maintenance Cost \$2,025,000
(to excavate sediment over 10 year period)

Note: Commercial extraction of sediments considered not economically
viable

.12 Level Controls on Hatzic Lake

This would keep lake levels high during August for recreation and may result in
pumping which otherwise could be avoided.

Capital Cost \$92,000 - if installed at floodboxes not
including debris control

\$228,600 - if installed at Lougheed Highway
including debris control

.13 Erosion Downstream of CPR Culverts

Capital Cost	\$16,000 (reslope bank and place riprap)
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Note: Erosion will eventually stabilize if no capital work is completed.

.14 Various Local Drainage Problems

Several internal, localized problems were investigated, the location of problem areas and cost of improvement works are summarized as follows:

- | | | |
|---|--|-----------|
| • | Replace culvert at Suex Road and Sylvester Road | \$ 7,000 |
| • | Culvert west of 35584 Durieu Road | \$ 6,000 |
| • | Drainage of Robson property | \$ 15,500 |
| • | Storm drainage system for Sward Road Subdivision | \$230,000 |
| • | Drainage of 8689 Sylvester Road | \$ 13,500 |
| • | Drainage work on MOTH right-of-way south of
CPR on McKamie and Newton Roads | \$ 12,000 |
| • | Drainage work on CPR right-of-way | \$ 6,600 |
| • | Drainage work on private property south of
CPR on McKamie and Newton Roads | \$ 9,000 |
| • | Drainage at 8413 McKenzie Street | \$ 1,400 |

.15 Local Dyking and Flood Protection at Benbow Street Subdivision

Options of local dyking or raising residences were reviewed.

Cost to Build Dyke	\$147,000 (earth fill dyke)
	\$312,000 (concrete wall)
	\$400,400 (granite landscaped wall)

Cost to Raise Houses	\$760,000 (19 houses at \$40,00 average each)
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.16 Local Dyking and Flood Protection at Riverside Road Subdivision

Options for local dyking or raising residences were reviewed

Cost to Build Dyke	\$368,000
--------------------	-----------

Cost to Raise Houses \$960,000 (24 houses at \$40,000 average each)

.17 Replace Sylvester Road Bridge (Pattison Creek)

This would facilitate development on alluvial fan with a reduced risk of avulsions (channel realignments caused by sedimentation, etc.)

Capital Cost \$1,157,000

.18 Improvements to Existing Pump Station

Replace impellers to improve efficiency. Saving \$6,000 to \$19,500 per year in power costs

Capital Cost \$85,000

These improvement works may be combined into numerous different combinations. Four possible combinations would be:

Option 1: Low Cost Option

This is essentially a do nothing option insofar as the local governments are concerned, and the improvements noted would do little to alleviate the flooding problems for the overall area. The CPR and Ministry of Transportation and Highways (MOTH) could be approached and requested to carry out improvements on their rights-of-way as follows:

- Replace culvert at Suex Road and Sylvester
- Drainage work on MOTH right-of-way south of CPR
- Drainage work on CPR right-of-way
- Replace bridge over Pattison Creek on Sylvester Road

Total cost estimated to be \$1,182,600.

Regular clearing of sediment from Lagace Creek will also be required at an estimated average annual cost of \$200,000.

Option 2: Floodproofing Houses and Minor Works

This option would address local drainage problems, effectively eliminate flooding of habitable areas due to internal floods for properties which have previously submitted damage claims, and result in some reduction in pumping costs. The overall area would not benefit.

Works could include the following:

- Replace bridge over Pattison Creek on Sylvester Road
- Local floodproofing in Benbow Street & Riverside subdivisions
- Address local drainage concerns
- Construct water bars in Pattison Creek watershed to control sediment production
- Replace impellers in existing pump station
- Provide debris control structure at Loughheed Highway

Cost estimated would lie in the range \$2,051,000 to \$2,206,000. Higher cost involves dyking at Benbow and Riverside subdivisions; lower cost provides for raising the nine houses in Benbow subdivision which have previously submitted flood damage claims and doing nothing in Riverside subdivision. Annual operating and maintenance costs would also increase on average by \$200,000 for removal of sediment from Lagace Creek.

Option 3: Pump Station, Control Gates, Sediment Trap, Floodproofing Houses, and Minor Works

This option builds on Option 2 by adding a new pumping station at the Fraser River Dyke, control gates to maintain higher lake levels in the summer, a sediment trap in Lagace Creek and eliminating improvements to the existing pump station.

These improvements would benefit rural residential lands, landowners adjacent to Lagace Creek downstream of the sediment trap, homeowners who participate in local drainage and floodproofing works, farmers with land or buildings in the low lying area, the Everglades Resort, and recreational users of the lake.

Costs for this option range from \$4,332,600 to \$4,487,600, depending, as in Option 2, on the nature of flood protection in the Benbow and Riverside Road subdivisions. Annual operating costs would increase on average by \$224,000 to cover sediment removal and pump station operating costs.

Option 4: Floodboxes, CPR Crossing, Pump Station, Control Gates, Sediment Trap, Floodproofing Houses, and Minor Works

This option builds on Option 3 by adding floodboxes at the Fraser River Dyke and replacing the CPR culverts with a bridge. The beneficiaries of this option are the same as for Option 3, and the added benefit of the works described would be minimal.

Costs are estimated at \$6,160,100 to \$6,315,100 depending on the option chosen for floodproofing in the Benbow and Riverside Road subdivisions. Annual operating and maintenance costs would be on average an additional \$224,000 as noted for Option 3.

Other Major Works

The following works would appear to be too costly in relation to the potential benefits derived:

Diversion of sediment to Amber Lake	\$ 2,350,000
Dyking Lagace Creek & Hatzic Slough	\$ 9,862,000
Dredging Hatzic Lake	\$12,000,000 to \$21,000,000
Raise Roads in Low Lying Areas	\$ 2,360,000 to \$ 3,694,000
Total Capital Costs	\$26,572,000 to \$36,906,000

Project Implementation

The local community must now determine which works should be pursued, who benefits from these works, what institutional or governmental framework will administer the construction and maintenance of the works, and how the works will be funded.

With respect to possible sources of cost sharing by senior governments several programs are, to varying degrees, still in effect; however, virtually all of these programs are either being phased out or have no funding allocated. These include ARDSA, Fraser River Flood Control Program, and the Provincial Flood Protection Assistance Program.

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1.0 INTRODUCTION

1.1 PROJECT BACKGROUND AND SCOPE

The Hatzic Lake watershed is located on the north side of the Fraser River east of Mission. The watershed, which consists of approximately 9,000 ha, contains a variety of land forms including a large lake, Hatzic Lake, valued by local residents as a recreational resource, low lying level arable land and high, steep sided, mountainous terrain.

The catchment drains to a central watercourse, Lagace Creek, which flows generally in a southerly direction to combine with Hatzic Slough before discharging into Hatzic Lake (see Drawing No. 1.2 Prairie Key Plan). The lake in turn flows out via the southern portion of Hatzic Slough under the Lougheed Highway and Canadian Pacific Railway (CPR) mainline tracks to the Fraser River. Flow into the Fraser River is controlled by four floodboxes and a pump station which allow the local drainage to pass through the Fraser River dyke.

The watershed area is subject to many problems. Several landslides have occurred in the Pattison Creek watershed, which lies on the east side of the catchment, and during large storms, heavy sediment loads are transported downstream. Material deposition occurs in the low lying areas. In addition, bank erosion occurs in localized areas potentially threatening nearby buildings. Occasionally, the creek has broken out of its established channel flooding nearby buildings and spreading gravel over adjacent fields. On several occasions, the Province has completed remedial work on an emergency basis under the Provincial Emergency Program (PEP).

In large regional storms and during the Fraser River freshets, the floodboxes at the Fraser River remain closed and the Hatzic Prairie lowlands are completely dependent on pumping for flood protection. The existing pumping capacity is considered inadequate as widespread flooding has occurred.

The flooding of homes and barns is most severe in intense winter storms. The properties that are most affected appear to be in the Benbow Street subdivision and the Riverside Road subdivision off Farms Road where the houses were built before floodplain regulations came into effect.

The greatest flooding damage to crops has occurred as a result of large summer storms. The agricultural area that is most affected is a large depression of about 500 ha in Hatzic Prairie, about 2 km north of the lake and centred on the Farms

Road and Dale Road area. The low elevation of this area and its location on the far side of the lake from the pump station, make it difficult to drain this area and prevent it from flooding.

Another problem that occurs towards the end of summer is the low water level in Hatzic Lake when the Fraser River levels recede. The shallow areas of the lake become exposed mud banks and the recreational value of the lake is substantially diminished.

In response to these concerns and the need to stabilize the creek and provide for its regular maintenance, the Dewdney-Alouette Regional District (DARD) commissioned this study to develop a comprehensive list of options designed to address these concerns. The work of the study is intended to update previous reports and address concerns raised by local residents. This report is prepared to provide landowners and the different levels of government with a technical and cost basis for deciding on future actions in the project area.

The Terms of Reference which detail the scope of services to be provided and problems to be addressed are provided as Appendix A of this report.

1.2 STUDY AREA AND GENERAL DESCRIPTION

The Hatzic Prairie drainage watershed is located on the north side of the Fraser River east of Mission as shown on Drawing No. 1.1, Location Plan. The total watershed is approximately 90 km² (9,000 ha) of which 337 ha is occupied by Hatzic Lake. Low lying arable area located around the lake occupies a further 2,531 ha. In addition, approximately half of the watershed is steeply sloping mountain side, some of which has been logged. The flood plain area is predominantly agricultural land with scattered small holdings, small subdivisions and recreational property. The lake is a popular area for swimming, boating and other water sports during the summer.

The Hatzic Prairie watershed lies within the Dewdney-Alouette Regional District. The southwest portion of the watershed lies within the District of Mission, while the north and northeast portion of the watershed are in Electoral Area E and the southeast portion is within Electoral Area B. A number of special interest groups exist within the floodplain area including the Hatzic Island Ratepayers Association, the Dewdney Ratepayers Association, the Hatzic Prairie-McConnell Creek Ratepayers Association and the Sheltered Cove Improvement District.

Several mountain streams drain the highland areas down to the central lowland where the flow is collected by Lagace Creek, which in turn, discharges into Hatzic Slough and Hatzic Lake. Approximately 95% of the catchment drains into Hatzic Lake which discharges through a bridge at the Lougheed Highway and three culverts

under the CPR mainline tracks to the Lower Hatzic Slough. The lower reach of Hatzic Slough flows south to its confluence with the Fraser River. Near to the Fraser River all of the runoff must be discharged either through four existing floodboxes at the Fraser River dyke, or pumped by an existing pump station which provides a capacity of 6.06 m³/s pumped discharge against peak Fraser River flows. Drawing No's 1.2 and 1.3 provide details of the Hatzic Prairie watershed and major tributaries and roadways in the central portions of the catchment.

The watershed boundary indicated on Drawing No. 1.3 is based on the topography of the area. Within the District of Mission, which occupies approximately 19 km² of the watershed, some areas are serviced by an underground, piped storm drainage system. The layout of this system has been reviewed and it has been confirmed that this piped storm drain system has not significantly altered the natural watershed boundary.

In large regional storms, and during the Fraser River freshet, the floodboxes are closed and the lowlands are completely dependent on the pumps for flood protection. The existing pumps are considered to be quite inadequate to the task and widespread flooding occurs. The flooding of houses and barns is most severe in the intense winter storms when higher flood stages on Lagace Creek and Hatzic Slough are reached. During these periods roads within the low lying areas of the catchment can be covered by high flood waters for several days causing considerable additional inconvenience. However during winter months, the floodboxes will normally open within a period of less than three days, greatly increasing the out-flow and providing some flood relief. Houses and developed areas most affected by flooding appear to be in the Benbow Street and Riverside Road subdivisions where houses were built before floodplain regulations came into effect. The Everglades Recreational property at the south end of Hatzic Island and the trailer park at the north end of the Hatzic Island are also areas that are heavily impacted.

Damaging storms also occur during summer months when agricultural crops are most sensitive to damage due to prolonged inundation. The agricultural area that is most affected by the flooding is a large depression of approximately 500 ha within Hatzic Prairie. This area is located approximately 2 km north of the lake and is centred on the Farms Road and Dale Road area.

Many efforts have been made to deal with the problems and a number of previous studies have been completed. Two basic problems exist in the area: the volume of storm water runoff; and sediment transport from a large upland section of the catchment, which impact heavily on a rural area which has no overall authority and only a limited tax base available to deal with the problems.

.1 Reference Material

A substantial quantity of reference material, as detailed in the project Terms of Reference, was available for use on the study. This material included oblique air photographs taken during the November 1990 storm showing flooding on the Hatzic Prairie. In addition, surveys of Hatzic Slough and Lagace Creek taken during 1982, 1984 and 1990 were provided. Mapping was also provided at 1:5000 scale showing contours, planimetrics, role numbers and parcel sizes. This mapping was derived from aerial photographs taken during September 1983. Sixteen previous reports completed by a number of agencies and consultants between 1963 and 1990 were also provided for background information. The value of land and improvements was obtained from information provided by the Regional District and was based on the 1990 assessment role. This reference material proved invaluable in establishing an understanding of the problems that exist in the project area.

The accuracy of the existing mapping was raised as a concern at the start of the project. As this mapping was to be utilized to develop conceptual designs for improvement options, including cost estimates, the accuracy of this information was clearly important. In order to assess the validity of the topographic mapping two independent approaches were adopted.

The first approach was to review the water levels on the Fraser River and compare them to levels calculated using the hydraulic grade line of the river. The water level surface shown on the Fraser River at the confluence with Hatzic Slough is 1.3 m (Drawing No. 92G.019.2.4). The water surface level on the Fraser River at the confluence with Nickomen Slough is 1.6 m (Drawing No. 92G.020.1.3). Under a previous assignment the hydraulic grade line on the Fraser River has been calculated to be 0.00006 by Northwest Hydraulics Consultants Ltd. Using this hydraulic grade line and the distance between the two elevations shown on the drawings of 5.5 km an elevation difference 0.33 m should exist. This compares favourably with the 0.3 m shown on the drawings.

In addition, field survey was carried out in the Hatzic Prairie area to compare actual field survey elevations with those elevations shown on the drawings. Elevations selected from the drawings occurred typically at road intersections or on the centre of bridges. The exact locations of other point elevations shown on the drawings would be difficult to determine with any degree of certainty. A survey traverse was completed commencing at the intersection of Farms Road and Sylvester Road. The traverse proceeded west along Farms Road, north on Farms Road and east on Dale Road to the intersection with Sylvester Road. The results of this survey are summarized on Table 1.1 together with the data shown on available topographic mapping.

The variation between the elevation shown on the mapping and field survey elevations at these 14 locations is considerable and varies in the range -0.41 m to +0.47 m. No apparent trends exist. The average variation is calculated to be +0.089 m with a standard deviation of 0.242 m.

This variation must cause some concern with respect to the quantity calculations used to calculate some of the construction cost estimates included in this report. However the average ground elevation shown on the mapping is close to that obtained by field survey and therefore it is reasonable to assume some balancing will take place and the overall quantities and resulting estimates are considered reasonable.

.2 Relationship between the Fraser River Gauge at Mission and Downstream Gauge at Pump Station

The relationship between the Fraser River stages on the downstream side of the floodboxes (confluence of Hatzic Slough) and the Water Survey of Canada (WSC) gauge at Mission was monitored during the period starting June 13, 1991 to September 6, 1991. These results are shown on Drawing No. 1.4.

For the relatively short period these readings were carried out, the Fraser River stage at Hatzic Slough downstream of the floodboxes is on average approximately 0.3 m (1 ft) higher than the WSC gauge at Mission. This is also confirmed by previous studies carried out by Northwest Hydraulic Consultants where the hydraulic grade of the Fraser River was determined to be 0.00006. Based on the measured distance of 5 km between the Mission gauge and Hatzic Slough, the elevation difference is also calculated to be 0.3 m.

The estimated flows at Mission are also plotted on Drawing No. 1.4. The estimated flow variation also follows the stage variation. WSC do not release flow information at Mission immediately as these flows are calibrated from their model, results of which are available only about a year later. Therefore the flow values shown on Drawing No. 1.4 must be considered as approximate.

1.3 PREVIOUS FLOODING EVENTS

The Hatzic Prairie area has been plagued with numerous floods over many years. The first recorded flood occurred in 1882 when Fraser River flood water passed through a 398 m (1305 ft) long trestle bridge of the CPR track into Hatzic Prairie. Flood levels at that time were noted as the highest in recorded history although no absolute level appears to be available.

Following this flood, it is understood the trestle of the CPR track was replaced with an embankment and four floodboxes. This embankment was intended to act as a dyke to protect the low lying area against floods caused by the Fraser River freshet. However in 1894 a flood which exceeded that of 1882 by 0.76 m occurred which washed out the CPR embankment complete with the floodboxes. The damaged embankment was repaired with a new trestle and therefore for a number of years the area had no dyke protection.

After a period of approximately 12 years, an agreement was again reached with CPR to utilize the embankment as a dyke and to construct new floodboxes and a small pump station. These new works appear to have protected the low lying area from Fraser River high water levels, however it is understood through this period, flooding continued due to high flows in Lagace Creek and sedimentation problems. In June 1948, the Fraser River rose to extreme levels once again; the CPR line was washed out and many residents in the low lying areas had to be evacuated. CPR once again reconstructed their tracks through this area, utilizing a trestle with an opening approximately 170 m long in the embankment. However, following this flood, CPR refused permission to utilize their embankment as a dyke and as a direct result, the Fraser River dyke was constructed along the right (north) bank of the Fraser River in 1949. The dyke reclaimed 730 ha (1805 acres) of land south of the CPR line which became organized in February 1950 as the South Dewdney Dyking District. The dyke constructed at this time was approximately 11.8 km long and 9 m high. In addition the pump station and four floodboxes which still remain at the confluence of the Hatzic Slough and the Fraser River were constructed.

In 1959 the CPR trestle was replaced with three culverts: two 3.05 m diameter and one 2.44 m diameter CSP culverts and these structures have remained in service since that time. The Fraser River dyke and floodboxes have provided the area with good protection from high water levels in the Fraser River since 1949 with some upgrading provided during 1987 and 1988. Internal drainage problems have however persisted throughout this period and numerous floods have occurred in the study area due to a variety of reasons.

In 1966 flooding of Lagace Creek caused 78 residents to submit a petition to the Water Rights Department requesting immediate action. Again, on September 17, 1968, Lagace Creek overflowed its banks downstream of Farms Road bridge, spilling across Farms Road. Flooding has since occurred in December 1979, June and July 1981, January 1983, July 1983, November 1989, February 1990 and November 1990. In addition flooding and avulsions have occurred on other minor creeks in the catchment area.

Flooding in agricultural areas begins at approximately 3 m (9.8 ft), flooding of the Everglades resort area on Hatzic Island begins at 3.35 m (11 ft) and flooding of buildings in low lying areas begins to occur when lake levels reach 3.85 m (12.63 ft).

During the events noted above, lake levels have risen to elevations in excess of 4.3 m (14.1 ft) and as a result, significant damage and distress is associated with these flooding events.

1.4 SEDIMENT-RELATED PROBLEMS

Sediment related problems exist at a number of locations throughout the catchment area including Pattison Creek, Lagace Creek, Hatzic Lake, Hereford Creek and Eng Creek. Significant land slides and debris flows have occurred on both the west and east sides of the catchment. These landslides and debris flows can cause significant damage as occurred in December 1980 when 50 m of Stave Lake Road was inundated with approximately 2 m of debris and one house was ripped off of its foundation and was irreparably damaged.

On the east side of the Hatzic Prairie in the Pattison Creek catchment, several landslides occurred between 1930 and 1960 along with small debris torrents. The most recent event in this area occurred in November 1990 when a landslide coupled with a debris torrent brought down more material in a single year than at any other time in recorded history. Material mobilized in the upper catchment from the steeply sloping mountain and hillsides is carried down the streams by water to the lower sections of the catchment where it is deposited in the stream beds and lake.

The most active sources of debris are in the Pattison Creek catchment. The sediment from this area is within the Lagace and Pattison Creek system where the sediment deposited in the creeks reduces the capacity of the existing channels, increasing the risk of avulsions and flood related damages.

The problems related to sediment such as aggradation of the creek beds, bank erosion, avulsions and creation of new channels, and slow filling of Hatzic Lake have accelerated in recent years, primarily because of recent debris torrents. The most recent debris deposition in Lagace Creek and lower reaches of Pattison Creek has been cleaned out under a Provincial Emergency Program funded project where 96,000 m³ of sediment was removed at a cost of about \$660,000. From an assessment of the Pattison Creek watershed it seems probable that more sediment will be generated from this watershed and the above mentioned problems will become more acute than in the past.

1.5 LOCALIZED PROBLEMS AND CONCERNS

In addition to the major problems of flooding and sedimentation which exist in the project area, a number of other local problems or concerns exist.

One concern is associated with Hatzic Lake. The lake provides a valuable recreational resource to the local residents who utilize the lake for boating, fishing,

swimming and other water sports through the summer months. During June and July the lake level is usually held at an elevation of 2.44 m as the flap gates on the floodboxes at the Fraser River dyke are held closed due to the annual freshet in the Fraser River. During August, when the peak flows in the Fraser River begin to subside, the floodboxes open and the water begins to drain by gravity out of the lake. Over many years the lake has become silted up with silts and clays which are carried into the lake by Hatzic Slough and other tributaries. As a result, when the lake level recedes, mud flats are exposed and the water becomes too shallow over large sections of the lake preventing the continued recreational use during August. The possibility of dredging the lake to remove sediments or maintaining lake levels at a higher elevation through August is therefore a goal of a number of local residents.

Another concern relates to a property on the south side the CPR rail line where aggressive bank erosion has occurred on the Hatzic Slough. The owner of the property, which is located on the left bank of the slough, is concerned regarding loss of land and has approached BC Environment to determine what measures could be put in place to prevent further bank erosion.

The final local issue, addressed within this section of the report, relates to drainage concerns which do not appear to be linked to the major flooding events which have occurred in the project area. A number of local residents have indicated on questionnaires, which were returned to the Regional District following the most recent major floods, that drainage problems exist on or adjacent to their properties which would appear not to be related to the high lake levels.

Each of these three concerns will be reviewed in detail, the causes of problems identified and where solutions are feasible, these are presented with construction cost estimates in Section 4 of this report.

1.6 COSTS AND CONSEQUENCES OF FLOODING AND SEDIMENTATION

The flooding and sedimentation problems occurring in the Hatzic Prairie drainage catchment are both significant and recurring. The impact of these recurring events is extremely difficult to measure. A significant financial investment is made on a repeated basis through the catchment in an attempt to remove sediment from the drainage channels and maintain the creeks within the banks of their natural watercourses. In addition substantial payments have been made to residents in the catchment area who have incurred damage from flood waters. These claims however do not truly reflect the cost of the problems which exist in this area. The Provincial Emergency Program will not pay any claim which could have been covered by

insurance. Numerous farmers have experienced crop damage for which no compensation is normally payable under the Provincial Emergency Program. Therefore, as the farmers recognize submitting claims is futile, the true record of the cost of this damage is unknown.

In addition, the existing drainage facilities in the project area do not permit control of water levels to satisfy accepted Agricultural and Rural Development Subsidiary Agreement (ARDSA) drainage criteria for farming areas. The ARDSA criteria recommend the water surface be maintained at an elevation 1.2 m below ground level in arable areas to allow drain tiles to function and to ensure the productivity of the land is fully realized. As the existing facilities are unable to accommodate this criteria, the farmers are unable to realize the potential of their farmland.

Notwithstanding the obvious financial consequences of the flooding and sedimentation, for the people living in the Hatzic Prairie area, each storm is a cause for apprehension. With the memories of recent floods still clear, any prolonged rainfall is understandably a concern as the threat for considerable damage and discomfort is very real.

In an attempt to quantify the cost of flooding and sedimentation, the following statistics have been assembled from the files of BC Environment, Dewdney-Alouette Regional District, and the Provincial Emergency Program:

- Value of land and improvements located below the 4.15 m contour for land adjacent to Hatzic Lake including Sward Road and below 4.6 m north of Hatzic Lake and subject to flooding on a repeated basis is assessed at \$7,018,994.

A more detailed breakdown of land values in areas susceptible to flooding is:

• Everglades and Hatzic Island	\$ 709,605
• Benbow Street Subdivision	377,464
• Sward Road Subdivision	733,092
• Riverside Road Subdivision	943,106
• General Farmland between 3.6 m and 4.6 m	791,915
• General Farmland below 3.6 m	3,630,255
• Farmland south of Chilqua Slough	<u>598,052</u>

TOTAL \$7,783,489

The value of land and improvements is based on tax assessment values not appraised market values.

- Total cost of sediment clean-up from creeks completed by BC Environment \$1,194,451.

- Replacement cost of bridge washed out at Seux Road and Lagace Creek \$208,000.
- Value of agricultural damage recorded by local residents for storms during both summer and winter months may be summarized as follows:

Date	Total Loss or Claim Amount	Equivalent Value in 1992 Dollars
June 1981	\$108,100.00	\$173,407.00
July 1983	496,085.00	681,987.00
November 1989	28,810.89*	32,040.00
November 1990	14,780.67	15,585.00
Totals	\$647,766.56	\$903,019.00

- * Including \$24,360 paid by PEP for reseeding as noted below.

During this 10-year period, this represents a cost of \$90,302 per year for agricultural damage alone.

- Payment made for property damage by Provincial Emergency Program in the value of \$118,367.01 for flood damage in 1989 and 1990 of which \$24,360 was paid for reseeding crops. These payments are made after any deductible has been deleted. Of 19 claims, no value was stated on seven of the claims. Therefore based on 12 claims, the average value of the claim approved was \$9,864. The claims covered damage to homes, businesses and farms and include the damage to buildings, contents, equipment and cleanup.

1.7 EXISTING FLOOD PROTECTION WORKS

Existing facilities designed to protect property against damage due to flooding are limited to the Fraser River dyke which incorporates four 1.83 m x 1.83 m floodboxes and a pump station, all of which were constructed in 1949. This facility is designed to protect the Hatzic Prairie watershed against flooding from the Fraser River during periods of extreme high water in the Fraser River. Other drainage works in the area, as detailed on Figure 2.23, have been constructed to accommodate the passage of internal drainage, however they are not designed to provide flood protection against major internal flooding events. No measures are in place to provide any mitigation measures or control of sedimentation problems in the catchment area.

The existing facilities have performed as intended since construction in 1949. The dykes were rehabilitated during 1986 to 1989 to comply with Fraser River Flood

Control Program standards. Apart from this work only normal ongoing maintenance has been required. The dyke is designed to protect the low lying area against a 1 in 200-year return period flow in the Fraser River. An inspection was completed of the existing pump station to review the condition of the existing station and assess the prospect of an extended operating life into the future.

The Hatzic Slough pumping station is located in the Dewdney Area Improvement District close to the confluence of the Hatzic Slough and Fraser River. The station consists of a two bay concrete sump protected by bar screens and a wood framed superstructure. The station accommodates two Paramount 1.22 m (48 inch) axial flow pumps driven by 350 horsepower English Electric squirrel cage induction motors operating at 495 rpm. The power supply to the pumps is 480 volt, 3 phase, 60 hertz. The rated capacity of each pump is 3.03 m³/s (40,000 Igpm) at 5.49 m (18 feet) total head.

Significant elevations (GSC) at the pump station are as follows:

- Bottom of sump, -0.30 m (-1.0 feet) approximately
- Bottom of suction bell, 0.38 m (+1.25 feet) approximately
- Centreline of pump discharge, 3.78 m (12.4 feet), pump No. 1, 3.73 m (12.23 feet), pump No. 2.
- Eye of impellers, 0.91 m (+3.0 feet) approximately
- Pump house floor, 4.82 m (+15.8 feet)

Static heads can vary between 1.34 m (4.4 feet) and 6.1 m (20 feet) depending on the level of the water in the Fraser River and also assuming that the Hatzic Slough is maintained at elevation of 2.44 m (8 feet).

Based on the pump manufacturer's curves, the pumps would operate at best efficiency of approximately 80% when the total dynamic head is 5.49 m (18 feet). To achieve this head, the Fraser River would need to be at an elevation of 7.01 m (23 feet) which occurs infrequently. The normal total head would lie within the range of 2.44 m to 3.05 m (8 to 10 feet) and therefore the pumps operate at an unstable and low efficiency point for much of their operating life.

Based on electrical demand figures supplied by BC Hydro it would appear the hydraulic efficiency of the pumps probably lie within the range of 40% to 50% for most of their operating period. In addition, readings taken on the primary side of the transformers have been recorded as high as 660 kw during May and June of 1991 with a stated average of 500 to 540 kw. This data suggests brake horsepower at the motor shaft would lie in the range of 293 horsepower to 375 horsepower peak. At the maximum condition this represents an intrusion into the motor service factor of approximately 50%. The pump logs indicate that the pumps (generally pump No. 2) "kick off" from time to time indicating a possible overload of the starter.

Within the normal operating range of the pumps, as observed during the site inspection, cavitation occurs. This could result in pitting damage to both the pump impeller and volute although no damage has been reported. Hydraulic Institute standards recommend a minimum submergence of 3.8 m (12'6") for flows of 50,000 US gpm and based on the critical elevations stated earlier, the submergence with water levels of 2.44 m (8 feet) in the slough would be only 2.05 m (6.75 feet). The Hydraulic Institute recommendations are recognized as being perhaps conservative. Based on data obtained from Aurora Pump for a comparable pump the recommended submergence would be 2.7 m (8'10"). No vortexing was apparent during the site inspection when pumps were operated and the water level was 2.65 m (8.7 feet) GSC.

The pump control system is basic but adequate for the task in hand. Three float switches, suspended into the inflow channel of the flood box, control the pumps in the automatic mode. A fourth float switch is suspended into the sump which provides low level protection for both pumps. There are no high level or low level alarms or indication. The pumps are alternated automatically after each cycle and a time delay has been provided to allow at least a 30 minute lapse between pump stop and the next pump start. Such a feature is essential given the minimum level differential between pump start and stop.

The original motor starters were replaced with new units in 1984 and these units appear to be in good condition. Other controls have also been replaced since the original construction and the present small control panel is in good condition.

No standby power facility exists for this pump station and therefore, in the event of an interruption in power supply to the facility, the station would be unable to function. This situation is not unusual for drainage pump stations in the lower Fraser Valley area.

The existing pump station, despite the fact that it is operating outside of its recommended service range, has performed satisfactorily for a long period of time and there is no reason to believe it will not continue to do so.

The existing trash racks are satisfactory but difficult to clean. The racks could be replaced with new screens designed to facilitate cleaning.

1.8 EXISTING INSTITUTIONAL FRAMEWORK

Six organizations and authorities are represented in the study area who potentially would have responsibilities of flood protection and internal drainage. The organizations concerned are the Dewdney-Alouette Regional District, BC Environment, Ministry of Transportation and Highways, Canadian Pacific Railway, the District of Mission, and the Dewdney Area Improvement District (DAID). Each

of these organizations was contacted to determine the responsibility for each of the organizations and to identify capital works expenditures, emergency works expenditures, regular operation and maintenance expenditures, and local taxation revenues for flood protection purposes over the past 25 years. CP Rail declined to be interviewed, however they did respond to written questions regarding their facilities in the study area.

In general, records regarding expenditures and taxation revenues were incomplete with the exception of the Dewdney Area Improvement District and therefore it is not possible to provide a comprehensive statement of revenue and expenditure for flood protection and drainage purposes in the project area.

Three of the organizations, CP Rail, the Ministry of Transportation and Highways, and the District of Mission, each have a responsibility for drainage within the study area. The responsibility for each of these groups is however limited to specific areas.

The District of Mission operates and maintains existing drainage works within the municipal boundary. No capital works have been completed by the District. Any capital work requirements would result from new development in the area and would be limited to storm drainage works to service housing. These works would be completed by the developer at no cost to the District. Draper Creek is the only significant tributary from the District of Mission into the Hatzic area and within this catchment there is very little enclosed drainage and only minor development has proceeded. The District is unaware of any ongoing drainage problems and records indicate \$106,605 has been spent on main-tenance within the catchment area during the period 1989 through 1990. Drainage works are completed from general revenue and therefore no specific numbers are available for taxation revenues.

The Ministry of Transportation and Highways (MOTH) carry out a maintenance program which is intended to divert water from road surfaces. The principal concern of MOTH is to provide clear and open channels for water to flow either across the right-of-way via structures or parallel to the road via ditches to the closest natural watercourse. The Ministry is responsible for all roads within the study area except for the roads within the District of Mission. The mandate of the Ministry, however, is limited to work within rights-of-way, except where upstream obstructions such as beaver dams exist which may threaten to break or cause excessive channel flow or debris flows resulting in damage to existing structures.

Maintenance costs are not identified by area within the Ministry and therefore the actual expenditure on maintenance is not available. The Ministry has not completed significant capital works for drainage purposes in the study area with the exception of the construction of Seux Road bridge. This bridge over Lagace Creek was washed out and reconstructed after several years in 1983 at a cost of \$208,000. The Ministry's facilities are maintained by a general contractor engaged through a

maintenance contract. Maintenance includes stream beds drainage appurtenances, ditches and water courses. In addition flood control and wash out response is provided. Each of these services, however, is intended to ensure the safety of the road structure and the travelling public. In addition flood protection and emergency response has historically been provided, however, such response has only been provided when the Provincial Emergency Program declares an emergency exists and the local authority or municipality has exhausted all of their resources.

CP Rail carry out maintenance in the study area. However CPR's interest is limited to its own right-of-way. The principal structures in the study area under the responsibility of CP Rail are three culverts through the railway embankment downstream of the Loughheed Highway bridge. These structures are the only outlet for the Lake which flows south into the lower reach of Hatzic Slough. CP Rail have confirmed it is responsible to maintain any structure under its ownership, including removal of debris as required. It is understood CP Rail has not completed any significant capital expenditures at this location within the last 25 years and no records are available regarding the cost of regular operation and maintenance.

BC Environment confirmed that it is not responsible for and has not completed any maintenance and operational work in the area. Construction projects have been limited to those completed under the Fraser River Flood Control Program. Other programs which have existed include the River Protection Assistance Program, however there is no record of this program being utilized in this catchment area.

Prior to the mid-1970's, any emergency work required on private property was typically carried out by the Ministry of Transportation and Highways. During the mid-1970's, the project management of this work was taken over by BC Environment and a number of projects have been completed under the Provincial Emergency Program (PEP). This work is basically limited to the restoration of flood damage such as sediment removal and creek restoration following flood events. Funding for PEP projects is provided through the Ministry of Solicitor General under special warrants issued by an Order in Council. The project may be eligible for some assistance from the federal government but not unless the cost of eligible work exceeds \$1.00 per B.C. resident (currently \$3,400,000) per flood incident. Work completed by PEP has been carried out during the years 1984, 1986, 1987, 1988, 1990 and 1991 and has covered the cost of sediment removal from Lagace Creek and some bank protection works. During the most recent sediment removal program by PEP in 1990, approximately 96,000 m³ of sediment was removed from Lagace and Pattison Creeks. The total cost of this work is estimated to be approximately \$962,611 between 1984 and 1991. Prior to 1984 incomplete records exist, however Lagace Creek and Pattison Creek have been cleaned on numerous occasions dating back to 1953. The cost of each creek clean-up varied between approximately \$10,000 to \$100,000 per year for a total recorded expenditure before 1984 of in excess of \$250,000. Therefore, although the exact expenditure is unclear, it is evident

significant sums of public funds have been expended on an ongoing basis within the catchments of Lagace and Pattison Creeks to restore the existing channels and overcome problems caused by sedimentation. All indications are this problem will continue into the future and potentially increase in severity.

The Dewdney Area Improvement District (DAID) is responsible for dyking and drainage of a defined area of land and for the construction, acquisition, maintenance and operation of works for these purposes. The area included within DAID is approximately 3,200 ha and includes the area south of Chilqua Slough and Hatzic Lake and the area of Hatzic Prairie, all of which would be inundated in the event of a breach of the Fraser River dyke during high flow events in the Fraser River. In reality, the Improvement District operates the pump station located near the confluence of the Hatzic Slough and Fraser River and maintains this pump station with the associated floodboxes and dyke within the Improvement District. The Improvement District has not completed any other drainage works and does not maintain any registered access easements or rights-of-way for channel or ditch maintenance. It is understood that DAID has no interest in taking on the responsibility for ditch and channel maintenance in the future.

The Improvement District was organized and began operating in the early 1890's as the Dewdney Dyking District. At that time, following the flood of 1894 the railway embankment was utilized as a dyke with internal drainage accommodated by floodboxes and a small pump station. The Dewdney Dyking District continued to operate with some financial difficulties up until the flood of 1948 which washed out the railway grade and control structures. Following this flood, CP Rail refused the District permission to once again utilize their embankment as a dyke and as a result higher levels of government constructed a new dyke along the right (north) bank of the Fraser River together with the pump station and floodboxes which remain in operation to date.

This new dyke reclaimed approximately 730 ha of land. This new area was prevented from amalgamating with the Dewdney Dyking District as a result of the Dyking District's debt and therefore in February 1950, the South Dewdney Dyking District was organized for this new land and the two districts shared responsibility, including operating and maintenance costs, for the new dyke, floodboxes and pumps. This arrangement continued until 1972 when the current Dewdney Area Improvement District was formed which incorporates the two earlier dyking districts.

DAID levy taxes within the Improvement District and set a mill rate based on assessed value. The mill rate is constant and independent of land use. Each year the mill rate is adjusted based on the previous year's operating costs and anticipated rate increases for power consumption. A minimum tax of \$50 is currently collected from each property within the Improvement District. A summary of the Improvement District's revenues and expenditures by year for the Dewdney and

South Dewdney Districts and the DAID is provided on Table 1.2. The DAID currently holds a reserve fund of approximately \$170,000 which has been assembled over time to enable the District to complete any major repairs which may become necessary.

The single organization which maintains some responsibility throughout the entire catchment area is the Dewdney-Alouette Regional District. DARD has however no responsibility for drainage capital works, or operating and maintaining drainage facilities in the study area. Regional Districts typically have a mandate for municipal financing and general government including hospitals. To expand these services, the Regional District requires a bylaw and possibly a vote by the electorate to obtain the authority to provide any additional services. This specified area function would require authority granted by the Lieutenant-Governor in Council. The Regional District has not indicated any interest in accepting this mandate and this would undoubtedly be a decision for the Regional District Board to determine. Should DARD accept the specified area function, it would require formal access rights-of-way to complete any maintenance. The corporate power of the Regional District allows access onto private land for water and sewage systems but not for drainage systems. The Regional District does have the power to expropriate property if required.

In conclusion, the catchment area which consists of approximately 9,000 ha contains large areas of overlapping jurisdiction under the control of:

Jurisdiction	Approximate Area (hectares)	% of Catchment
Dewdney Alouette Regional District	9000	100%
District of Mission	1900	21%
Crown Land	2650	29%
Dewdney Alouette Improvement District	3200	36%

The storm water runoff from these contributing areas in rough measure will be approximately proportional to their areas. No single authority has responsibility for operating and maintaining the drainage systems from top to bottom of the catchment. In organized areas such as municipalities, surface drainage from roads and from public and private property is the responsibility of the local authority. In unorganized areas such as this, surface drainage from private property is the responsibility of the property owner.

Table 1.2 summarizes revenue and expenditures for flood protection and drainage works during the last 25 years. The total sums are, on occasions, approximate only and incomplete and therefore this summary may act only as a guide to the cost of these works.

2.0 HYDROLOGY

2.1 FLOW FREQUENCY ESTIMATES

.1 Introduction

As no streamflow data are available for Lagace Creek or its tributaries, data from the adjacent basin have been used. Rain gauges on the north side of the Fraser Valley were also used in the analysis; however, because they are at lower elevations, they are not considered to be representative of the higher elevations in the Hatzic basin. This section begins by reviewing available data and previous studies, estimates flows for Lagace Creek and its tributaries for specified frequencies, analyses specific storm events, and then comments on the effect of logging and future upland development on flood hydrology.

.2 Available Data

Streamflow data is not available for Lagace Creek or any of its tributaries. Lagace Creek (Drawing No. 2.1) is fed by five major tributaries from forested mountainous regions to the east, extending to elevations close to 1200 m, and by four major tributaries from forested hill areas to the west, extending to elevations of about 550 m. Of the approximately 55 km² drainage area at the entrance to Hatzic Lake, roughly 70% is mountain and hill area and the rest is flat valley-bottom land in the Hatzic Prairie, the Lagace Creek alluvial fan, and the area west and south of Allan Lake.

The closest gauged streams that appear suitable as indicators for the hydrologic characteristics of Lagace Creek are Norrish Creek, about 7 km to the east, and Kanaka Creek, about 15 km to the west (Drawing No. 2.1). Both streams have records extending from 1960 to 1990, although maximum flows are missing for some recent years, especially in Norrish Creek. Norrish Creek has a drainage area of 117 km². It is almost all mountainous and generally similar to the eastern part of the Lagace Creek basin, but with some lakes in the headwaters. Kanaka Creek has a drainage area of 48 km², with generally flatter slopes than Lagace or Norrish Creeks but extending to elevations of over 900 m.

Table 2.1 lists reported annual maximum instantaneous flows (instantaneous peaks) for Norrish and Kanaka Creeks from 1960 to 1990, and the sequences are plotted in Drawing No. 2.2. There is no indication of significant time trends. In 12 out of the 31 years, the peaks in both creeks resulted from the same storm event. Drawing No. 2.3 shows a correlation of the data for these

12 years. The average ratio of Kanaka to Norrish peak instantaneous flows is approximately 0.39. It is almost the same ratio as the ratio of drainage areas (0.41). Drawing No. 2.4 shows the approximate annual and summer flow frequency relationships for both creeks. The average ratio of Kanaka to Norrish annual peaks, at corresponding frequencies, is about 0.31.

A number of rain gauges on the north side of the Fraser Valley can be used as indicators of regional storm events. These are Haney Research Forest (El. 143 m), Alouette Lake (El. 137 m), Stave Falls (El. 55 m), Mission West Abbey (El. 221 m), and Agassiz (15 m). None of these gauges are believed to be representative of the higher elevations in the basins of interest (up to El. 1200 m). Studies show that precipitation increases for higher elevations, and increases more rapidly up valleys than up open slopes, owing to convergence and uplift (Wright, 1966; Church and Miles, 1987). Winter and summer patterns show the same features, but winter amounts of precipitation are greater. A further complication in examining response to winter storms is the augmentation of rainfall runoff by snowmelt at higher elevations.

.3 Previous Related Studies

Three previous studies can be used to assist in the estimation of the annual maximum instantaneous flow for Lagace Creek below Pattison Creek (17 km²). The results are summarized below:

	Mean Annual Instantaneous (m³/s)	25-year Instantaneous (m³/s)	200-year Instantaneous (m³/s)
Cheng 1980	(45)	92	(140)
NCH 1985	68	150	225
NCH 1985	(60)	120	180

(Values in parenthesis are inferred)

Both the studies by Cheng (1980) and Northwest Hydraulic Consultants (1985) used streamflow analysis of streams in the general area, with attention to precipitation. Given the limitations of the data base, the difference in estimates between the first two analyses is not excessive.

The final analysis in the above summary is based on a study by Northwest Hydraulic Consultants (1988) of Weaver Creek (42 km²) and its tributary Sakwi Creek (18 km²), located approximately 20 km east of Lagace Creek, and east of Norrish Creek. This study placed a reliance on the annual maximum instantaneous flow records for Norrish Creek and limited data for

Weaver Creek itself. A well-defined instantaneous flow frequency relationship for Norrish Creek was adapted to Weaver and Sakwi Creeks, adjusting for differences in drainage area according to the trend of the Creager flood envelope curves (see Appendix E). 25-year and 200-year peak instantaneous flow estimates for Norrish Creek were found to correspond to Creager C values of approximately 20 and 30. Application of the same C values to Lagace Creek below Pattison Creek results in 25-year and 200-year peak instantaneous flow estimates of 120 m³/s and 180 m³/s, roughly midway between the Cheng 1980 and NHC 1985 estimates. To set these figures in context, about 10 of the highest known C values in Canada are in the range of 40 to 45 (see Appendix E). The recurrence interval of these highest known C values is a matter of debate, but is probably at least 500 years.

.4 Other Approaches to Extreme Flow Estimates

Several other empirical approaches based on rainfall-runoff relationships have been applied as a check on the 200-year flow estimates for Lagace Creek below Pattison Creek. Rainfall values are based on AES Intensity-Duration-Frequency charts for several stations in the region. Selected 100-year rainfall data (200-year estimates were not given) are as follows:

Station	Duration hours	Intensity mm/hr	Total mm
Alouette Lake	0.5	26	13
	1	20	20
	2	17	34
Mission W. Abbey	0.5	44	22
	1	28	28
	2	19	38

The results of several approaches are summarized below:

Rational formula. The basin is assumed to be saturated, with a runoff coefficient of 1.0. The time of concentration of a basin like Pattison Creek is estimated to be in the order of one hour. A 200-year rainfall intensity of 30 mm/hr was adopted. The resulting peak instantaneous flow for a drainage area of 17 km² is approximately 140 m³/s.

Watt and Chow synthetic hydrograph (Watt et al 1989). Time to peak for Pattison Creek is 0.5 hours on the basis of the empirical formula given.

Assuming 20 mm total runoff, peak instantaneous flow is estimated as 180 m³/s.

Empirical hydrograph. Total runoff is assumed 20 mm over an effective hydrograph timebase of 2 hours, and peak flow is assumed 3 times the average flow. The estimated peak instantaneous flow is 140 m³/s.

These results range between the Cheng estimates and the NHC (1988) method based on a Creager C value of 30. It appears that they do not seriously conflict with the results obtained by transferring streamflow data and parameters from nearby basins.

.5 Flow Estimates for Specified Frequencies for Upland Tributaries to Lagace Creek

Annual maximum instantaneous flood frequency estimates for major upland tributaries, where they enter the flood plain, are shown in Table 2.2. These values were derived on the following basis:

- .1 Drainage areas were taken as 100% of mountain and hill areas to the edge of the flood plain.
- .2 200-year peak instantaneous flows were based on the Creager formula with C equal to 30. Basically, the Creager relationship is used to transfer the Norrish Creek data to other drainage areas.
- .3 Ratios of peak instantaneous flows to the mean annual peak instantaneous flow for various flow frequencies were adopted as below, based on a review of analyzed ratios for Norrish and Kanaka Creeks:

Recurrence interval in years	Ratio of peak instantaneous flow to mean annual flow
5	1.35
10	1.65
25	2.0
50	2.3
200	3.0

As the response time of these basins is a matter of hours only, ratios of instantaneous to daily maximum flow are large, and are highly sensitive to the duration of the hydrograph and to the clock time of the peak (see Appendix F); therefore, it is judged that estimated maximum daily flow would be unreliable and consequently are not quoted in Table 2.2.

It should be noted that the estimates in Table 2.2 are based on the concept of a typical flood hydrograph resulting from intense rainfall, without special surge or flood-wave phenomena associated with debris torrents, landslides, log-jam releases or similar occurrences. Such phenomena might result in transitory flood discharges of substantially greater magnitude.

.6 Analysis of Specific Storm Events

The determination of recurrence intervals for specific past storm/flood events in the Hatzic area presents some difficulties. In general terms, recurrence intervals can be estimated by frequency analysis of a particular statistic, such as average 3-day rainfall or maximum daily flow, but the recurrence interval of a total event that occurs over several days is essentially indeterminate or subjective. Likewise, the selection of rainfall or streamflow duration which defines an "event" is also subjective. The approach taken here is to present recurrence intervals for selected flow and rainfall statistics for each of seven events, based on analyses of records for selected precipitation and streamflow gauges. In addition to the six events between 1980 and 1990 as specified in the Terms of Reference, the additional event of January 1968 has been included.

Table 2.3 summarizes selected streamflow and rainfall data for the seven events and indicates estimated recurrence intervals for maximum flows (either instantaneous or daily, depending on the available data) and for a precipitation duration that appears to define the event. Precipitation data are for selected AES gauges that indicated n-day precipitations, which are reconcilable with tabulated flood runoff depths and that had available depth-duration-frequency analyses from the Ministry of Environment. In the absence of streamflow records for Lagace Creek, streamflow data for WSC gauges in Norrish and Kanaka Creeks are presented (see Table 2.1, and Drawing No. 2.2). In considering indicated recurrence intervals for streamflows, it should be noted that the validity of the published data series can be questioned because data are often missing or unreliable for the highest events (Quick, 1991).

Storm scenarios for six of the seven events (excluding January 1968) are shown in Drawing No's 2.5 through 2.10. These show daily precipitation hyetographs and streamflow hydrographs, as well as locations of all relevant gauges. Major runoff events in Norrish and Kanaka Creeks generally last two or more days, in contrast to durations of hours mentioned in connection with flood frequencies for upland tributaries. This is partly because of larger drainage areas, and because critical short-duration rainfall intensities, which

determine peak instantaneous flows from small basins, are normally contained within rainstorms of longer duration, or occur in summer thunderstorm events.

The following comments pertain to the individual events:

January 1968. Heavy precipitation occurred on 18-19 January, of about 230 mm at Alouette Lake and Stave Falls. The 2-day runoffs in Norrish and Kanaka Creeks were similar. The recurrence interval of the 2-day precipitation at Stave Falls appears to be about 40 years, but the recurrence interval of the maximum instantaneous flows is less than 10 years, presumably because rainfall intensities were not severe.

December 1980. Heavy rainfall occurred on 25 December, with a smaller amount the following day. The 2-day runoffs were comparable with 2-day precipitations. Recurrence intervals for precipitation and for maximum instantaneous flows are around 5 years.

June 1981. Precipitation occurred for over two-thirds of the month. The largest rainfalls were recorded at Stave Falls with the 3-day precipitation having a 30 year recurrence interval. In the Mission area there was a single day of moderately heavy rainfall that ranged between 50 and 70 mm which has a summer recurrence interval (May - August) of approximately 5 years. The 2-day runoff in Norrish Creek was comparable with the recurrence interval of summer maximum flows of about 3 years.

July 1983. There was a 5-day regional rainstorm from 10 July to 14 July, totalling about 190 mm in the Mission area. Total runoff in Norrish Creek for 6 days was about 315 mm. Kanaka Creek had a total runoff of about 170 mm. These figures suggest that rainfall must have been much higher at higher elevations of the Norrish basin. Summer recurrence intervals (May-August) were large with the 2-day rainfall for Pitt Polder and Mission West Abbey being 100-years and 60-year, respectively. Recurrence intervals of maximum daily flows was in the order of 25 years as summer maximum.

November 1989. Definition of the event is difficult because there was a long rainy period, and various gauges show different patterns. The 4-day precipitation at Stave Falls, 6 November to 9 November, was about 290 mm, and 3-day runoffs, 8 November to 10 November, were slightly higher which was possibly because of snowmelt. The recurrence interval of rainfall is about 20 years. Recurrence intervals of maximum daily flows appear to be in the order of 10 and 50 years for Norrish and Kanaka Creeks respectively.

February 1990. There was a 3-day precipitation, 8 February to 10 February, of about 140 mm at Stave Falls, and 115 mm at Mission West Abbey. The 2-day runoff, 10 and 11 February, was about 170 mm in Norrish Creek and

130 mm in Kanaka Creek, which possibly included some snowmelt. Recurrence intervals are about 5 years for precipitation, and about 3 years for maximum daily flows.

November 1990. There was a 3-day precipitation, 8 November to 10 November, of about 260 mm at Stave Falls and 195 mm at Mission West Abbey. The 3-day runoffs, 9 November to 11 November, were about 350 mm in Norrish Creek, and 270 mm in Kanaka Creek. The recurrence intervals for precipitation had a large range. Pitt Polder experienced a 3-day rainstorm with a recurrence interval of 15 years while this was a 50-year event for Mission West Abbey. The 2-day rainfall at Mission West Abbey was a 100-year event, though this value should be viewed with caution. The recurrence interval for maximum daily flow in Norrish and Kanaka Creeks for a 3-day rainfall was 15-years.

The following conclusions are indicated from these analyses:

- In severe events, runoff is generally made up from a high proportion of reported rainfall, and can considerably exceed reported rainfall in some cases because of snowmelt, or greater precipitation at higher elevations.
- In Norrish and Kanaka Creeks, maximum flow tends to lag at least one day behind maximum precipitation.
- In some events, estimated recurrence intervals for maximum flows are similar to those for total precipitation, but in other events the precipitation recurrence interval is greater. This may be a result of the sensitivity of the maximum flow to the maximum rainfall intensities, which have not been examined. For example, the Norrish Creek peak instantaneous flow of January 1968 corresponds to a rainfall intensity of about 13 mm/hour, based on the Rational Formula with a runoff coefficient of 0.8. The maximum reported daily average rainfall had an intensity of about 6 mm/hour which for a duration of 24 hours would have a larger recurrence interval than the short duration intensity associated with the peak flow.

.7 Flow Estimates for Specified Frequencies for Points Along Lagace Creek

Table 2.4 lists the 200-year, 50-year and 25-year instantaneous and daily flood flows at four road crossing points along the Lagace Creek system. These flows were derived on the following basis:

- .1 Effective drainage areas are as 100% of mountain and hill areas plus 50% of flat valley areas.
- .2 200-year instantaneous peaks are based on the Creager formula with C equal to 30, and applied to the effective area as defined above.
- .3 Flow frequency ratios for instantaneous flows are assumed to be as stated in Section 2.1.5.
- .4 Instantaneous/daily ratios are judged to be approximately 4 at Seux Road, 3 at Farms Road, and 2 at Dale Road.

The estimates for Farms Road and Dale Road are based on the assumption that spill flow and spill storage are negligible, which implies that the creek is dyked or dredged to retain all flows. Under present basin conditions, actual flow peaks would be reduced significantly by overspill and storage.

The maximum daily estimates indicated for Farms Road in Table 2.4 are only about 60% of the corresponding estimates in the previous NHC (1985) study. However, the 25-year daily estimate for Dale Road (gross drainage area 44 km²) is about 40% greater than the estimated 25-year maximum daily flow for the gauged Kanaka Creek (area 48 km²), and is also higher than the runoff per unit area for Norrish Creek. However, it is concluded that the values in Table 2.4 are sufficiently high and provide conservative maximum flow estimates which are recommended for the design of flood control works.

.8 Impact of Logging on Watershed Hydrology

Before discussing the impact of logging on flows down the Pattison/Lagace Creeks, a brief discussion on the hydrologic processes is presented to illustrate the changes that logging can impose upon the flow and storage of water within the watershed. In this discussion, the emphasis will be placed on the role of forest cover on the magnitude of peak flows since this study is concerned with mitigation of flood damages.

The process that leads to streamflow from rainfall is somewhat complex, but the following concept is generally agreed upon after research by Horton (1933), Hewlett and Hibbert (1967), Kirby and Chorley (1967), Musgrove and Holton (1964) and Hewlett (1982). Within humid areas the type of storm runoff is based on the concept of variable source area (see Drawing No. 2.11). During storms, rainfall causes a thin layer of soil on some parts of the basin to saturate upwards. Then the rainfall cannot infiltrate further into the saturated soil and runs overland as the saturation overland flow. The overland flow velocities are usually much higher than the subsurface storm

flow (throughflow) as shown in Drawing No. 2.12. Hewlett and Hibbert (1967) also point out that the saturated area contributing to runoff expands outward from the streams as streamflow increases, and this indicates that the entire watershed is not contributing to saturation overland flow (see Drawing No. 2.11). For rainstorms extending to 4 or 5 days, however, the entire watershed may be contributing to saturated overland flow. Field measurements have also indicated that subsurface storm flow reaches channels slowly (11 m/day for example) and does not contribute significantly to the magnitude of the peak discharge (Dunne and Leopold, 1978). The saturation overland flow can be 100 times that of the subsurface throughflow (Dunne and Leopold, 1978). Therefore, in considering high flows down the Pattison/Lagace system it is the saturation overland flow that predominates; subsurface storm flows occur slowly and do not significantly affect the values of the peak discharge for which flood protection measures are designed.

The impact of logging on the high flows has received considerable attention especially from residents adjacent to flooding rivers, from environmentalists, and from watershed/forest managers. In assessing the impact of logging, the water balance equation serves as a framework (Cheng, 1977):

$$Q = P - \Delta S - E$$

where: Q = streamflow;
P = gross precipitation just above the forest canopy;
 ΔS = change in watershed water storage, including soil

water storage, groundwater storage and surface water

storage; and

E = evapotranspiration which is all the vapour losses from the surface of plants, soils, water and snowpack.

Beginning with precipitation, it is generally accepted that a forest canopy intercepts a portion of the gross precipitation resulting in a smaller net precipitation reaching the ground. Of significance in assessing logging impact is the amount of rain being intercepted. Cheng (1977) concludes that

"net precipitation (rainfall) on the Coquitlam River watershed under the forest would average approximately 95 percent of the gross precipitation in winter and 75 percent in summer".

Cheng (1977) bases his conclusion on studies by Rothacher (1963) in western Oregon and by Cartwright (1970) in the Seymour River watershed. The reason for the greater interception during the summer months is that rainfall

amounts and intensities are low and storm events are separated by warm periods allowing more complete evaporation of intercepted rainfall. Therefore, as most of the high flows in the Pattison/Lagace system occur during the winter months (October - April) it is probable that logging will not significantly increase the amount of rainfall reaching the ground surface.

During winter, the canopy can intercept snow and reduce flow magnitudes (Goodell, 1969; Anderson, 1963), but these effects are only significant at elevations over 700 m where snowpack can be heavy. A number of factors such as logging pattern, prevailing winds, temperature changes, etc must be considered when assessing the impact of logging on snowmelt runoff. Quoting Beaudry and Golding (1985):

"snowmelt runoff rates were greater from the forest than from clear-cuts when the storm began as snow and then changed to rain. The snow in the canopy melted at a rate faster than that in the clear-cut because of greater energy inputs at canopy level...for rain storms when there was no snow in the canopy, snowmelt runoff rates were greater from clearcuts."

The storage and evapotranspiration components of the water balance equation will now be treated. Logging generally eliminates rainfall interception and reduces transpiration because tree roots are no longer absorbing water. Evaporation from ground surface will be slightly increased, but the net effect will be reduced evapotranspiration. The result is that more water ends up as storage and

"for a storm of a given intensity and duration, the recharge phase for the clearcut slope was therefore much shorter than for the forested one", (Cheng, 1977)

Cheng (1977), and Gessel and Cole (1965) all report annual water yield increases from logged plots and attribute this to the reduction of interception and transpiration losses. However, their studies did not acknowledge the concept of variable source area and it is probable that another factor resulting in increased water yield is that logging could cause increases in source area resulting in a larger proportion of a total watershed undergoing saturation overland flow instead of subsurface storm flow. Logging roads could also result in larger source areas.

Therefore, the annual water yield from a logged watershed should increase and this conclusion has been supported by many studies (Hibbert, 1967); the effect of logging on the magnitude of peak flows is less clear.

The impact of logging on peak flow magnitudes depends upon many factors

as noted by Hamilton and King (1983):

- the area of the logging compared to the total watershed area;
- the amount of biomass removed (including how much slash remains on the area);
- the product removal methods;
- the timing with respect to wet and dry seasons;
- the soil, geologic conditions, and topography;
- the extent, nature, and usage of roads, skid trails and landings;
- the methods of slash disposal and site preparation;
- the promptness with which regeneration occurs (or reforestation is carried out);
- the presence or absence of adequate riparian buffer strips; and the nature of climatic events following disturbance.

Because the situation is complex and because there is no pre-logging nor post-logging flow data along Pattison/Lagace Creeks, an examination has been made of the evidence gathered from research studies in watersheds along the Western Coast of the Pacific as well as from other comparable regions.

.9 Assessment of Research on Flood Flows From Logged Areas

Jamieson Creek, an experimental watershed in south coastal British Columbia, was established in 1969 to expand the knowledge of hydrology in this region, and to assess the effects of timber harvest on stream flow. Golding (1987) reported on the effects of clear cutting 19.2% of the basin:

"For the period 1978-86, winter storm peaks showed a treatment-induced increase of a maximum 13.5% with no indication that the increases were limited to a particular storm-size class. Summer storm peaks showed no treatment-induced increase. Winter streamflow peaks were highly correlated with total snowmelt runoff from lysimeters in the forest and in clear-cut, but less well correlated with peak snowmelt runoff rates."

Other long term studies on Carnation Creek indicate that peak discharges of storms were not significantly increased by logging (Hetherington, 1982; 1983). Hetherington (1987) summarized the study results as follows:

"Changes in streamflow after logging have occurred in a small 12 ha tributary (H Watershed) in clearcut 864-4, but have not been detected in the main stream....on the basis of data analysis to date, logging impacts on streamflow generated by rainfall alone have been minimal in the main Carnation Creek".

Research of logging on small watersheds, however, has indicated both

increases and decreases in peak flows during rainstorms. Cheng et al (1975) indicates 22% decrease in peak flow for 71% clear-cut watershed in Haney, B.C. while Heatherington (1982) shows a 20% increase in 'H' Creek tributary of Carnation Creek which was 90% clear cut and no increase in creek flow detected in the overall Carnation Creek where 40% watershed was clear-cut.

Another study along the Pacific Coast of the effects of logging in Casper Creek, California by Wright et al (1990) yielded the following conclusion:

"Road building on 5% of the watershed and selectively removing 67% of timber volume by tractor logging significantly increased runoff volumes and peak flows only in the very smallest storms (those with peaks less than 566 L/s). The large flows in this study were not increased even though over 15% of the watershed was compacted in skid trails, landings, and roads. Although a decrease in lag time showed that the storm hydrographs were shifted forward in time, their shapes were unchanged".

Hewlett (1982) reviewed the research during the decade prior to 1982 and concluded that:

"forest operations do not seriously increase flood flows in major streams. This conclusion is extended to the major forest regions of the world. However, the conclusion does not extend to erosional effects of forest operations, or to the effects of alternate types of land use (cultivation, urbanization or grazing) on flood water discharges from source areas."

The above conclusion was also presented some ten years earlier by Lull and Reinhart (1972) and data gathered over the next decade has reinforced the findings that deforestation does not cause higher flood peaks. A thorough review of the topic has also been undertaken by Hamilton and King (1982) who concluded that:

"Flooding may be increased close to the area cut, but as water is routed down a major river basin, this effect is rather quickly reduced to insignificance amid other processes of paramount importance, such as the nature and intensity of the precipitation, the direction it moves across the basin, and the size and morphometry of the basin. Hewlett (1982) has recently examined the evidence worldwide from forest watershed research and reported that there was no cause-effect relationship between forest cutting in the headwaters and floods in the lower basin. Even if a whole basin were under a forest harvesting regime, normally it would not be logged off all in one year. Those portions that are logged rather quickly return to a prelogging hydrologic regime as the forest regenerates and full canopy is restored, even though it is young growth. Moreover, a substantial part of this stormflow/peakflow effect

is due to poorly located and designed roads, skid trails, and log landings, all of which speed water off-site. Thus, proper conservation logging can reduce any small effects on upstream flooding. Floods occur due to too much precipitation falling in too short a time or over too long a time (prolonged), beyond the capacity of the soil mantle to store it and the stream channel to handle it."

It has been acknowledged, however, the peak flows can be increased if a substantial percent of a watershed is logged. From review of research by Heatherington (1987), who found that logging of 40 percent of the watershed had not altered storm runoff, and from Wright et al (1990) who found that removal of 60 percent of the canopy had no effect, it is the opinion that logging up to 50 percent of a watershed will not influence storm runoff peaks.

From these various studies, it may be concluded that the logging has not caused excessive increases in the magnitude of peak flows in the lower river system. However, during logging, it appears that surface erosion of soil had increased and that landslides occurred which significantly increased sediment loads down the river system. More details on sediment loads are presented in Section 3.

.10 Assessment of Logging and Impacts Within Lagace Creek Watershed

The extent of logging within the Pattison Creek watershed, as well as within the Hatzic Valley, has been relatively small. Of the 6.0 km² drainage area of Pattison Creek, 1.53 km² (153 hectares) were logged between 1952 and 1972. The logged area represents 26 percent of the watershed. Most of this area has experienced regrowth, and there has been no significant logging since 1972 as shown by Drawing No's 2.13 and 2.14.

Along the eastern slopes of Hatzic Valley, south of Pattison Creek, a total of 105.6 hectares were cut between 1964 and 1991. North of Pattison Creek, on the east side of the valley (Lagace Creek basin), a total of 100 hectares were cut between 1965 and 1971.

On the west side of Hatzic Valley, 19.6 hectares have been logged within the Stave Supply Block. While not recorded, the date of the logging is thought to be prior to 1955. The logging controlled by the City of Mission is uncertain, but 1990 air photos show that only one block of about 21 hectares has been logged.

The actual impact of logging upon peak flows within the Hatzic Valley cannot be quantified because there are no discharge gauging stations within the basin; however, since only 4.4 percent of the total watershed has been logged,

it is judged that logging has not increased the magnitude of peak flows along the lower reaches of the floodplain.

Locally, however, there is evidence that increased production of sediment has raised bed levels which in November 1990 resulted in an avulsion downstream from the Farms Road Bridge (northern). There is also evidence (Drainage Inventory Photos - office data) of raised bed levels along both the North and South Hereford Creeks which resulted in local flooding in November 1990. The logging within the watershed of South Hereford creek appears to be resulting in erosion of materials which are transported and deposited near Sylvester Road. This deposition has caused overtopping of the road and local flooding.

.11 Development of the Upper Watershed

From discussions with local officials and from review of the "McConnell Creek-Hatzic Prairie Official Community Plan" by DARD there are no immediate plans for upland development in the catchment area. However, should any development be proposed, then DARD should ensure that consideration be given to the impact of this development on peak flow estimates or the time averaged flow estimates contained in the Wigmore (1983) report. The development plan should set out stormwater management measures that result in minimal changes to downstream conditions.

.12 Effect of Global Warming on Flood Magnitude

There are few studies on flood trends in this specific area, but the Westwater Research Centre (1991) has studied the Fraser River Basin and concluded that:

"the empirical approach indicates that the future streamflow should decrease as a result of global warming. The water balance approach, on the other hand, indicates that streamflow in the Fraser River at Hope may increase due to increases in precipitation in the mountainous portions of its drainage basin. This lack of agreement highlights the uncertainties associated with making projections of changes in water supply associated with global warming."

Studies in other parts of Canada, such as the Great Lakes area (Sanderson, 1992) report that:

"recent research on the impact of climate change on Great Lakes levels...indicate that with the higher temperatures projected by the scenarios, average lake levels may be expected to drop as much as 2

m...and decreased runoff from the land areas of the basin."

The predications are based on historical records and on assumptions as to future cause and effect relationships. For the Lower Fraser region, the historical records have not been analyzed to ascertain patterns of high and low runoff; neither B.C. Environment nor AES has conducted studies. Therefore, at this time there is no basis to predict whether floods in Hatzic Prairie will become larger and more frequent as a result of climate changes.

2.2 FLOOD ROUTING

.1 Introduction

Lagace Creek originates 15 km northeast of Mission in the Douglas Forest. Running through the central lowlands, Lagace Creek collects the flow from several mountain streams and discharges into Hatzic Slough which in turn discharges into Hatzic Lake (Drawing No 2.1). Approximately 95% of the catchment drains into Hatzic Lake which discharges to the Lower Hatzic Slough under a bridge at Lougheed Highway and through three culverts under the CPR mainline tracks (two 3.05 m diameter and one 2.44 m diameter CSP culverts each 33.8 m in length). The Lower Hatzic Slough flows south to its confluence with the Fraser River. Near to the Fraser River, all of the runoff must be discharged either through four existing floodboxes (each 1.83 m x 1.83 m and 45.7 m in length), or by an existing pump station (capacity of 6.06 m³/s against peak Fraser River flows).

This section begins by describing the watershed and how it functions during flood events, estimates inflows to the Hatzic basin for six storms (December 1980, June 1981, July 1983, November 1989, February 1990, and November 1990), determines the effect potential improvement structures would have on flood levels and durations, analyses a 5-day 10-year storm, and determines the effect of lake dredging.

.2 Watershed Description and Function

Drawing No's 2.5 through 2.10 show precipitation and streamflow data for the six storms. Also shown are the water levels of the Fraser River and lower Hatzic Slough at the pumphouse. Table 2.5 lists the recorded peak slough levels at the pumphouse. The river level at the pumphouse was estimated by adding to the river level at Mission the expected difference between the Mission gauge and the pumphouse. Previous studies by NHC have assigned a hydraulic gradient of 0.00006 for the river reach near Mission. This represents an increase of about 0.3 m for a 5.0 km distance between the Mission gauge and the pumphouse.

During major regional storms, the watershed can be broken into a number of sections that function and respond differently during flood events. The following identifies these sections and describes their different functions and responses:

Mountainous Regions. Rainfall in the surrounding mountains produce high energy steep gradient flows (ie. Pattison Creek), which quickly reach the central lowlands and Lagace Creek. The flow response of these smaller regions to a heavy rainfall is a matter of hours only.

Hatzic Slough/Prairie. The runoff from the highlands is collected in the lowlands by Lagace Creek and discharged into Hatzic Slough. The slough acts as a sump providing storage¹ for the runoff from the highlands.

The potential storage in Lagace Creek and Hatzic Slough is relatively small. If the storm runoff is large and the required storage is greater than the potential storage, the creek will overspill its banks and flood onto the adjacent prairie. In some cases, the creek can also break through its banks or dykes and flood onto the prairie.

For Hatzic Prairie, the agricultural area that is most affected by flooding is a 500 ha depression about 2.0 km north of Hatzic Lake centred on the Farms Road and Dale Road area. This area begins to flood at elevation 3.0 m GSC and is difficult to drain and prevent from flooding for two reasons: the area is located on the far side of the lake, 7.0 km upstream from the pump station; and, the pumps normally operate for levels above 2.41 m GSC. Even though the pumps could be operating, their immediate effect is not felt on the prairies.

Hatzic Lake - Hatzic Lake provides the greatest storage potential in the system (Drawing No 2.15). In combination with Hatzic Slough and Hatzic Prairie, the total storage provided by the two sumps effectively delays the initial runoff from reaching the outlet works at the pumphouse. Once the sumps are full, the energy gradient available to drive the water from the prairie to the pumphouse, a distance of about 7.0 km, is about 0.00004, or

¹Storage is the difference between the volume of water entering a region and the volume of water leaving the same region over a given time interval. Storage is generally positive at the beginning of a rainstorm and water levels increase. When water levels begin to fall, storage is negative, since more water is leaving the region than is entering.

0.4 m per 10,000 m. This is quite different from the high energy steep gradient flows that enter Lagace Creek and the prairie. For example on Pattison Creek the maximum gradients are about 0.75, or 0.75 m per m.

Culverts at the CPR mainline. Flow exiting Hatzic Lake must pass through three culverts at the CPR mainline adjacent to the Lougheed highway bridge. The capacity of the culverts is sufficient to meet the combined discharge of the pumps and floodboxes; however, during previous floods, the culverts have on occasions become partially blocked with debris (December 1980, November 1980 and November 1990; Photo 2.1). A noticeable trend developed in the water level recorded at the pumphouse. When the culvert blockage was noted in the pumplogs; the gauge reading at the pumphouse suddenly decreased. This lends support to the conclusion that the flow through the culverts was reduced, possibly resulting in higher lake levels upstream of the culverts, lower water levels downstream of the culverts, and a slower overall draining of the lake and prairie regions.

Pumps and Floodboxes. The pumps and floodboxes provide the only outlet for the runoff of the Hatzic catchment basin to the Fraser River. Generally, the pumps operate when the level of the lake is greater than 2.41 m GSC and the floodboxes are open only when the level of the slough at the pumphouse is higher than the Fraser River level. During the Fraser River freshet when the floodboxes are closed by the high river level, a major regional rainstorm can produce serious long-term flooding (June 1981 and July 1983). During these periods the lake and prairie are solely dependent on the pumps for flood protection which are not adequate to remove the large volumes of inflow to the prairie and lake. During severe regional winter storms, the Fraser River level quickly rises and closes the floodboxes (November 1989, November 1990). Even smaller winter storms can produce severe flooding if combined with a high Fraser River level that lasts for several days. December 1980 provides a good example: a 5-year event (see Table 2.3) produced the second highest flood level recorded in the past 50 years because the high Fraser River level kept the floodboxes closed for over 3 days (see Dwg. No. 2.5), the longest duration of any of the three winter storms analyzed. Flood relief during freshet periods is only achieved by pumping. However, during winter months, the floodboxes will normally open within two or three days, increasing the outflow and providing some additional flood relief. Winter storms have provided the two highest flood levels recorded at the pump station since it began operating in 1948. The December 1980 and November 1990 floods reach levels of 3.993 m GSC (13.2 ft) and 4.115 m GSC (13.6 ft) at the pumphouse, respectively.

Two approaches were used in the routing analysis: backward routing and forward routing. Both methods were based on the Muskingum Method and

used the recorded Fraser River levels at Mission, adjusted to the Hatzic pumphouse, as the downstream boundary condition. The backward routing analysis used the lake levels recorded at the pumphouse to determine the outflow and then back-calculated the necessary inflow, routing it through the prairie and lake storage; however, because this approach relied on a single level recorded daily at the pumphouse and the mean daily Fraser River level, the resulting calculations were not refined enough to reflect rapid changes in the system due to Fraser River levels which are tidal and can vary more than over 1.0 m each day. Forward routing was used to overcome this deficiency as it allowed the system to be routed using an hourly time increment. The forward routing analysis assumed an inflow hydrograph, routed it through storage, and then calculated water levels throughout the system. The slough levels at the pumphouse were compared to the recorded daily value and the inflow hydrograph was adjusted until a reasonable match was achieved. Once an inflow hydrograph for each storm event was established, the effect potential improvement structures and dredging have on the flooding were determined.

Hydraulic capacities were calculated for the floodboxes and culverts as a function of upstream and downstream water levels, assuming no blockage by debris. However, this assumption is not wholly correct since accumulation of debris and rubbish has been observed (Photo 2.1). This phenomenon is considered to be an important source of unresolved discrepancies in the routing results.

It was assumed that the pumps operated at maximum capacity throughout the period of operation. While this is a generalization, the routing analysis is not very sensitive to this factor because of the relatively small contribution of the pumps to total outflow.

Drawing No. 2.15 shows the storage curves that were used in the analysis. Each curve shows the total volume for each of Hatzic Lake, Hatzic Prairie, and Chilqua Slough. It was derived using contour maps and results from the recent lake survey (Appendix G), and encompasses the total volume from elevation 0.5 m to 5.0 m (GSC). For the backward routing analysis, it was assumed that during flood conditions the Hatzic system acts as a single unit, as defined by the total storage curve. For the forward routing analysis, the Hatzic system was broken into three sections.

.3 Backward Routing

Using the backward routing computation and the known conditions at the pumphouse (Fraser River levels and Lower Hatzic Slough levels), the inflow conditions to the basin were estimated as shown in Table 2.6.

Comparing the flood runoffs of Table 2.6 with the rainfalls and flood runoffs of Table 2.3, it can be seen that for four of the six events - December 1980, July 1983, November 1989 and November 1990 - the two sets of values are more or less reconcilable; however, in three of these events the CPR culverts are believed to have been partially blocked by debris. For the other two events - June 1981 and February 1990 - the total runoffs appear to be grossly excessive. The large February 1990 routed runoff may be the result of using river levels recorded at the pumphouse instead of levels recorded at the Mission gauge which was inoperative; the river level recorded at the pumphouse are not necessarily the mean daily levels which were used for the other storms. The maximum daily flows do appear to be more or less reconcilable with recorded maximum daily flows in Norrish and Kanaka Creeks for all six events.

Besides uncertainties over culvert blockage and actual pumping rates, data and computational limitations may be responsible for some of the discrepancies. Initially, the computations were based on daily time steps, and the reported daily lake levels were assumed to have been recorded at midday. However, Fraser River levels, which affect outflows significantly, can vary greatly over a day. Routing was attempted using a half-day increment; however, half-day estimates of both the lake levels and river levels were difficult to make and the results were too difficult to interpret.

.4 Backward Routing Summary

Several conclusions can be drawn from the backward routing analysis:

- The analysis gives total runoff estimates more or less compatible with gauged runoff from adjacent basins.
- The analysis appears to give reasonable results with respect to maximum daily flows, providing a basis from which to develop the inflows using forward routing.
- Proper modelling of floodbox outflows from Hatzic lake to the Fraser River requires more frequent lake level data, preferably at hourly intervals.
- Further refinement of the backward routing analysis using available data would not provide better results.

.5 Forward Routing

The Hatzic system was broken into three sections: Hatzic Prairie above the entrance of Hatzic Slough to Hatzic Lake; Hatzic Lake above the CPR culverts at Lougheed Highway; and, the Lower Hatzic Slough between the CPR culverts and the pumphouse. For each section, routing utilized the individual storage curves (see Drawing No 2.15). The routing analysis used an assumed inflow hydrograph to the Hatzic system, and then calculated the water levels that such an inflow would produce. The predicted levels at the pumphouse were then compared to the observed levels and the hydrograph was adjusted accordingly until an acceptable inflow hydrograph was produced. Initially, rainfall records at Mission West Abbey were used to estimate the inflow hydrograph; however, this was found to be unreliable. A more direct approach was then adopted, using simplified inflow hydrographs based on the study by Wigmore (1983), and on hourly streamflow records on Kanaka and Norrish Creeks. The rising leg and peak of the inflow were adjusted so that the predicted levels at the pumphouse were near to the observed levels. For most of the past events, these levels are the only ones that can be directly checked. Predicted water levels in Hatzic Lake and Prairie can be checked against surveyed high water marks only for the November 1990 flood.

Water level recorders were installed as part of the present study at Dale Road Bridge (Lagace Creek), Shook Road Bridge (Hatzic Lake), and at the Hatzic Pumphouse. Levels recorded during a small storm the end of August 1991 (Drawing No. 2.16) were then used as a check on output from the computational model.

.6 Estimated inflow hydrographs - previous storms

Inflow hydrographs were estimated for four storms: June 1981, July 1983, November 1989, and November 1990. Water levels at the pumphouse for these events are shown in Drawing No's 2.6, 2.7, 2.8, and 2.10. At the time the model was developed, pumphouse records were not available for the December 1980 flood. For the February 1990 flood, hourly Fraser River levels were unavailable at Mission because of an inoperative water level gauge.

Drawing No's 2.17 through 2.20 show the routing results for existing conditions for each of the four storms. The predicted water levels generally follow the same water level trends as recorded during August 1991 (Drawing No. 2.16). The June 1981 and July 1983 water levels agree with the observed level at the pumphouse. The rising leg for both November storms also agree with the observed levels; however, there is a noticeable difference of up to 1.0 m on the falling legs of the November storms. It is believed that recorded

water levels at the pumphouse, to which the routed flows were matched were lowered by blockage at the upstream CPR culverts by significant quantities of debris (Photo 2.1).

Table 2.7 shows the computed peak inflows and the computed runoff. The total runoff values in Table 2.7 are smaller than the total runoff values for Kanaka and Norrish Creeks for the same storm events, especially for the November 1989 and November 1990 storms. Even though the runoff from the Hatzic basin is considered comparable with those of Norrish and Kanaka Creeks, the results are acceptable given the variability between the recurrence intervals for runoff and precipitation for the four storms (see Table 2.3). Also, the model does not account for blockages, assuming that the flow through the Hatzic system is unimpeded. For both November storms, debris was observed to accumulate at the CPR culverts between the 11th and 14th. The flows and rising water levels prior to the 11th are correct. After the debris build-up of the 11th, the modelled flows through the culverts and floodboxes would be higher than the actual flows, and the water levels upstream of the culverts would be higher than the modelled levels. The difference between the observed and calculated pumphouse levels is in the order of 0.5 to 1.0 m (Drawing No's 2.19 and 2.20). The headloss in the small pumphouse approach channel is about 0.1 m for a discharge of 30 m³/s and a water level of 3.323 m GSC (11 ft). Because this is not sufficient to account for the difference between the observed and calculated pumphouse levels, it was concluded that a blockage of the CPR culverts did occur.

The highest flooding occurred in November 1990. A survey was made of available water levels marks in Hatzic Lake and Prairie for the November 1990 flood (see Drawing No. 2.10). Photographs taken by residents at the north end of Hatzic Lake on Sward Road indicated a water level of 4.32 m GSC, although this is not necessarily the high water since no date or time was recorded with the photograph. At the south end of the lake at the Everglades, a level of 4.25 m GSC was recorded, although again no date or time was recorded. The model calculated a lake level of 4.17 m GSC, which is considered to be in strong agreement. On Hatzic Prairie near the second Farms Road Bridge, a high water mark of 4.26 m GSC was surveyed and a level of 4.50 m GSC was generated by the model. Several additional flood levels have been included in the drawings. These represent the water level above which flooding will occur at the Riverside Rd. Subdivision (4.04 m GSC), the Benbow St. Subdivision (3.85 m GSC), the Everglades (3.35 m GSC), and on Hatzic Prairie (3.00 m GSC). These levels are useful in establishing the number of days each of these regions would experience flooding.

Table 2.8 lists the number of computed days each location was flooded during each storm, based upon the results of the modelling. Hatzic Prairie and the Everglades experienced flooding during each of the four events; the longest flooding occurred during the July 1983 flood and lasted longer than 20 days. However, the actual duration of flooding may be longer than the computed duration owing to the CPR blockage.

Of the storms studied, the summer storms produced the longest duration floods. During this period, spring runoff kept the Fraser River high, preventing any floodbox relief. In contrast, during the winter storms, the Fraser River level quickly rose and fell, allowing the floodboxes to operate on the falling leg of the flood, thus reducing the duration of the flood.

The inflow hydrographs, although approximate, cannot be substantially improved given the present scarcity of hard data in the Hatzic basin. Flows in Lagace Creek are ungauged; water levels at the pumphouse were recorded once each day but the time was not recorded (which is significant since the Fraser River is tidal at the pumphouse); and, there are no long-term data showing the change in water levels from Hatzic Prairie to the Fraser River. The first steps were taken to improve this situation by the installation of the water level recorders at Dale Road Bridge, Shook Road Bridge, and at the pumphouse. With these recorders, enough data will become available to enable the system to be back routed on an hourly basis to generate inflow hydrographs.

Model-generated water levels in Hatzic Prairie, Hatzic Lake, and at the pumphouse show a pattern of rapid rise in Lagace Creek with a slower rise in Hatzic Lake and at the pumphouse. Water levels for an event of August 1991, recorded by the new level recorders, confirm the trend.

.7 Effect of potential improvement works

Potential additions to the Hatzic Lake system include the following:

- .1 Four additional floodboxes at the pumphouses, each 1.829 m by 1.829 m (6 ft by 6 ft), doubling the number from four to eight.
- .2 New pumps with an additional capacity of 8.34 m³/s (designed to handle the 10-year 2-day summer storm).
- .3 Two additional culverts at the CPR crossing, each 3.0 m (10 ft) in diameter.

These changes were appraised using the model and three different scenarios were analyzed:

- .1 Existing conditions with additional pumps.
- .2 Existing conditions with additional floodboxes and additional CPR culverts.
- .3 Existing conditions with additional floodboxes, additional CPR culverts, and additional pumps.

For each scenario, the inflow hydrographs determined in the previous section were used. The pump operation was the same as in the existing condition; however, for the June 1981 and the July 1983 floods, the additional pumps were allowed to operate down to 1.8 m GSC.

This section determines and assesses the effect additional pumps, culverts and floodboxes have on the water levels and the duration of flooding. The comparison between the existing and proposed conditions is then based on a system that has no blockages at the CPR culverts.

The results for each of the following scenarios are shown in Drawing No's 2.17 through 2.20 and the reduction in flood durations are given in Table 2.8:

Scenario 1: Existing conditions with additional pumps

The June 1981 flood event experienced the most dramatic improvement. The additional pump capacity effectively eliminated any significant flooding.

For July 1983, the additional pump capacity significantly reduced the duration of flooding on Hatzic Prairie and the Everglades, from greater than 20 days to 10.5 days and 6 days, respectively. Flooding at Benbow St. Subdivision was eliminated. A further improvement of up to 1 day was achieved when the additional pumps were allowed to operate down to 1.8 m GSC. The largest reduction was achieved when the initial lake level was assumed to be near 1.8 m GSC, since the additional pumps could theoretically maintain this level prior to the storm inflows. With this condition, flooding on Hatzic Prairie was eliminated for June 1981 and was reduced from greater than 20 days to 8 days for July 1983.

For both the November floods, the additional pump capacity reduced the peak levels in the lake and at the pumphouse, but had no effect on the peak levels on Hatzic Prairie.

The durations of the November floods were not as dramatically reduced by pumping as the June 1981 and July 1983 storms. For the November storms, the reduction in duration was achieved primarily on the falling leg of the flood, and was influenced by the amount the peak level reduced by pumping. This was especially important for levels at Hatzic lake and the pumphouse.

The results for the two November storms indicate that the water levels at Dale Road are difficult to control using the pumphouse. The water level of Lagace Creek will always respond faster than the lake to runoff and the hydraulic gradient between the prairie and the pumphouse, which ranges between 0.00002 and 0.00005, is too small to effectively drive water from the prairie to the pumphouse.

The projected power consumption costs for combined pumping are discussed in Section 4 (see Table 4.1). Generally, with the additional pumps the time which the pump stations are running is reduced; however, because of the increase in the number of pumps, the total pump hours and pump operating costs increase. For the two summer storms, pump costs increase 45%. For the two winter storms, pump costs increase 140%. The effect of the additional pumps on the flooding associated with several storms between 1980 and 1991 is also shown on Table 4.1

Scenario 2: Existing conditions with additional floodboxes and additional CPR culverts

It is expected that additional floodboxes would require a greater capacity at the CPR culverts, and therefore additional culverts were included in the model. June 1981 and July 1983 were not analyzed for this scenario because the floodboxes played no role in flood relief. For the November floods, the additional floodboxes and culverts did not have any significant effect on the initial rising water levels and peak levels. During this period, the Fraser level was higher than the pumphouse level, effectively eliminating any benefits provided by additional floodboxes.

On the falling leg, a slight difference exists between the November floods for Scenario 1 and 2. For November 1989, the water levels for Scenario 1 and 2 are the same three days after Hatzic Lake peaked. For November 1990, the water levels for Scenario 2 are always greater than for Scenario 1. This difference was caused by two factors: the Fraser River experienced two peaks during November 1990, reducing the benefits of additional floodboxes; and, the total inflow volume for November 1990 was 25% greater than for November 1989.

The duration of flooding, as compared to existing conditions, was most improved on Hatzic Prairie and at the Everglades. For November 1989 the duration on Hatzic Prairie was reduced by 1.5 days, from 7 days to 5.5 days.

The same reduction was realized for November 1990, reducing the duration from 12 days to 10.5 days.

Scenario 3: Existing conditions with additional floodboxes, additional CPR culverts, and additional pumps

Scenario 3 produces the best results, however the improvement over Scenario 1 is quite small. The primary reason for this is that for both scenarios, only the pumps are operating during the initial rising stage of the flood - the floodboxes were shut by the higher Fraser level. On the falling leg, the additional floodboxes and culverts do provide a greater outflow, but because the peaks were originally reduced by the same amount, the difference between the flood durations is not significantly improved (Table 2.8).

.8 Forward Routing of the 10-year 5-day Event

The expected total precipitation for a 10-year 5-day rainfall event based on long-term rainfall records at Pitt Polder, Mission West Abbey, and Stave Falls is about 237 mm (Table 2.9). Assuming a curve number of 88 for an antecedent moisture condition III (Wigmore, 1983), the direct runoff will be 205 mm. This is equivalent to a total runoff volume of $22.8(10^6) \text{ m}^3$ over 5 days.

A review of previous storms isolated a 5-day rainfall event in December 1979 as having an approximate 10-year recurrence interval (Drawing No. 2.21; Table 2.10). Essentially, two 5-day totals can be used, from the 12th to 16th and from the 13th to 17th. At Mission West Abbey, the first 5-day period is a 10-year event, whereas at Pitt Polder and Stave Falls the second 5-day period is a 10-year event. In either case, the Fraser River during this period is accepted as being a 10-year flow. The peak stage reached by the river at the pumphouse was slightly greater than 3.0 m GSC which was about 1.5 m less than the stage reached during both the November storms (1989 and 1990).

The December 1979 event was forward routed using the same procedure as previously described. Drawing No. 2.22 shows the results for the existing conditions and with potential improvement works. The inflow volume to the system for the existing conditions was computed to be approximately $21.5(10^6) \text{ m}^3$, slightly less than the total runoff volume predicted by the antecedent moisture computation.

The peak level recorded at the pumphouse occurred on 18 December, one day after the entire storm event. Although this could be caused by the slow response of the Fraser River to different rainfall conditions further upstream, the response of the lake to the rainfall is difficult to reconcile, especially with the single day of no rainfall occurring in the middle of the storm.

For existing conditions, the model predicts that Hatzic Prairie and the Everglades would have been flooded for 9.5 and 5 days, respectively. The addition of pumps reduced these durations to 8 and 2.5 days. As in the previous scenarios, pumping does not affect initial levels at Dale Road bridge; the benefit is realized on the falling leg of the flood. The addition of floodboxes and culverts, with or without additional pumps, produced only slight improvements.

.9 Effect of Lake Dredging

Residents have suggested that Hatzic Lake should be dredged to achieve the following goals:

- .1 To increase the volume of live storage thus providing a reservoir for large inflows during storm events.
- .2 To increase the depth of the lake for recreational users (i.e. waterskiing sailing, etc.) between May and September.
- .3 To provide a sediment reservoir for future sedimentation of the lake.
- .4 To reduce the formation of dust storms caused by large areas of the lake bottom being exposed during windy winter days.

The lake delta deposit extends lakeward as a submerged levee system, which will eventually grow until the Hatzic Slough channel extends through the lake to the railway culverts. This process is expected to take several hundred years. Dredging of the submerged levees and channel will not significantly alter this process as new sediments will continue to develop the system of levees. In addition, during high flow events, the lake levels are high and the lake sediments do not significantly impede the flow of water through the lake. Therefore, at this time, localized dredging is not considered beneficial to the hydraulic characteristics of the lake.

The effect dredging would have on Hatzic Lake's volume can be calculated using the results obtained from the 1991 bathymetric survey (Appendix G). In this study, proposed dredging was limited to the east side of Hatzic Lake from the mouth of Hatzic Slough down to the Everglades (sectors E2, E3, and

E4). Proposed dredge cuts to elevations 0.0, 0.5, 1.0, and 1.5 m GSC were estimated. The lowest dredge cut, to elevation 0.0 m GSC, resulted in the largest increase in lake volume, $2.6 \times 10^6 \text{ m}^3$. Table 2.11 summarizes the changes in lake volume and Table 2.12 lists the volume of dredge material removed for each cut.

The volume of live storage depends on the lake level at the beginning of a storm. Hatzic Lake levels can be separately defined as summer and winter lake levels. The six previous major storms were used to determine the typical summer and winter lake levels (Drawing No. 2.23). During the summer, the lake level averages 2.41 m GSC, although towards the end of August the lake can drop substantially. During the winter, the lake level can fall as low as 0.5 m, although it generally is about 1.0 m GSC.

For a storm between May and the middle of August, dredging would provide no additional live storage and would have no effect on flooding. On the other hand, it is during these months that the lake experiences the most recreational use and any additional depth on the east side of the lake would be a benefit to the recreational users. Table 2.13 shows that dredging to 0.5 m GSC could provide an additional depth of between 0.5 and 1.2 m on the east side of Hatzic Lake.

Because the winter lake level varies between 0.5 m and 1.0 m GSC, no storage benefit would be gained for dredging below 0.5 m GSC. Therefore, a dredge cut to 0.5 m GSC could provide a maximum of $1.7 \times 10^6 \text{ m}^3$ of additional live storage, provided that the initial lake level is less than or equal to 0.5 m GSC (Table 2.14). This volume represents approximately 19 mm of runoff from the entire basin (90 km^2) which is a small proportion of any previous event (see Table 2.7). Of the winter storms discussed in this chapter, all except the November 1989 event had a lake level greater than 1.5 m GSC prior to the flooding, effectively negating any effect dredging would have on flooding. The initial lake level for November 1989 was about 1.25 m GSC which would have provided less than $0.9 \times 10^6 \text{ m}^3$ of additional live storage. This represents only 6% of the total inflow volume and would have had no significant effect on the flood levels.

Drawing No. 2.24 shows the flood peaks for an idealized winter storm event (the inflow has been adapted from the November 1990 storm). The model was set up such that the east side of Hatzic Lake was dredged to 0.5 m GSC; the floodboxes remained closed; and, the existing pumps operated for the entire duration. These represent the worst possible operating conditions. For an initial lake level of 1.0 m GSC, the maximum reduction in the peak level is only 18 cm. For the more typical situation with the initial level at 1.5 m GSC, dredging only produced a peak lake level reduction of 7 cm.

Table 2.15 outlines additional winter storm events that further illustrate the small effect dredging would have on flooding. For each of these events, the large total precipitations were associated with a low Fraser River level (Drawing No's 2.25 and 2.26). In all these cases the Fraser level rises rapidly, effectively preventing the floodboxes from providing any flood relief. Because the winter lake levels are essentially the same as the mean river level for the 5 days prior to the beginning of a storm, no flood storage benefit would be gained since the lake level would be near or above 1.5 m GSC.

.10 Forward Routing Summary

Several conclusions can be drawn from the forward routing analysis:

- Blockage of the CPR culverts occurred during the November 1989 and November 1990 floods and may have increased the level and duration of flooding upstream of the CPR culverts.
- The addition of pumps (increased capacity of 8.34 m³/s) had the most significant effect on reducing flood levels and flood durations. Additional pumps would have essentially eliminated flooding during June 1981, halved the number of days Hatzic Prairie was flooded during July 1983, and reduced the duration of flooding for November 1989 and November 1990 by up to 1.5 and 3.5 days, respectively.
- Operating the additional pumps down to 1.8 m GSC further reduced flooding especially when it was assumed the lake level prior to flooding was 1.8 m. For this condition flooding during June 1981 was eliminated.
- Additional floodboxes and additional CPR culverts, in combination with the additional pumps, did not further reduce peak flood levels and had only a minor effect in reducing flood durations.
- Additional floodboxes and additional CPR culverts, on their own, had no effect on flood reduction when the Fraser River was higher than the slough level at the pumphouse. This was the case for June 1981 and July 1983 and could be expected during spring runoff and the summer months. During a large winter storm when the Fraser quickly rises, the floodboxes close and restrict any floodbox relief to the falling leg of the flood; however, it was found for the November 1989 and November 1990 floods that additional pumps with existing floodboxes and CPR culverts provided the best overall flood reduction.

- Control of flooding in the Dale Road area will be difficult using additional works at the pumphouse. This arises because the hydraulic gradient between Dale Road and the pumphouse is too small to drive the flow to the outlet works, and Lagace Creek responds faster than the lake to storm runoff.
- A 5-day storm during December 1979 was found to have a 10-year recurrence interval. Only additional pumps were found to reduce the flood stage on Hatzic Prairie and at Hatzic Lake, and the benefit occurred primarily on the falling leg of the flood.
- Dredging the east side of Hatzic Lake would provide no additional live storage for a summer storm; however, it would provide additional lake depth for recreational users. For winter storms, dredging the east side of Hatzic Lake would not provide significant volumes of additional live storage for two reasons: the runoff volumes associated with winter storms is significantly greater than the volume added by dredging; and, the winter lake levels at the beginning of a winter storm are typically too high which eliminates the storage benefits.

2.3 AGRICULTURAL AND RURAL DEVELOPMENT SUBSIDIARY AGREEMENT (ARDSA) REQUIREMENTS

.1 Inventory and ARDSA Criteria

An inventory of drainage channels and their road crossings is summarized in Drawing No. 2.27. The channels on Hatzic Prairie are generally overgrown and show little evidence of high velocity flow, their intricate and tortuous patterns being similar to those found on tidal flats (Drawing No. 2.28). Such tidal channels are formed by the efficient conveyance of large volumes of water to and from the flats during a short tidal cycle of 12 hours. Somewhat similar conditions apply at Hatzic Lake, where the water level can fluctuate by up to 4.0 m over a year and almost 1.0 m during a large storm.

The standard ARDSA drainage criteria for forage crops are the following:

- .1 In the dormant period 1 November - 28 February, the estimated runoff from a 10-year 5-day storm should be removed within 5 days.
- .2 In the growing period 1 March - 31 October, the estimated runoff from a 10-year 2-day storm should be removed within 2 days.
- .3 Between storms, in the seasons of the year when drainage is still the primary need, the base flow water level in the channel should be held

at least 1.2 m below the surface of the adjacent land, in order to provide a good outlet for tile drains.

Considering the first criteria, a 10-year 5-day storm was found to occur in December 1979 as shown in Drawing No. 2.22. For the existing situation, the Lagace Creek level at Dale Road would remain above level 3.0 m GSC for 9.5 days and under the optimum situation with additional pumps, floodboxes, and culverts the time would be reduced by 2.5 days. Therefore, under this condition, the runoff could not be removed within the specified 5 day time period.

For the third criteria, the mean lake level for the period 1 March to 31 October is 2.41 m GSC (Drawing No. 2.29) and, given the required freeboard of 1.2 m, half of Hatzic Prairie remains undrained at this level. Therefore, this criteria cannot be met for much of the area. Through much of this period the lake levels cannot be reduced below 2.41 m as the floodboxes are held closed during the Fraser River freshet and the existing pumps are unable to pump the water down below this elevation. This situation could be improved or eliminated if a new pump station capable of pumping down to 1.8 m were constructed. It should be noted however that this would have an adverse impact on the recreational value of the lake. The impact of this change on the recreational value of the lake and pump station operating costs would need to be considered. In addition, the lower lake level would have only a minor reduction on the degree of flooding during a major storm. During the spring planting period when summer storms occur, the existing freeboard of 0.6 m would be satisfactory in dry summers when high water levels and subirrigation would be beneficial to the area.

Observations of conditions also indicated that water levels in the creeks responded rapidly to lake levels, that most of the drainage channels were not unduly obstructed to flood water flow, and computations indicated that the Hatzic Slough and Lagace Creek could convey substantial flood flows provided that there exists a sufficient hydraulic gradient, which requires a low lake level. The drainage ditches and locations of bridges and culverts are shown in Drawing No. 2.27. Examination of historical air photographs as well as field reconnaissance completed during the early summer of 1991 indicates that low flows, from the drains flowing into Lagace Creek just upstream from Dale Road, were partially blocked by sand deposits within Lagace Creek, but this low obstruction would not block drainage during higher levels in the drains. Also, the recent excavation of Lagace Creek under the Flood Restoration Project has resulted in removal of this partial blockage and unobstructed drainage of ditches into Lagace Creek.

Recent water level surveys showed that during October 1991, while the lake levels were less than 1.2 m GSC and no rainfall had occurred for more than a month, the water levels in one drainage channel ranged between 0.5 m and 1.0 m below the adjacent fields. This level was kept high due to local bars and obstructions such as gravel deposits but is not representative of the drainage situation in Hatzic Prairie. One culvert (SW1 on Drawing No. 2.27) was blocked by the farmer in order to hold water on his land. Other than this blockage, flood flows moving down drains are not impeded unless local gravel accumulations occur such as occurred at Hereford Creek. However, this accumulation was also cleaned out under the Flood Restoration Project and there is presently no obstruction to high flows.

.2 ARDSA Conclusions

ARDSA requirements for agricultural drainage would not be met by enlarging the existing drainage channels on Hatzic Prairie.

If the lake level cannot be lowered quickly enough, meeting ARDSA requirements may require dyking of the main creeks and installation of secondary lift stations in the field drainage channels to isolate the fields from the main drainage channels. One farmer has already installed this type of system (Drawing No. 2.27; item SW2). The installation includes a 50 cm pump, 60 m of 900 mm culvert, 500 m of dyking and the blocking of two Ministry of Highways culverts to prevent water from coming back on to his land while pumping.

3.0 SEDIMENTATION

3.1 SEDIMENTATION AND FLOODING PROBLEMS

.1 Introduction

A large portion of Hatzic Prairie is an alluvial fan and flooding has been closely linked to the natural deposition of sediment over this fan. However, the rate of deposition has recently been accelerated by landslides and debris torrents which have brought down massive volumes of sediment. This section begins by describing the extent of flooding, reviews the historical development of channel avulsions, presents a picture of the water/sediment processes from the upper watershed to the lake, then presents more detail related to volumes of sediment moved along the channel system (sediment budget) taking into account the sources of sediment, the future sediment budget, and concludes by discussing the impact of logging on sediment production.

.2 Flooding

The Hatzic Lake watershed area, situated east of Mission on the north side of the Fraser River, covers approximately 90 km². Half of the watershed is covered by steeply sided mountains, which have been partially logged. Level, arable land is situated around the east and north sides of Hatzic Lake and accounts for 25.3 km² of the watershed. Hatzic Lake covers an area of 3.37 km².

Runoff from the watershed is carried primarily by Lagace Creek which originates in the Douglas Forest 15 km northwest of Mission and flows southward 10 km before entering Hatzic Lake. Runoff from the creek and the adjacent Hatzic Slough is eventually discharged into the Fraser River through Hatzic pumphouse and outlet works (Drawing No. 3.1, Photo 3.1).

Lagace Creek and Hatzic Lake have experienced persistent flooding problems over the last 50 years (Table 3.1) due to three main causes:

- .1 Channel aggradation and sedimentation on the Lagace-Pattison Creek Fan (Drawing No. 3.1).
- .2 Flooding of Hatzic Prairie from Lagace Creek breaking out of its channel.

.3 **Flooding of Hatzic Prairie and Lower Lagace Creek from backwater of Hatzic Lake.**

In the past ten years, flooding has occurred in December 1980, June 1981, July 1983, November 1989, February 1990, and November 1990. Drawing No. 3.2 shows the lake level and Fraser level at the pumphouse for five events (Fraser levels were not recorded at Mission during the February 1990 flood). The three most recent events provide a broad representation of the type of flooding that has occurred in the Hatzic Lake area. The following describes each of these three events and are partly based on conversations with B.C. Environment (BCE) personnel:

November 1989 Flood. Flooding occurred on Hatzic Prairie during the initial part of November. Heavy rains began on 3 November and continued intermittently for 7 days. The peak lake level, measured at the pumphouse, was 3.93 m GSC (13.0 ft) and occurred on 10 November. Both pumps, having a combined capacity of 6.06 m³/s, operated continuously from 9 November to 14 November .

Flooding on the prairie was observed to be caused primarily by backwater from Hatzic Lake. Lagace Creek and the sloughs leading into the creek did not overflow their banks. However, considerable sedimentation occurred in the Pattison/Lagace Creeks and along the lower reaches of several tributaries.

February 1990 Flood. Flooding on Hatzic Prairie was caused when Lagace Creek overflowed its left bank upstream of the Dale Road Bridge (Drawing No. 3.3). The flow across the prairie was from west to east. Lagace Creek, above the Dale Road bridge, was observed to have lost its full flow capacity, probably caused by sediment deposited during the November 1989 flood. At the time the creek overflowed its bank, backwater from the lake was not a factor in the flooding. The pumps were not used during the flood. The peak level reached by the lake was 2.41 m GSC (8.0 ft).

After the flood, some of the material deposited in Lagace Creek at Dale Road Bridge and upstream to Farms Road Bridge was removed, partly by local residents as well as by BCE.

November 1990 Flood. The main flooding occurred between 9 and 15 November. The estimated peak level reached by the lake on 10 November at the Loughheed Highway bridge was 4.32 m GSC (14.6 ft), 2.0 m above the normal lake level. Two pumps were used continuously from 9 to 18 November and from 23 to 27 November.

Drawing No. 3.4 shows the sequence of flooding for the November 1990 event and Photos 3.2 and 3.3 show views of the flooding. Table 3.2 lists observation of events as determined from BCE photographs of the Hatzic area during the flood. On 9 November, flow from Hereford Creek (North and South) and Dale Creek brought debris down from Dewdney Peak, causing the culverts under Sylvester Road to plug (Hereford South - 900 mm culvert; Hereford North - 2.13 m Multiplate culvert). This forced the water over the road. Water flooded out onto the east side of Hatzic Prairie and into Hatzic Slough upstream of the Dale Road Bridge. Beaver dams in the slough were thought to block water from flowing into Lagace Creek near the Dale Road Bridge. Water continued to flow over Sylvester Road during 10 November. In general, the flow across the east side of Hatzic Prairie was from east to west, opposite to the February 1990 flood. It was felt that Lagace Creek did not overflow its bank, as it did previously, because the creek had been partially excavated following the previous flood. Material with a diameter of approximately 50 cm (20 inches) was observed to have remained stable in the Hereford Creeks.

Downstream of the northern Farms Road bridge, Lagace Creek makes a sharp bend, turning from due west to due south. On November 10, at 4:00 pm, a debris flow down Pattison Creek occurred, bringing down large quantities of debris. At the sharp bend in the creek, the flow overtopped the right bank protection works and flowed out along the west side of Hatzic Prairie. Water now was able to flow down Bouchier Creek and back to Lagace Creek near the Dale Road Bridge. A large amount of wooden debris collected at the Dale Road Bridge and at the small wooden bridge upstream on Bouchier Creek.

The upstream extent of the flooding was approximately to the Clarke House, which is halfway between the Dale Road bridge and the Northern Farms Road bridge. On 11 November, the lake was observed to be backing up to the Dale Road Bridge.

.3 History of Channel Avulsions

Maps from May 1920 and airphotos from May 1930, showed that Lagace Creek flowed in a small, meandering channel as far upstream as Farms Road bridge (Drawing No. 3.5). The stream appeared similar to other inactive drainage channels and was not transporting large quantities of gravel bedload. By 1940, Lagace Creek had changed to a wider braided channel downstream of Pattison Creek and had shifted out of its former course approximately midway between Seux Road and Farms Road (Drawing No. 3.5). This dramatic change resulted from a landslide (Slide A) on the south side of Pattison Creek basin and a debris torrent along the tributary north of the slide

which introduced a large volume of sediment into Lagace Creek (Drawing No. 3.6). A debris torrent is a rapid movement of saturated soil, rocks and other debris down a steep mountain channel (Swanston and Swanson, 1976) and has been described as wet concrete moving big stones and trees (McPhee, 1989). At the time of this first slide there was no logging or road construction in Pattison Creek watershed and the landslide and debris torrent were probably caused by a major rainstorm (Won, 1970).

Ministry of Environment records indicate rapid sedimentation was occurring along the entire extent of the fan in 1951 and that annual dredging was necessary. By 1952, Lagace Creek had been diverted back to its former course and was straightened downstream of Farms Road bridge (northern).

A second major slide (Slide B) took place on the north side of Pattison Creek basin between 1952 and 1962 below a recently constructed logging road (Drawing No. 3.6). This second slide worsened aggradation problems along Lagace Creek throughout the 1960's. Ministry files recorded that in 1968 the creek aggraded up to 1.5 m in a single flood event, and that between Seux Road and Northern Farms Road bridge, the channel (between its natural levees) was up to 1.0 m above the surrounding floodplain.

Serious flooding problems appear to have decreased throughout the 1970's until the flood of December, 1979. This flood caused the creek to shift out of its channel at Farms Road bridge and flow into Bouchier Creek (Cheng, 1980).

During the severe 1989 and 1990 floods, both Pattison and Lagace Creeks experienced bank erosion as a result of debris torrent deposits which were generated by a debris torrent occurring at approximately 4:00 pm on 10 November, 1990. This debris torrent was triggered by a landslide (Slide C) which occurred above Slide B (Dwg. No. 3.12). Details of these slides will be presented in the next section. The most extensive bank erosion appears to be in the vicinity of the Sylvester Road bridge as shown in Photo 3.4. On Lagace Creek, the outside of the bend downstream from the Northern Farms Road bridge was eroded and a minor avulsion took place.

3.2 WATER-SEDIMENT PROCESSES ALONG PATTISON/LAGACE CREEKS

The water/sediment processes along a stream involve erosion, transport and deposition of the materials (sediment). An idealized river profile is shown in Drawing No. 3.7 which depicts the water - sediment processes along four distinct reaches: erosion, transport, deposition and delta reach, and this will be used as a framework to assess the processes along the Pattison/Lagace Creeks.

A slope profile of Pattison Creek is shown in Drawing No. 3.8 which indicates the erosion or source reach of this creek system as well as the transport and lower deposition reaches. Generally, the erosion reach is undergoing surface erosion and mass wasting and the amount of material supplied is usually less than the sediment transport capacity of the creek. This aspect changes in the transport reach where the sediment supply from upstream equals the channel transport capacity (channel is in equilibrium) and further downstream the sediment is deposited because of flat water surface slopes and reduced sediment transport capacity. A detailed discussion of these processes down the Pattison/Lagace system follows.

Along the upper erosion reach, the material on the slopes varies. The evidence of types of material comes from field assessment of three landslides. Along the southern portion of the watershed, coarse alluvial deposits form scree slopes below rock outcrops as shown in Photo 3.5. Further to the north, a recent landslide (Slide C) has exposed surficial glacial deposits of coarse colluvium, gravels and sands as shown in Photo 3.6. The material is rather loose throughout the length of the landslide scar (Slide C) as shown in Photo 3.7 and can be readily moved by future flows. Thurber Engineering Ltd. (1990) is of the opinion that the watershed "contains an unusually large proportion of slopes covered by soil deposits." The thickness of the soil is evident in Photo 3.6. Most of the material appears to be sand, with some larger gravels and cobbles. An estimate of the amount of material eroded during the debris torrent event is treated in the next section in dealing with the sediment budget. Farther down the slope, the material in Slide B appears to be more layered with gravel/cobble stratification and stiff silts and clays as shown in Photo 3.7 and Photo 3.8.

As the material within the upper watershed (erosion reach) erodes by flowing water and by mass wasting, it is moved downstream and deposits along the river bed (deposition reach), particles of sediment drop out wherever flow velocities are too low to transport the material. Deposition usually results in stratified deposits with coarser sizes at the upstream end of the stream. This scenario generally applies along Pattison/Lagace Creeks, except when a debris torrent occurs. Debris torrents are rapid movements of saturated soil, rocks and other debris and because of the occurrence of a debris torrent on 10 November 1990 (Slide C), the deposited material downstream from the landslides is a mixture of logs, boulders, cobbles, gravel and sand as shown in Photo 3.9. The deposit shows no stratification, with log jams serving as low dams to trap sediment and a post -debris torrent degraded channel winding its way up the alluvial fan. Field inspection indicates that the torrent deposition ends just downstream from the confluence of the Lagace with Pattison Creek (Photo 3.10). The deposit contains logs, has small boulders (maximum size about 30 cm) and a majority of sand in between the large bed sediments. Prior to the debris torrent, the reach from the waterfall to the confluence

would be classified as an erosion reach; however, during the debris flow event (less than 1 hour) the reach experienced deposition. After the event, the reach reverted to an erosion reach.

Further down Lagace Creek, the material is more sorted and reduces in size - between the Seux Road and the northern Farms Road Bridge the material has a D_{50} of about 30 mm (one inch) and a maximum size of about 200 mm (8 inches) as indicated in Photo 3.11 and Drawing No. 3.9. The size of bed-material decreases in a downstream direction - the material at Sta. 1+500 is generally smaller than at Farms Road bridge (see Photo 3.11) while still farther downstream at Dale Road Bridge (Sta. 0+050) the bed-material is predominantly coarse sand (see Photo 3.12). These reaches can be classified as deposition reaches.

Within the Hatzic Slough channel and in Hatzic Lake, the predominant sediment is sand and silt. The samples obtained during the study are summarized in Drawing No. 3.10 and indicate percentages of sand, silt and clay. At the delta deposit, there is a predominance of sand and this is thought to be due to sorting of the sediments by local shoreline currents and waves. A complete analysis of the sediment samples is presented in Appendix H.

The lower reach of Lagace Creek (downstream from Dale Road) and Hatzic Slough bed levels have not changed significantly from 1984 to 1991 as shown in the slope profile (Drawing No. 3.11). Also, the right and left banks (levees) are higher than the surrounding overbank areas.

3.3 SOURCES OF SEDIMENT

.1 Introduction

The recent (1990) avulsion downstream from the Northern Farms Road Bridge (northern) on the Lagace-Pattison Creek alluvial fan is partially due to the rapid sedimentation (deposition of sediment) along these creeks. Local residents have repeatedly stated that sediment loads from the watershed have increased and have attributed this phenomenon to logging on the upper slopes. This aspect will be examined in more detail.

Sediment is removed from upper portions of watersheds by surface erosion and by mass wasting (landslides) and both these processes will be examined in assessing the amount of sediment generated from the upper slopes.

This section begins by reviewing the sources of sediment within the watersheds of Lagace and Pattison Creeks, and comments on the magnitude of sediment production and the changes incurred over the years.

.2 Sediment Sources in Pattison Creek Watershed

Pattison Creek has a rather small drainage area of 6.0 km² out of the total 90 km² for the entire watershed flowing into Hatzic Slough, but it produces most of the sediment that ends up in the lower system. The slopes of the Pattison Creek watershed are relatively steep with the slopes ranging from 10 percent to 76 percent (8 to 37 degrees). Most of the watershed is now covered with regrowth, however, about 26 percent was logged between 1950 and 1972 and most of this is now partially reforested as shown in a recent airphoto (Drawing No. 3.12) and in Photo 3.13.

A small portion of the watershed, in the upper southeast corner, has experienced sliding in 1935 during a severe winter storm (Won, 1970). The sequence of mass wasting was previously shown in Photo 3.6. The early landslide now appears to be stable and shallow as shown in Photo 3.5. However, in the middle of the watershed two large, active landslides are evident; the lower one (Slide B) occurred sometime after 1952 and before 1962, while the upper landslide (Slide C) reportedly occurred at 4:00 pm on 10 November 1990. From photographs taken during a helicopter flight, the lower slide (B) appears to be progressing upslope as a headcutting process, but the sides of the slide are now somewhat stiff and there is little loose material at the base of the slide. Photo 3.7 shows the head of Slide B and Photo 3.8 shows the entire slide area. This slide was investigated in October 1982 by Dr. A.G. Mercer of NHC who described the slide as follows:

"The slide is about 75 m wide, its head is at el. 600 m and its foot is at el. 460 m. The creek slope is at about 0.25 below the slide. The slide is very dramatic in appearance but is more of a head cutting erosion area than a slide. It appears that the stream flowing into the steep gully has been cutting back into the gravel and boulder bed causing the gully sides and ends to collapse. A comparison with 1968 airphotos shows the slide has cut back about 10 m since then. This represents about 100,000 cubic metres of material eroded in 14 years." (NHC, 1982).

Mercer estimated that the rate of sediment supply due to the headcutting process was about 7,100 m³/year.

Stereo measurements undertaken in 1991 from air photos indicate the following sediment rates:

RATE OF SEDIMENT SUPPLY FROM LOWER SLIDE B

Year	Volume of Slide Material (m ³)	Volume Difference (m ³)	Rate of Sediment Supply (m ³ /yr)
1962	72,200		Instant 72,200
1968	222,700	150,500	25,080
1979	280,200	57,500	5,230
1990	382,000	101,800	9,250

The landslide has developed into a large scar on the hillslope and is now about 50 m deep and 400 m long.

As in other watersheds experiencing mass wasting, the volume entering rivers is controlled by the frequency of gully failures and the proportion of steeplands along the basin (Rood, 1990). There does, however, appear to be a trend towards reduced sediment production with time for Slide B.

The more recent upper landslide (C in Drawing No. 3.12) occurred during a major storm on 10 November 1990 and generated a debris flow which deposited large material as far as the Lagace Creek confluence. The volume of material initially moved as part of the landslide was estimated to be 18,000 m³ by Hungr (1990) and this mass removed more trees and material as it moved down in the form of a debris torrent. The total amount of material generated was estimated to be as follows (see Drawing No. 3.13 for graphical representation):

landslide =		= 18,000 m ³
debris erosion =	240 m ³ /m x 300 m	= 72,000
debris torrent =	20 m ³ /m x 1500 m	= <u>30,000</u>
		120,000 m ³

During the November 1990 rainstorm, there was also material eroded from Slide B (probably 10,000 m³) which was also moved down the system.

Therefore, the mass wasting (landslides and debris torrents) have supplied between 500,000 to 600,000 m³ of material from the upper portion of the Pattison Creek watershed since 1962. Most of this material deposited on the bed of the Pattison upstream of its confluence with the Lagace.

From assessment of the most recent air photo, 1990, (shown in Drawing No. 3.12), it appears that surface erosion of logged areas is no longer

a significant process. During flights over the watershed, visual inspection indicated a dense forest with evidence of landslides and debris torrents as shown in Photo 3.13. The surface erosion from this well treed watershed would be in the order of $50 \text{ m}^3/\text{km}^2/\text{yr}$ (based on data from the Pacific Northwest - Rood, 1984). However, surface erosion as well as minor gullying could have significantly increased during the 1960's and early 1970's when a portion of the watershed was logged as shown in a 1968 airphoto (Drawing No. 3.14). The logged areas show extensive rill erosion, small gullies, and small landslides which could result in sediment production as high as $2,500 \text{ m}^3/\text{km}^2/\text{yr}$, about 50 times the production from an unlogged area. If one considers that the period from 1963 to 1973 experienced high surface erosion, then the amount produced over this time may be about $60,000 \text{ m}^3$ which is only about ten percent of the sediment volume generated from the two more recent landslides. Therefore, the primary source of sediment within the Pattison Creek watershed has been major landslides coupled with debris torrents.

From assessment of the watershed, it is probable that more major landslides could occur and as reported by Golder(1991):

"Thus it is concluded that there is a 75 percent chance of a total sediment load of $550,000 \text{ m}^3$ being accumulated over 30 years and a 50% chance of $925,000 \text{ m}^3$ being accumulated".

This potentially high amount of sediment would be the result of Slide B headcutting into Slide C and/or the development of a landslide and subsequent debris torrent at the high logging road as shown in Photo 3.14. The conditions for generating landslides and debris torrents are now worse than in the past.

.3 Other Sediment Sources in Hatzic Basin

A number of small creeks enter Lagace Creek as well as Hatzic Lake (Drawing No. 2.1 and 3.1). From inspection of the creeks and the perimeter of Hatzic Lake, the only other creek bringing sediment into the lake is Draper Creek (see Photo 3.15), and the quantity is rather small. Lake surveys in 1991 indicate that the top of the delta was raised by about one metre since 1951 (see Appendix G). The upper watershed of Lagace Creek is not producing significant quantities of sediment and whatever material is eroded ends up on the floodplain near Allen Lake which is shown in Photo 3.13.

3.4 SEDIMENT BUDGET

.1 Introduction

A sediment budget down a river system shows the amount of sediment, sometimes of a specific size, moved through successive river cross-sections during a specified duration of time such as a day, month or year. The budget indicates where sediment is derived and stored within a river system. Sediment budgets have been developed for a number of watersheds, (Lehre, 1982) and are useful in development of sediment management programs. For the Pattison/Lagace system, it is important that the sediment budget is developed for the debris flow events since these events move the most sediment from the upper watershed to locations farther downstream. Therefore, the following section treats an approximate sediment budget for the November 1990 debris flow.

.2 Sediment Budget For the November, 1990 Debris Flow

The development of a landslide near a high-altitude logging road (see Drawing No. 3.12) triggered a debris flow which moved material from an altitude of about 975 m to the confluence of the Pattison-Lagace Creeks (elevation 60 m). An estimated 18,000 m³ of material moved downslope creating a debris flow which ultimately resulted in about 130,000 m³ approaching the apex of the Pattison Creek alluvial fan (see Drawing No. 3.15b), which is located at Sta. 6+200. From estimates of the thickness and width of the deposits over the alluvial fan, it appears that about 60,000 m³ of sediment was deposited between the apex at the waterfall and Sylvester Road Bridge, and another 40,000 m³ was deposited prior to the confluence leaving about 30,000 m³ to pass downstream as "through-flow" sediment. The previous estimates pertain to a short time period, probably less than one hour, over which the debris flow occurred. Subsequent to the debris flow, sediment was eroded from these deposits due to continuing heavy rainfall (Section 2.2) and another sediment budget is shown for the post - debris situation which lasted throughout the remainder of 10 November 1990. It is difficult to estimate transport rate, therefore, the amount of sediment removed was estimated from photographs that showed the size of the low flow channel over the Pattison Creek alluvial fan (see Photo 3.9 as an example) and came to 30,000 m³ at the confluence. The post-debris torrent sediment budget is shown in Drawing No. 3.15c. The amount of sediment passing Dale Road bridge is uncertain for both the debris torrent and the post-debris torrent situation, but information from the PEP-funded Flood Restoration Project in the summer of 1991 indicates that about 40,000 m³ of sand and gravel was excavated from Lagace Creek between Farm's Road Bridge and about 500 m downstream from Dale Road Bridge. Therefore, the amount of material

entering Hatzic Lake from the debris flow is uncertain, but it could range from 5,000 to 10,000 m³. During times of no debris flows, prior to November 1990, the sediment inflow rate would be significantly less.

A survey of Hatzic lake in July 1991 resulted in a contour map of the existing lake bed which was compared to a previous survey undertaken in 1951. Appendix G presents contour plots of the two surveys and the changes in lake bed elevation between 1951 and 1991. The deposition along the east side of the lake was computed to range from 306,750 m³ to 437,500 m³ over the 40 year period which means a sediment inflow rate of 7,600 m³/year to 11,000 m³/year. These rates of infilling are similar to the estimated sediment volumes generated from the watershed, but, most of the sediment production ended as deposits on the Pattison Creek alluvial fan, the bed of Lagace Creek and Hatzic Prairie and did not move into the lake. The 1951 surveys may be suspect - the datum during the 1951 survey is uncertain and this could account for the high computed deposition in the lake. However, even if the sediment budget is imprecise, it may still serve as a guide for future management of sediment.

The improvement works necessary to manage sediment are dealt with in Section 4, however, a scenario of sediment production is presented here for a ten year period.

.3 Annual Sediment Budget Over the Next Ten Years

A relatively large amount of sediment has been stored between the waterfall and Sylvester Road bridge due to the November 1990 debris torrent and as previously described, much of this material can be easily removed by flowing water. From field examination of post-debris torrent erosion, it was judged that 30,000 m³ could be removed from this stored sediment and another 20,000 m³ could be derived from the upper part of Slide C and from Slide B. This erosion rate would diminish, due to coarsening of the creek bed, with time to about 10,000 m³ which was the long term sediment transport capability computed by NHC (1985). In addition, a debris torrent or the headcutting of Slide B into Slide C could result in another major input of say 100,000 m³ into the Pattison/Lagace system.

For development of a sediment management plan, the following annual sediment budget is assumed over the next ten years:

- .1 Year 1 to 5 - assume an initial annual volume of 50,000 m³ reducing gradually to 10,000 m³ at the end of year 5; then

- .2 Debris Torrent at End of Year 5 - assume about the same sediment budget ($100,000 \text{ m}^3$) as during the 10 November 1990 debris torrent with the Lagace Creek bed rising to the same levels as after the 1990 torrent; and
- .3 Year 6 to 10 - assume an initial annual volume of $50,000 \text{ m}^3$ in year 6, reducing to $10,000 \text{ m}^3$ at the end of year 10.

The initial sediment volume of $50,000 \text{ m}^3$ may appear somewhat high, but there is a large volume of loose material at the bottom of Slide C which will be moved readily by flowing water and farther down the gravel excavation has resulted in a loose surface of sand and gravel which will also be easily moved as has already occurred during a small rainstorm on 11 November 1991 and shown in Photo 3.16.

3.5 IMPACT OF LOGGING ON SEDIMENT PRODUCTION

Since the majority of sediment along the Pattison/Lagace Creek system was derived from landslides and debris torrents that were triggered by landslides, it is essential to assess the reasons for the landslides.

The landslide in 1935, (Slide A), occurred during an extremely intense rainstorm as reported by Won (1970) and was probably a result of an intense local rainstorm. As stated by Hungr, Morgan and Kellerhals (1984) -

"Debris torrents in coastal B.C. are triggered primarily by concentrated 'cells' of high precipitation, occurring within relatively common storm events with apparent return periods of 2-5 years".

The date of the occurrence of Slide B is uncertain, but it may have occurred during an intense storm in January 1961. The airphoto in Drawing No. 3.6, dated 21 July 1962 shows the slide, so it could have occurred during the winter of 1961. The head of the slide is located adjacent to a recently constructed logging road which may have served to trigger the slide.

The 10 November 1990 debris torrent was initiated by a landslide in a small gully and as stated by Thurber (1990) - "where a logging road crossed the head of the gully." From study of 1989 and June 1990 air photos, a small slide scar was evident just downslope from the road in the small gully - this slide became much larger on 10 November 1990. The initiation of this slide would probably be due to two factors, the logging road and a high intense rainstorm in the watershed. Recent oblique photos also indicate four small slides on the logging road (Photo 3.14) and these could develop into larger slides in the future. Also, historical airphotos (1968) indicate paths of former debris torrents originating at logging roads.

From assessment of the logging roads, the condition of the watershed (steep slopes) and the landslides, it appears that logging has significantly increased the production of sediment which has resulted in aggradation of the Pattison/Lagace Creeks. If this material was not excavated, then flooding could occur in the future. This conclusion corresponds to findings in the Queen Charlotte Islands (Rood, 1984) where:

"Road building and the removal of vegetation accelerate the overall frequency and yield of landsliding. As well, logging alters the average size of debris slides and debris flows and their relative importance to the total yield. Debris slides in clearcut are accelerated 30 times in frequency, but only approximately 15 times in yield."

There will probably be more landslides and debris torrents in Pattison Creek watershed in the future even if constructive logging practices are used. Guidelines for improved logging practices are presented in Section 4.0.

4.0 INVESTIGATION OF IMPROVEMENT WORKS

This section of the report provides details of capital works improvement options designed to mitigate the drainage and sediment transport problems identified within the study Terms of Reference. Details of the improvement options are provided together with a brief statement on environmental concerns, maintenance and cost estimates. A detailed break down of construction cost estimates is provided for future information in Appendix D of this report. Reports received from specialist geotechnical and environmental subconsultants, which have been used in the preparation of this report and these cost estimates are included in Appendix B and Appendix C respectively.

The anticipated impact on flooding and sediment problems is addressed in Section 2 and 3 of this report and therefore a description of the benefit of the major improvement options is not included within this Section.

4.1 MAJOR FLOOD RELIEF OPTIONS

.1 Pump Station and Floodbox Structure

A review of the study area indicates two possible locations for the new floodbox/pump station which would appear viable. The most obvious location for the new structure is in the vicinity of the existing floodbox and pump station near the confluence of the Hatzic Slough and Fraser River. The second option is on the Chilqua slough. No other locations appear viable.

It had been suggested that an outlet from the Chilqua Slough might be a better outlet to the Fraser River rather than improving the existing outlet works. A site inspection was carried out to investigate the feasibility of this proposal.

The possible location for an outlet from Chilqua Slough to the Fraser River is at the bend of Nickomen Slough closest to Hawkins Pickle Road, west of Catherwood Road. Chilqua Slough flows westward from this point towards Sylvester Road. After crossing Sylvester Road, the slough currently flows in a northwesterly direction to Hatzic Lake. From contour-planimetric sheets, it is noted that Chilqua Slough has a drop in grade of 0.6 m from the proposed outlet to the Lake, made up of a drop of 0.2 m over a distance of 2600 m from the proposed outlet to Sylvester Road and a drop of 0.4 m over a distance of 1150 m from Sylvester Road to the Lake.

Also it should be noted that the proposed outlet (to the Fraser River) which is on Nicomen Slough is at a higher elevation by 0.3 m than the existing outlet

at Hatzic Slough. Therefore overall this proposal has to overcome a hydraulic grade disadvantage of 0.9 m (3 feet), or the Lake level must be allowed to rise by 0.9 m in order to achieve gravity outflow from the Lake to the Fraser River.

To determine whether the preferred location for a new floodbox/pump station structure would be at the Chilqua Slough or adjacent to the existing pump station, several factors must be considered including:

- Operational and maintenance advantages associated with placing any new structure close to the existing structure.
- Construction capital costs.
- Operating costs.

In the consideration of capital construction costs several work items would be common to either location. These items include placing culverts under the railway, pump station and floodbox construction. The cost of constructing these works at either location are considered to be essentially the same and therefore will not be considered further in this comparison.

In order to achieve gravity flow at the proposed outlet on Chilqua Slough, the slough has to be regraded to reverse the flow from the Lake to the proposed outlet, over a distance of 3750 m (2½ miles). In addition four access road culverts (between the Lake and the outlet) have to be installed to pass the projected flows unimpeded to the proposed floodbox outlet.

Following is a rough cost estimate only of the additional capital construction costs to convey the Lake flows to the upstream end of the proposed outlet from the Chilqua Slough.

.1	Regrading of Chilqua Slough: 3.75 km (by 0.6 m minimum)	
	$3,750 \times 20 \times 0.6 = 45,000 \text{ m}^3 @ 6.00$	\$270,000
.2	4 Road Access Crossings from the Lake up to proposed outlet structure. Each crossing with 2 rows of 3 m dia culverts 15 m long 120 m x 3 m dia CSP @ 1,000.00/m	<u>120,000</u>
	Subtotal	\$390,000

35% Engineering and Contingencies (15% survey and detailed design, 10% contract administration and inspection, 10% contingencies)	<u>136,500</u>
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TOTAL	<u>\$526,500</u>
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This is a preliminary cost estimate for work required to convey the flow up to the inlet side of the proposed outlet works, without considering any right-of-way costs and dealing with property owners along Chilqua Slough.

To offset this, additional capital work costs associated with placing the proposed new structure close to the existing structure include:

.1	Upgrading existing power poles to accommodate 25 kv	\$ 6,100
.2	Replace existing main electrical switch	6,000
.3	Replace existing transformers at pump station to accommodate upgraded power supply and primary metering	<u>40,400</u>
	Subtotal	\$52,500
	35% Engineering and Contingencies	<u>18,375</u>
	TOTAL	<u>\$70,875</u>

Therefore, there is a saving of approximately \$455,000 in capital construction costs which would be realized by selecting the existing outlet location at Hatzic Slough rather than the proposed Chilqua Slough location as the location for any further outlet structures.

A significant difference would also exist in operating costs depending on which outlet location is selected. If any new pumping facility were placed close to the existing facility, it would be possible, subject to approval by BC Hydro, to meter the power consumed using primary metering with a single meter servicing both pump stations.

Based on a total power consumption of 210,000 kWh/month this arrangement would result in a savings of \$219/month based on current rates for those months when pumping will occur.

Of perhaps more significance is the minimum monthly charge. BC Hydro will invoice a minimum monthly charge of 50% of the highest maximum demand charge billed in any month wholly within an on-peak period during the immediately preceding eleven month period. The on-peak period is from

November 1 to March 31. Should the pumps be operated during this period, which is likely to occur, the minimum charge applies, regardless of actual power consumption. This cost is estimated to be approximately \$1225/month for the existing station. By metering the two stations through one primary meter the possibility of facing two individual minimum monthly charges is eliminated.

In conclusion, it will be much more advantageous to construct any new works close to the existing outlet location with necessary improvements to the CPR culverts.

The Terms of Reference indicate that a total pumping capacity of $14.4 \text{ m}^3/\text{sec}$ would be sufficient to control runoff from a 10 year -2 day summer storm occurring between May and August when the Fraser River is in freshet. This criteria has previously been accepted as the basis to size a pump station for an ARDSA drainage improvement project in the Matsqui Prairie. Hydrology and flood routing studies carried out under this study confirm the proposed capacity suitable for the new pump station. In addition, doubling the existing floodbox capacity would provide a suitable floodbox within reasonable economic guidelines. Accordingly a new pumping station if constructed would require a total capacity of $14.4 - 6.06 = 8.34 \text{ m}^3/\text{sec}$.

The location and type of pumping station best suited and most appropriate to operate in tandem with the existing station to deliver $8.34 \text{ m}^3/\text{sec}$ is the subject of this investigation. Four possible locations were investigated together with several different pumping configurations. The following information details the results of the investigation.

.1 Pumping Station Location

Four possible locations in the vicinity of the Hatzic Slough were considered for the location of the new pump station.

- .1 Adjacent to the existing station as an extension to the existing structure.
- .2 At the middle of the slough, adjacent to the dyke.
- .3 On the north side of the slough adjacent to the dyke.
- .4 On the south side of the slough adjacent to the dyke.

A station located adjacent to the existing pumphouse is feasible but cofferdaming, dewatering and excavation of the foundations will be very expensive. Construction of a pumping station at the middle of the slough is not practical due to soil conditions, dewatering and excavation costs and the questionable stability of the dyke. A station

located on the north or south side of the slough adjacent to the dyke appears feasible and practical. Based on geotechnical investigation (see Appendix B) the south side location has been confirmed as the better location for a new station and this is therefore the recommended site for a new pumping station.

.2 Pumping Station Configurations

The evaluation of the pumping station configuration focused on the arrangement and number of pumps, specific pump types, the type of drive and the optimum pumphouse location. All pumps irrespective of physical construction are of the axial flow type capable of delivering large volumes of water at low heads and high specific speeds. The two specific pump types considered were long column wet well propeller pumps and submersible close coupled draft tube mounted propeller pumps. The former type have been traditionally employed as flood control pumps throughout the Fraser Valley and delta regions and have provided long and useful service. However, the development of large submersible motors for flood control applications from domestic sewage pumps has provided an economical and highly practical alternative to the traditional long column pump. This alternative has been successfully adopted in both western Europe and western Canada with highly satisfactory results. Earlier criticism of the submersible types with respect to low hydraulic efficiency and a narrow marketplace have been addressed by improved pump design and the expansion of several pump suppliers into the market in the last ten years. However, when the prime mover is to be an internal combustion engine then the long column pump really becomes the only choice.

Pumping stations that have been designed specifically for submersible pumps tend to be more price effective with respect to capital works. From an operational and maintenance aspect there is little difference between the submersible pump and the long column pump, both units having a history of providing long term satisfactory service. The submersible unit does have an advantage over long column pumps when maintenance is required, as removal from the station and transportation to the nearest shop for general repair and overhaul tend to be simpler and more economical due to the unit construction and compact size of the submersible pump. This ease of installation and removal also permits submersible pumps to be quickly replaced with a spare standby unit when necessary.

Accordingly, all pump configurations in this study focused on the submersible pump application except where an internal combustion engine is considered as the prime mover.

Pumping arrangements for simplex, duplex and triplex systems were examined.

.3 Simplex Pump Arrangements

A single submersible pump having a capacity of $8.34 \text{ m}^3/\text{sec}$ is not readily available in the marketplace at present, accordingly this option was not pursued further.

Large capacity long column wet well propeller pumps are available from a limited number of suppliers. They can be supplied with electric motors or directly driven (via a right angled gear box) by a suitable internal combustion engine, i.e. - natural gas, gasoline or diesel powered. A single pump capable of delivering $8.34 \text{ m}^3/\text{sec}$ (132,000 US gpm) at 3.65 m (12 feet) total head would require an input horsepower to the pump shaft in excess of 500 hp. The minimum installed electric motor recommended for the above condition is 600 hp. If the motor is to be sized to handle the peak total head condition of 7.0 m when the Fraser River is at extreme high flood level then a minimum Bhp rating of 750 hp would be required. Depending on pump selections actual discharge rates would be approximately 70% to 80% of the low head design flow at the high head condition. A 750 hp electric motor will require a high voltage (i.e. 4160 volts) service which will substantially increase the cost of electrical components and requires that only certified high voltage electricians service the motor and equipment.

For a diesel powered pump capable of meeting these discharge requirements the recommended motor size would be approximately 1000 hp.

A single pump arrangement of the long column type driven by an internal combustion engine is the logical choice for a simplex pump system. The optimum engine type would be diesel fuelled. The pumping unit could be housed in a wood framed structure supported on timber piles. The arrangement would consist essentially of the following:

- .1 One (1) long column wet well type propeller pump.
- .2 One (1) right angled gear drive.

- .3 One (1) diesel engine complete with centrifugal clutch and engine control panel.
- .4 Timber pile structure with housing for pump and diesel engine.
- .5 One (1) fuel system consisting of a 5000 gallon fuel storage tank and 500 gallon day tank. The main storage tank will be contained in a concrete structure to contain spills and prevent damage to the tank and environment from vandalism.
- .6 A single 2,134 mm (7' dia) steel discharge pipe.

The advantages of the above arrangement are the elimination of the BC Hydro demand charge and the flexibility to operate the system at two or more speeds, from an engine mounted governor, thus adjusting flow output to suit flood levels. In addition diesel power would provide the additional protection that in the event of a power outage on the BC Hydro grid, the pump station would be able to continue to operate. The capital costs of such an arrangement are estimated to be \$1,611,700 as detailed in Appendix D, along with estimates of operational and maintenance costs. The cost estimates reflect a wood framed structure extending into the slough with a timber access bridge. This arrangement has the disadvantage that it is expensive to construct and operate, will require regular maintenance, and must be continuously refuelled by tanker trucks. A 5000 gallon fuel tank is expected to provide up to 7 days continuous running time of the engine. The single unit system provides the least security and flexibility when compared with multiple pump use in that a failure of any vital component would effectively eliminate 100% of the pumping capability.

.4 Duplex Pump Arrangement

A duplex pumping system delivering 8.34 m³/sec ultimate capacity is possible using either submersible type or long column wet-well type pumps. Pumping units would be selected to meet two conditions.

- .1 Deliver 4.17 m³/sec at 3.65 m total head.
- .2 Deliver at least 70% of the average total head design flow at a maximum total head of 7 m.

A preliminary review of possible pump selections indicates that these conditions can be met by both submersible and long column pumps. Installed motor horsepower would range from 260 hp to 400 hp for the two head condition; therefore to meet the higher head condition a 400 hp motor would be used. Hydraulic efficiencies would range from 72% to 82% depending on static head conditions. Costs for the duplex

arrangement are estimated to be \$2,174,100 for a duplex long column pump station and \$1,830,000 for a submersible pump station as detailed in Appendix D for both pump configurations. The submersible pump arrangement would include a two sump system protected by a trash rack complete with one fish pump. Each pump would be fitted in its own draft tube and elevate the water into a separate discharge chamber. Individual discharges from each discharge chamber would be fitted with flap gates and would discharge to the river via a common pipe or concrete flume. The long column pump arrangement would be similar to the existing station.

The submersible pumping station provides a low profile structure of relatively simple construction which results in an attractive, economical facility. The modest superstructure necessary to house the electrical components requires few building services. Maintenance advantages of these pumps have already been noted in the previous section. Diesel engine drives are not applicable to the submersible pump configuration. The pump control system would be from simple level sensors in the wet well which would start/stop each pump on some predetermined slough levels. Low and high level alarms with low level pump cut-off would be included in the control system.

The duplex pump arrangement provides a distinct advantage over the simplex arrangement in that should one pump fail to operate the station will still be able to pump 50% of its capacity.

.5 Triplex Pump Arrangement

The triplex pump arrangement is identical to the duplex system except that it would include three pumping units instead of two. Pumping units would again be selected to meet two different conditions, as described below.

- .1 Delivery of 2.8 m³/sec at 3.65 m total head.
- .2 Delivery of at least 70% of 2.8 m³/sec at 7 m total head.

The installed motor horsepower to meet the two conditions range from 200 hp to 260 hp for each pump, therefore a 260 hp motor minimum is recommended to satisfy the high head condition. Hydraulic efficiencies range from 70% to 80% depending on the static head condition. The three pump system offers greater operational flexibility but at the price of an additional pump with attendant components. The three pump system also provides the added benefit that should

one pump fail to operate the station will still be able to pump 67% of its capacity.

The capital costs associated with a triplex arrangement of \$1,908,300 are higher than for alternative systems but the operational flexibility and potential for energy savings make it the most attractive choice.

.6 Screens

Various screen types are used on flood control pumping stations to screen out debris and protect pumps from damage. The most common type is the coarse bar screen which consists of galvanized steel bars welded at approximately 75 mm centres in a metal frame. They are installed at an angle to permit trapped debris to be raked up and off the screen as required. Screens should be used in conjunction with a floating log boom to prevent floating debris from entering the forebay of the station.

Inlet bar screens with automatic raking devices are available from manufacturers, mostly located in Europe. They are of two types: the fixed type and transversing type; the fixed raking device is limited to widths of up to 6 m. Automatic raking devices operate from head differential over the screen. Screens with automatic raking devices are not recommended for this application due to costs and additional maintenance.

Special screens designed to keep out small fish have been used by the Department of Fisheries and Oceans. However, high head losses can be expected over such screens. Therefore, much larger openings to the wet wells are necessary. Stationary and automatic travelling screens which are commonly used for potable water supplies or in power plants are equipped with fine wire screens having openings of 2.5 mm in British Columbia and 9.5 mm in other parts of western Canada. Approach velocities as demanded by Fisheries can be as low as 0.15 m/sec, resulting in very large screen structures. Automatic travelling screens are energized by differential head over the screen, by timer or simply manually started. The safety and well being of small fish and fry is becoming more important to Fisheries with threats that they may levy a tax on the operating municipality based on the fish loss due to pumping stations and its economic impact on the fishing industry.

Based on a previous application in the District of Matsqui, it is proposed to provide fish screens and a fish pump for the new station.

For the triplex arrangement, one pump can handle the summer base flow of approximately 2.5 m³/sec. From the existing pump station operating records, it is noted that for the months of May to August on the average one pump operates for approximately 10 hours per day. For the duplex configuration, the size of the screens would need to be significantly greater than installed at Matsqui.

.7 Operating and Maintenance

The introduction of this additional pumping facility will have an impact on both the operating and maintenance costs of the pumping stations. The impact on general operating and maintenance cost is relatively small for this new facility and is estimated to be approximately \$3,000 per year. The operating costs however could increase significantly depending on how the station is actually operated. Table 4.1 is included to illustrate the impact of adding this additional pumping station on flood peaks and durations and on operating costs during the past 11 years. During this period the average annual power consumption costs would have increased by 57% from a calculated value of \$36,443 to \$57,319.

This increase in cost is associated primarily with pumping during winter months. During these periods the pumps work when the Fraser River is high preventing any flow through the floodboxes which may coincide with a rainfall event. This usually occurs for relatively short periods of time, after which the Fraser River levels fall allowing water to flow by gravity out of the Hatzic Lake system. Under these circumstances the total volume of water discharged by the pumps is substantially more than with the existing pump station and the power charges are correspondingly higher. The use of the larger number of pumps during winter months (November to March) also impacts heavily on the minimum monthly charges imposed by BC Hydro. Therefore the net result of using the additional pumps during winter periods is a significant increase in operating costs. The cost calculated for Table 4.1 is based on all pumps starting simultaneously at elevation 2.4 m which is conservative. Costs would be reduced by providing for a cascading start for the pumps as the water level rises.

.2 CPR Crossing of Hatzic Slough

The Hatzic Slough is crossed by two major transportation facilities south of Hatzic Lake: Highway No. 7, a provincial highway, and the Canadian Pacific Railway mainline tracks. The highway crosses the slough by means of a bridge and the two railway tracks, which lie south of the highway, are

supported on an earth embankment. The slough waters are carried by three corrugated metal culverts (two 3.05 m dia and one 2.44 m dia) through the embankment.

These existing culverts are sized to pass sufficient water to match the capacity of existing floodboxes at the Fraser River dyke and therefore do not normally represent a significant restriction in the capacity of the overall Hatzic Slough system. The exception to this general rule occurs when floating debris builds up in front of the railway culverts, partially blocking flow, which has occurred during recent flood events. Some head loss is however associated with these structures. Should additional outlet works (floodboxes) be constructed at the Fraser River it would be necessary, however, to provide additional cross track drainage facilities at the CPR embankment to ensure the water flowing in the slough is able to reach the improved outlet works and not be restricted by these existing culverts.

Additional cross track drainage could be provided by two means: installation of two additional 3 m diameter culverts; or removal of the existing culverts and installation of a bridge across the slough. From a hydraulic point of view the preferred alternative would be the construction of a bridge. This approach would overcome the problem which currently exists with debris build up at the culverts. During extreme flood events, large volumes of floating debris, including picnic tables, have accumulated at the entrance to the culverts. The entry configuration and location of the diverter boom have proved inadequate for the quantity of debris and as a result it is understood a significant restriction in the flow can result.

Any proposed work at this location would require the approval of CP Rail. The possibility of installing either a new bridge or additional culverts has been raised in correspondence with CPR and it is understood no work would be permitted which would limit the use of the tracks. To meet this requirement, both tracks would need to be in service 24 hours per day and therefore to build a bridge would require the construction of at least one "shoo-fly" or by-pass track. This requirement would appear to be too onerous as the construction of this track would undoubtedly make the bridge alternative economically unacceptable and quite possibly environmentally unacceptable. Construction of a by-pass track would require widening the existing embankment either north into the large drainage ditch between the highway and railway or south into the Hatzic Slough. Either approach would potentially result in some loss of habitat for the fisheries resource.

A more realistic approach would be to close one track at a time in the vicinity of the new bridge and install crossovers on either side of the bridge site. By controlling rail traffic using on site flagman, it is believed construction of two

bridges, one per track, one at a time, could be completed at little or no inconvenience to CPR. Utilizing this approach, the cost of two single span bridges 32 m long is estimated to be \$1,105,000 including engineering, contingency and railway charges. Also included are the costs of temporary works required to support one track when the second track is removed to allow construction of the first bridge.

The second alternative is the installation of two 3 m diameter culverts. As no geotechnical work has been completed, it is unknown whether installing these large diameter culverts is viable without additional ground stabilization measures. In order to install these culverts tunnel liners are required. For this technique, a tunnel is excavated in advance of the pipe. The tunnel must be self supporting until the pipe is assembled in place using a number of liner plates to form the complete pipe circumference. As the site was previously a rail trestle and it is believed the embankment was formed by simply end dumping granular material between the trestle bents, which most likely remain in place, it is quite possible the ground will not be self-supporting. The cost of installing these two culverts is estimated to be \$1,000,000 if ground stabilization is not required or \$1,750,000 if ground stabilization is in fact required.

Although the installation of culverts is potentially the least expensive approach, the bridge is considered to be the preferred option. As no geotechnical data is available for the site, the degree of uncertainty associated with installation of the large diameter culverts is considerable. This degree of uncertainty and hence possible increase in cost is not as critical with the bridge. The only unknown associated with the bridge is the length of piles which are only a small fraction in the overall costs. Therefore should the length of piles increase significantly over the length assumed in the cost estimate, the associated increase in cost would be relatively small.

In addition, construction of a bridge would overcome existing problems with debris and overcome the need to construct a new diverter boom north of the highway. However, as noted above in order to make the bridge a viable alternative negotiations would need to be completed with CPR to enable one track to be closed for approximately 14 days. (7 days per track).

The responsibility for installation of any additional cross track drainage works is unclear. It is understood the Regional District believe CPR would be responsible for the cost of any new works at this location, however, CPR have not confirmed this understanding. Once again this would need to be established during negotiations with CPR and the Regional District may wish to obtain a legal opinion on this matter.

3 Debris Control Structures

As noted earlier, the Hatzic Slough is conveyed through the CPR embankment south of the Loughheed Highway by three large diameter corrugated metal pipe culverts. The culverts are protected by a small floating log boom directly in front of the culverts which prevents debris from becoming lodged in the culvert inlets. This diverter boom is relatively small and is located between the railway and highway.

During recent major flooding events large volumes of debris have accumulated adjacent to these culverts. (See photograph No. 2.1) The debris has consisted of large floating objects including picnic tables and sheds. The volume of debris has been greatly in excess of that which is able to be accommodated by the existing diverter boom and as a result significant head losses may develop at this location within the overall drainage system.

The anticipated head loss at this location is confirmed by the flooding routing analysis contained in Section 2 of this report and by anecdotal information obtained from the pump station operator Mr. Albert Oliver. Mr. Oliver indicated a significant drop in water surface elevation occurred across the culverts during the November 1990 flood when a large volume of debris accumulated in front of the culverts.

Two alternatives for a debris control structure were reviewed for this general location. The preferred location for any control structure is north of the Loughheed Highway where sufficient space is available to divert debris away from the main flow path. The two alternatives are shown on the attached drawings (Drawing No's 4.7 and 4.8).

The first alternative comprises an articulated walkway and floating boom supported between two rows of steel pipe piles. In addition, an access road is provided on the east side of the diverter boom to enable maintenance vehicles to park off of the highway and assist with the removal of debris. The estimated cost of the floating walkway and piles including access road, engineering and contingencies is \$116,500.

A second alternative for debris control upstream of the Loughheed Highway bridge is shown on Drawing No. 4.8. A three pile dolphin is installed in the centre of the outflow channel together with two anchor piles installed on either side of the highway bridge. Boom chains are attached to the centre dolphin and the anchor piles. For this arrangement access ramps are required on both sides of the bridge to remove the accumulated debris from the booms. The estimated cost of the piles, boom chains and access ramps is estimated to be \$93,000.

In the event that the existing culverts under the CPR mainline tracks are removed and a bridge is installed at this location the debris control structure will not be required at this location. However, under these circumstances any debris would pass through the highway and railway bridges and accumulate adjacent to the existing or proposed pump station and outlet works. Therefore should a bridge be installed at the CPR tracks a debris diverter boom should be installed on the Hatzic Slough adjacent to the outlet works. At this location a simple floating boom should provide adequate protection as the slough is quite wide and could accommodate significant quantities of debris without effecting the overall performance of the outlet works. The estimated cost of a diverter boom at this location is \$27,000.

Any structure to be installed on the upstream side of the Lougheed Highway would require vehicular access to the highway, and therefore, a permit from the Ministry of Transportation and Highways would be required for this installation. In addition, CPR have confirmed they are responsible for maintenance of the existing culverts at this location and this would include the removal of debris. Prior to relocating the diverter boom to the upstream side of the highway bridge it is recommended an agreement be reached with CPR to ensure they will continue to maintain this structure and be responsible for removal of any debris which accumulates adjacent to the diverter boom.

.4 Lake Dredging

Local residents have requested the Lake be dredged of sediment to improve the live storage potential for storm water, and therefore reduce flooding, and to improve the recreational value of the lake. The effect of lake dredging on flood levels is addressed in Section 2 of this report. In this section cost estimates for the work are provided together with a brief description of potential environmental concerns.

Three contracting companies were approached to obtain data for the dredging of Hatzic Lake. Companies approached were Fraser River Pile and Dredge Ltd., Miller Contracting Ltd., and Dillingham Construction. Dillingham Construction confirmed they only carry out dredging using a clam shell. Miller Contracting and Fraser River Pile and Dredge are the only two local companies who carry out suction dredging. The bathometric survey for the lake together with planimetric drawings for the study area were provided to both contractors to indicate the extent of dredging and possible disposal sites.

Two estimates were obtained from each company. The first estimate was for dredging and pumping to a lakeside disposal, suggested to be in the southwest corner of the lake where a possible golf course development may proceed. The second disposal site was more remote, at the Anderson Sod Farm south

of CPR lines. This would require trucking approximately 6.5 km and should this site not be available, it is anticipated a disposal site could be located within 6.5 km of the lakeside pump discharge point.

BC Lands were approached to determine the royalties payable for dredged material from the lake. BC Lands have indicated that if the project were completed by the Regional District in all likelihood no royalties would be payable. Royalties are generally established based on the difference between the market value of the material and the cost of extracting the material. It is understood from discussions with local residents that the material extracted from the lake could be disposed of on local properties, however, the indication is this material would be accepted only if provided at no charge. Based on this understanding there would be no royalties payable as the material has no market value.

An analysis of the lake sediments has been completed. This material is suitable for placing on fields for use as topsoil, provided adequate fertilizers are added to the material. No heavy metals in significant concentrations were present in the samples of lake sediments. The results of sediment analysis are provided in Appendix H.

Due to environmental concerns, lake dredging, if permitted, would be restricted most likely to the months of July, August and September. Based on this time period, at least two years will be required to remove the anticipated 1,700,000 m³ of material which would be removed if the lake was dredged to 0.5 m. The cost of dredging material to lakeside is estimated to be \$4,400,000 by Miller Contracting, and \$10,550,000 by Fraser River Pile and Dredge. The estimate by FRPD would appear to be most realistic based on most recent construction costs.

Dredging costs received for the Pitt Meadows dyking project and more recently for the Annacis Island sewage treatment plant would indicate a realistic minimum budget cost of dredging is approximately \$4.00/m³. In addition, the discharge pipe length for this application is quite long - up to approximately 4.5 km - and additional difficulties are likely to be encountered including maintaining the lake water levels high enough to provide adequate flotation for the dredge. Therefore a budget figure of \$10,000,000 is recommended. This estimate includes the cost of mobilization and demobilization for two construction seasons. Suction dredges are usually floated into position on the Fraser and Pitt Rivers. However as Hatzic Lake is not accessible by water borne vessels of this size, the equipment must be dismantled and reassembled at the lake. This work is estimated to cost \$1,300,000 which is a significant component within the overall cost estimate. To this estimate must be added GST, engineering and contingencies. For a

project of this magnitude and anticipated duration, engineering is estimated to be an additional \$200,000 and a contingency allowance of 10% is recommended. Therefore, the total budget figure would be \$12,000,000 not including trucking costs.

Should trucking be required within a distance of 6.5 km from the lakeside disposal site, an additional cost of \$4.80/m³ plus contingencies would be incurred. No additional engineering is anticipated as the material would be spread, but not compacted at the disposal site. For a total volume of 1,700,000 m³, the additional cost of trucking and disposal would be \$9,000,000 resulting in a total estimated construction cost of \$21,000,000.

Dredging of the entire lake area is therefore expensive. Partial dredging of the lake may also provide benefit to lake users. Localized areas may be identified where removal of material would allow the lake to be used for longer periods during the year enhancing its recreational potential. Localized dredging would also remove vegetation which, it is understood, is beginning to clog the lake and become a nuisance. Localized dredging at the north end of the lake could possibly change flow patterns to encourage sediment deposition to occur in the deeper areas of the lake. Deposition in these areas could continue with no impact on lake live storage or recreational use. The feasibility of redirecting the sediment to the west side of the lake is beyond the scope of this study.

Environmental concerns exist regarding the removal of material from the lake. The lake is utilized by Chum salmon for spawning and potentially by Coho salmon for rearing. It should be anticipated the environmental agencies would have concerns regarding the timing of lake dredging and any impact on marsh areas available to fish and wild fowl. In addition, dredging may be restricted or prohibited in certain areas where spawning is known to occur (e.g. - the mouth of Draper Creek).

.5 Lagace Creek Dyking

Peak discharges on Lagace Creek during extreme events, cause wide-spread flooding on Hatzic Prairie as illustrated on Drawing 3.3. One option to overcome this recurring problem would be to construct containment works along Lagace Creek to maintain the flow in the creek during all but very extreme events. This could be achieved by constructing dykes along both sides of the creek from Hatzic Lake up to Seux Road to contain 50-year return period average daily flows. A conceptual arrangement for dykes including preliminary location of pump stations and flood-boxes is shown on Drawing No's 4.9 to 4.12 inclusive. Construction of the dykes would prevent flow from the tributary streams from reaching the main watercourse and

therefore to maintain these flows, it would be necessary to construct floodboxes and/or pump stations at the confluence of Lagace Creek and Hatzic Slough with its tributaries. For the purpose of this assessment, it has been assumed that pump stations would only be placed at the major tributaries including Hatzic Slough north of Dale Road, Bouchier Creek, two unnamed creeks north of Youngston Road, Kenworthy Creek and an unnamed creek north of Farms Road. The most significant tributary where no pump station is proposed is Belcharten Creek. A review of the confluence of Belcharten and Lagace Creeks indicates that very little storage would be available for storm-water at this location and therefore the construction of a pump station is not considered viable.

Each of the pump stations would be sized based on the ARDSA criteria and it has been assumed the flow from Belcharten Creek would be rerouted to the west via an overflow diversion into Bouchier Creek which parallels Lagace Creek and rejoins the main stream at Dale Road.

As indicated earlier, the pump stations would be sized to accommodate the ARDSA criteria and each pump station would be provided with a fish pump and screen in order to address environmental concerns. The dykes along the main channel have been assumed to be set back 15 m from the edge of the existing creek. This setback is provided to address environmental concerns and to provide a buffer between the dykes and the creek. This buffer provides additional storage within the dyked channel, enables natural stream processes to continue and reduces the need for significant bank protection works. The requirement for a minimum setback of 15 m is becoming standardized on new dyking projects in British Columbia.

In general, an earth-fill dyke designed to accommodate the 50-year return average daily flow with a freeboard of 0.6 m is provided. As site conditions dictate, the earth-fill dyke has been replaced with a wall to overcome specific local construction constraints, for example adjacent to buildings. In addition, a number of roads and bridges are affected by the proposed works. The bridges at Lagace Creek and Farms Road, Dale Road, Youngston Road and Farms Road adjacent to the new Riverside subdivision require reconstruction. It has been assumed the bridges would be rebuilt to provide a 30 m clear span at each of these locations with the bridge soffit coinciding with dyke crest elevation. In order to accommodate these new bridges, the approaches must also be regraded to the new elevations and the costs of this work are included in the construction estimate. The total cost of completing this work as detailed in Appendix D is estimated to be \$9,862,000. This estimate includes the cost of all construction, including pump stations, floodboxes, bridges and roadworks and the cost of engineering, contingencies and land acquisition.

In preparing these estimates, it has been assumed material for dyke construction will be available from gravel deposits previously placed along the creek banks following creek restoration and sediment being deposited in the creek. This material is not impervious and therefore some seepage will occur through the dyke. Should the construction schedule for these proposed works be such that the work cannot be phased to coincide with the availability of material in the creek, the estimated costs, revised to include the cost of imported fill materials, would be significantly higher.

The dyke crest elevation has been established based on a starting water elevation of 4.0 m in Hatzic Lake. For the majority of the proposed dyke system, the dyke crest elevation is not sensitive to the starting water surface elevation in the lake. However, the area south of the Farms Road Bridge adjacent to the Riverside Road subdivision, is affected by this assumption.

In addition to the dyke crest elevation has been calculated on the assumption that the channel invert is the same as surveyed in 1984 by the Ministry of Environment. This approach may be slightly conservative but is intended to ensure the channel and dykes would provide adequate protection after some sediment deposition in the channel if necessary.

Construction of the dykes would decrease the detention effect of flood storage that is presently occurring on the Hatzic Prairie. This would result in a more rapid rise in lake levels and depending on what other works are implemented, a higher water surface elevation in the lake itself. These higher water elevations would have a negative effect on low-lying properties adjacent to the lake, including the Benbow Street subdivision and properties on the island within Hatzic Lake. In addition, back water effects would adversely impact on the area adjacent to the Chilqua Slough unless a major pump station and floodbox was also constructed where the Chilqua Slough discharges into Hatzic Lake.

The prairie area itself would also not be fully protected against flooding during this 50-year event. During these events, the floodboxes, which connect the drainage channels to Lagace Creek and Hatzic Slough, would be closed due to high water levels in the main channel and all of the pump stations would be working. The pump stations, however, have only been designed to handle 10-year 5 day return period storms and therefore, flooding on the prairie would continue during these events. This flooding however would be less severe than would occur without the dykes as the prairie area is protected from the major flows in Lagace Creek by the dykes.

.6 Raising Roads above Flood Levels

During periods of flooding, vehicular access is severed throughout much of the study area. Questionnaires received from residents stated that some were unable to reach their homes for a number of days during more recent flood events and school children had to be boarded out for several days. This prolonged flooding is confirmed by hydrologic analysis as sections of Farms, Sward and Dale Roads are below the high water level estimates obtained from Flood Routing analysis. To overcome this problem, it has been suggested the major roads in the Prairie area should be raised above high water levels. The highest water level at Dale Road, from Flood Routing, is 4.6 m for the November 1989 storm and the highest lake level is 4.15 m obtained for the November 1990 storm. In order to provide some contingency value over these calculated high water levels for more severe events or the effects of debris build-up or further sedimentation in the creeks a freeboard of 0.3 m is recommended. To provide an allowance for this freeboard a road elevation of 4.9 m is recommended for Dale and Farms Roads and 4.45 m for Sward Road.

Cost estimates for raising various road segments are provided below. These estimates include the cost of replacing affected bridges and fencing, extending existing culverts and restoring driveways. It is assumed that these roads would be built to BC Ministry of Transportation and Highways standards for local and minor roads. The sections of road included in the study are detailed on the attached Drawing No's 4.13 to 4.15.

The following table summarizes the average change in finished grade elevation, the length and estimated cost for each of the segments. Potentially, dredged river gravels could be used as subbase and fill material. This would reduce the estimated costs significantly. Cost estimates are provided for both imported subbase and fill material and dredged river gravels.

Section	Δ in Elevation (m)	Length (m)	Cost Thousand (\$)	
			Imported Subbase	Dredged Subbase
A-B	0.50	406	186	124
B-C	0.75	405	228	136
B-D	0.90	550	351	200
D-E	0.80	1000	600	358
F-G	0.30	170	68	51

Section	Δ in Elevation (m)	Length (m)	Cost Thousand (\$)	
			Imported Subbase	Dredged Subbase
H-I	0.70	190	105	65
E-J	1.00	1243	931	549
E-K	0.45	1200	705	540
E-L	1.00	225	154	85
L-M	0.05	650	299	201
N-O	0.25	190	66	50
TOTAL	-	6.3 km	3,694	2,360

These costs include all materials, equipment, labour and the cost of engineering design and contingencies.

For the section of road between points H and I, Youngston Road, no allowance has been included for raising the bridge across the Hatzic Slough. The bridge deck is currently shown at elevation 4.2 m and would need to be raised 0.7 m to reach the preferred elevation of 4.9 m. As the cost of raising this structure is estimated to be \$180,000 and completing this work would improve access to only two homes, it has been omitted from the scope of work and cost estimates in an attempt to keep the scope and estimated costs within reasonable levels.

4.2 SEDIMENT MANAGEMENT OPTIONS

.1 Introduction

The intent of managing the sediment produced in the Pattison Creek watershed is to reduce the damages caused by sediment deposition and potential overbank flooding along Lagace Creek. From an assessment of the water/sediment processes and history of flooding, the following sediment management options were considered:

- .1 Retain Sediment in the Upper Watershed Watershed Management
Diversion of Pattison Creek to Amber Lake
Debris Torrent Retention Structure

- .2 Remove Material From Alluvial Fan
Annual Excavation (Maintenance)
Annual Excavation From Sediment Trap
Divert Pattison Creek Over Hatzic Prairie

After discussing the above options, the manner of dealing with the flooding problem is presented.

.2 Assess Watershed Management

Most of sediment produced in the watershed above Hatzic Lake comes from landslides and debris torrents along Pattison Creek. The potential for holding material within the landslides or for preventing further landslides is limited. There are several landslide scars on the upper road as shown in Photo 3.14 and these locations should receive local remedial treatment such as culverts through the road, water bars and clearing of deposits from the road. These works would have to be done by hand labour and helicopter as opposed to bringing equipment along the deteriorated road. The cost of constructing these water bars is estimated to be \$55,000.

For future logging, the guidelines established by the B.C. Ministry of Forests (1991) for management of landslide prone terrain should be used. These guidelines were originally introduced in 1991 and therefore were not in use when logging was undertaken in the Pattison Creek watershed. A procedure has been developed for recognizing unstable terrain which involves a thorough office evaluation followed by field evaluation. This evaluation technique was based on earlier work by Howes (1987) who studied the Norrish Creek watershed and by Wilford (1987). Also, the B.C. Ministry of Environment publishes *Guidelines for Watershed Management of Crown Lands used as Community Water Supplies* as prepared by a Task Force of Provincial Government Agencies. The objectives of the guidelines (and the planning process) are to maintain existing water quality and water resources on Crown Lands through management of activities such as forest harvesting. Lagace and Pattison Creeks are not classed as community watersheds, but many of the guidelines and practices used in community watershed management may successfully be applied to these creeks.

It is recommended that the watersheds contributing flow to Hatzic Lake be managed as if they were community watersheds. The Community Watershed Planning Process put forward by the B.C. Ministry of Environment (1980) requires a planning group to prepare integrated watershed management plans for the community watersheds. The planning group nominally includes representatives from government agencies, local government as well as resource user groups. Should future logging be undertaken in the watershed,

the systems approach to forest harvesting should also be encouraged (Conway, 1982). The approach ensures that all the following components of a harvesting system are planned for:

- .1 timber cutting
 - felling;
 - bucking;
 - measuring;
 - limbing; and,
 - topping.
- .2 primary transportation
 - skidding;
 - bunching; and,
 - forwarding.
- .3 loading
- .4 secondary transportation
 - log transportation; and,
 - unloading.

The components involved in the systems approach are defined and clearly presented in the book prepared by Conway (1982).

In regard to future plans for logging, the District of Mission (internal memo) have plans to harvest 630 hectares (TFL26) in the Kenworthy watershed but the harvesting is well into the future. The District does not plan any timber harvesting during their current Five Year Development Plan.

Several local residents are undertaking timber harvesting under the Ministry of Forests Woodlot Licence Program, but these woodlots do not cause changes to flow conditions nor do they contribute sediment into the Pattison/Lagace main lower channels. However, locally, that is, along the minor tributaries, logging does generally increase erosion of hill slopes and sediment is deposited wherever the tributary enters the floodplain of Hatzic Prairie. An example is North and South Hereford Creek which were partially filled with gravel just upstream from the culverts through Sylvester Road. During the November 1990 flood, the flows from these Creeks overtopped Sylvester Road because of partial filling of the channel with sediment.

3 Pattison Creek Diversion To Amber Lake

Upstream from Sylvester Road, there is a low area to the south of Pattison Creek that may be used as a sediment deposition basin (see Dwg. No. 3.1). The area has two shallow lakes, the larger northern lake called Amber Lake. As noted by Golder (1991) -

"an existing fan spills into the saddle towards Amber Lake. The creek (Pattison) is incised into the north flank of this fan. With the construction of a diversion embankment, and excavation of a channel, the creek and hence debris floods could be routed towards the saddle."

In order to assess the feasibility of the diversion, a deflection berm was investigated. The layout of the berm is shown in Drawing No. 4.16 and quantities are listed in Table 4.2. The berm commences at elevation 190 m on the north side of Pattison Creek and runs in a south-west direction for about 400 m to end up within Amber Lake. The maximum height of the berm would be about 20 m measured from the base of the creek. This height allows for a run-up of debris on the upstream side of the berm (Hung et al, 1984). An approximate volume of 80,000 m³ of rock and gravel fill will be necessary for construction of the berm. In order for the debris to move along the berm it will also be necessary to excavate a channel along the upstream face of the berm. The excavation will require rock blasting and about 30,000 m³ will have to be moved. Some of this excavated material could be used as fill material for the berm. The deposition will probably occur in a manner similar to deposition over an alluvial fan with large material dropping out at the top where Pattison Creek is blocked by the berm. This material will have to be bulldozed further down to the vicinity of the lakes which means that annual maintenance is necessary.

There will have to be provisions for the return of flow into Pattison Creek which will require a spillway whose approach will have to be kept open. The structure is estimated to require about 17,000 m³ of rock.

A closure dam will also be required at the end of the deposition reservoir in order to prevent the development of a new river along Hereford Creek and flowing through lower farmland.

A smaller diversion structure was investigated, but the following disadvantages were noted:

- .1 The structure will be overtopped by debris torrents.
- .2 More debris will be deposited along Pattison and Lagace Creeks.

In summary, the deposition of sediment and debris in the lake area would require expensive structures and annual maintenance and the estimated cost for this option is \$3,350,000 with an annual maintenance cost of \$30,000. An annual additional maintenance excavation of 30,000 m³ will be required downstream for the next five years.

The infilling of Amber Lake may not however receive approval from the regulatory agencies. Marsh areas at the north and south ends of the lake may be used by local and migratory wild fowl as well as resident fish stocks. Additional biophysical information, which would be obtained by biophysical field survey and beyond the scope of this study, is required to fully assess the value of these marshes.

.4 Debris Torrent Retention Structure (Barrier)

Debris torrent retention structures have recently been constructed on several creeks flowing into Howe Sound and these are designed to trap the maximum volume of debris that can be transported in a single event (Hung, et al 1987). Because of the large volume of debris generated during the 10 November 1990 debris torrent and the fact that the valley of Pattison Creek is relatively narrow, a high structure would be required which would be expensive. The structure would be similar in character to, although larger than, that constructed on Harvey Creek near the Squamish Highway and this structure is about 15 m high and costs more than \$1.5 million.

The construction of a low barrier would have similar performance to a low diversion berm which would not be effective in that the storage space upstream would fill rapidly and the structure would thereafter serve no purpose.

.5 Annual Excavation

This option assumes no upstream works and involves an annual removal of the material brought into Lagace Creek which ranges from 50,000 m³ in the first year to 10,000 m³ in year five. This sequence then repeats for another five years after an assumed debris torrent event at the end of year five. The excavation is assumed to be spread over the entire length from the Lagace/Pattison confluence to Dale Road and over the ten year period would total 300,000 m³ plus the volume dropped out during the debris torrent. The costs for this option are estimated to be \$2,025,000. This option is the equivalent to maintenance necessary to keep the creek flow capacity at its design magnitude.

The creek invert as shown in Appendix I and cross section which resulted from the restoration work completed by PEP in 1991 provides a good channel to convey the stream flow down to Hatzic Lake. This invert and cross section should be adopted as a reasonable design for the creek. A monitoring program should be put into place and the creek bed checked at intervals to confirm whether significant sedimentation is occurring. When maintenance work is carried out the creek should be cleaned out to return the creek invert and cross section to that which existed following the PEP work in 1991.

.6 Annual Excavation From Sediment Trap

A sediment trap would confine the annual excavation of the majority of the sediment which passes Seux Road to a length of about 1,000 m and could be located in the rather wide reach upstream from the Farms Road Bridge (northern). One of the main reasons for selection of this location is that the sediment size is in the gravel range and this material could be utilized for construction. Farther upstream, such as upstream from the Sylvester Road Bridge, the deposits contain cobbles and boulders and would not be as suitable for construction. Also, this reach is exceptionally steep and sediment will probably move through a sediment trap; therefore, this option was eliminated. Also, small local sediment traps were considered near bridges and several were constructed under the Flood Restoration Project, but these filled rapidly during a single medium flow event and were considered ineffective. Therefore, a sediment trap near Farms Road Bridge (northern) to store up to 50,000 m³ per year is recommended and the costs are estimated to be \$366,000 capital costs plus on going maintenance cost which are essentially the same as for the annual excavation option detailed above.

The advantages of this option are a lower risk of avulsion and a restricted length of channel over which maintenance would be carried out on a regular basis which should reduce interference to fish. Maintenance however would still be required in the creek both upstream and downstream of the trap. Maintenance upstream from the trap is unaffected by the construction of the trap, however the frequency of sediment removal operations downstream from the trap would be reduced.

.7 Divert Pattison Creek Over Hatzic Prairie

Another option for dealing with a rising stream bed flowing over an alluvial fan is to allow the stream to flow down one of two alternate routes having a lower bed level and space for the storage of sediment. When sediment builds up to a predetermined level, the creek is diverted into the other channel which has been cleaned of sediment to provide its full carrying capacity. Once the flow is diverted out into the alternate channel, the first channel can

be cleaned out to reinstate its full carrying capacity. In this manner the creek alignment is changed back and forth between two channels and channel clean out is carried out when no flow is routed through the channel. As an example, the Black Birch fan in New Zealand was managed by construction of two sets of levees and zoning of the space between the levees (Whitehouse and McSaveney, 1990).

A possible diversion path over Hatzic Prairie could be a former creek route approximately in the middle of the alluvial fan (see Drawing No. 3.5). However, a number of considerations would rule out this option:

- .1 A diversion structure would be required in Lagace Creek.
- .2 Land acquisition costs would be high.
- .3 It would be disruptive to local residents.
- .4 Fish passage may be impeded if there are several routes and control structures and fish recovery during diversion could be expensive.

.8 Commercial Gravel Excavation

Four contractors were contacted to establish the commercial viability of gravel extraction operations from Lagace Creek or Amber Lake. The contractors were advised that sedimentation traps would be constructed at either location and at each point an access road would be provided with land available for storage of material extracted during any given year at no charge. In addition, contractors were advised to assume no royalties would be payable.

The contractors were selected as they represented a wide cross section of contractors in the Lower Mainland area who have an interest in gravel sources. The contractors represented the hot mix asphalt concrete industry, ready mixed concrete industry and suppliers of fill material. Each of these groups require different materials and therefore it was considered important that each potential market should be reviewed.

Each of the contractors indicated prior to reviewing the site that they were looking for gravel sources on an ongoing basis and one of the contractors, Steelhead Aggregates, was currently undertaking a gravel removal operation in the Fraser River at Chilliwack. Each of the contractors visited the site, reviewed gradation curves for the material which would be most likely extracted from the sediment trap and reviewed materials currently stored on the banks of Lagace Creek which was removed during the PEP gravel operations in 1991.

None of the four contractors, after visiting the site and reviewing the material, considered gravel extraction from Lagace Creek or Amber Lake on an annual basis to be commercially viable. Each contractor provided different reasons for their conclusion as listed below:

- The hot mix asphaltic concrete contractors indicated the material contained insufficient rock to be of value. The rock in the material could be extracted however this would leave an excess of "sand" which would then have to be disposed of. The limited fraction of rock made this approach undesirable. In addition, significant deposits are available closer to the existing markets. Some wood debris was also present in the material which would increase the cost of any processing work. The hot mix asphaltic concrete contractors considered this material was only suitable for use as general fill.
- The ready mix concrete contractor considered there was too much oversize material to be suitable in their industry. The suitable material could be extracted, however, once again the proportion of unsuitable material which would then need to be disposed of made this approach uneconomic. The current sources of material which lie closer to Greater Vancouver and the major market places have sufficient supply for in excess of twenty years and therefore once again this source is not considered commercially viable. The contractor indicated the material would be suitable for use as road base.
- General fill suppliers shared the concerns of the hot mix asphalt concrete contractor and ready mix concrete contractor. The material is essentially too far away from the market place and several suppliers exist who are much closer than this potential source. In addition, as material would most likely have to be extracted on a yearly basis it was imperative material extracted the year before had been disposed of from the storage area prior to removal of new gravels from the creek. As the market is uncertain this additional requirement would also impact on the attractiveness of this source.

Environmental constraints also exist on any proposed gravel extraction operation. Should the gravel, following removal from the creek, require processing, it is considered this processing would need to be completed remote from the creek where sediment traps could be constructed to prevent fine material being washed into the creek. This requirement would increase the cost of any commercial operation as the material would need to be double handled and would also require additional land to be provided to the contractor. These requirements would therefore increase the initial capital costs for land acquisition and also increase the operating costs.

In conclusion, the material which would be removed from a sediment trap is most likely only suitable for general fill and road construction materials. If used for this purpose no additional processing would be required and therefore additional land would not be required remote from the Creek or Amber Lake. However, the material source is too remote from markets and therefore even without paying royalties this gravel removal operation is not considered to be commercially viable.

4.3 OTHER IMPROVEMENT OPTIONS

.1 Lake Level Controls

In the months of August and September, Fraser River levels will begin to drop, as a result, the flood gates at the dyke open and the lake level drops exposing the mud flats in the lake. This curtails the recreational activities such as boating and water skiing. Therefore the lake users would like to maintain the lake level at approximately 2.44 m through to the Labour Day weekend. This is the same level as the lake is maintained at during the Fraser River freshet period between May and July each year. On the contrary, the farming community in the Hatzic Prairie are concerned that any attempt to maintain the lake level at a higher elevation during this period may increase the risk of flooding in the event of a storm during this period.

As noted in Section 2 the prairie area is low lying and flooding will begin when the water surface level reaches approximately 3.0 m. According to ARDSA criteria, the water surface should normally be maintained at least 1.2 m below the adjacent ground, i.e. at an elevation of 1.8 m, at times when drainage is required. This criteria has not been met in the past within the Hatzic Prairie. In dry periods, when subirrigation would be desirable, the ARDSA criteria call for a freeboard of 0.45 m ie, a water surface elevation of 2.55 m.

Rainfall data for the July - August period were obtained from the Atmospheric Environment Service (AES) for the years 1960 to 1975 and 1985 to 1989 for the Hatzic Lake Station. No records were available for the years 1976 to 1984. Also extreme value analysis was computed by AES for 1 day to 5 day events for return periods of 2, 5, 10, 25, 50 and 100 years. For the design of the lake control structure, a 2 day, 10 year return value of 40 mm/day was used. Using this value, the runoff without using lake storage is estimated to be 12.9 m³/s for AMC II conditions and 5.0 m³/s for AMC I condition. In addition, considering the precipitation records for the month of

August only from a combination of Pitt Polder, Mission West Abbey and Stave Lake Stations as well as Pitt Polder station on its own, the 2 day - 10 year return value is computed to be 28 mm/day and the runoff is estimated at 7.6 m³/s for AMC II and less than 5 m³/s for AMC I conditions.

Considering the above if the control structure can accommodate an outflow of say 7.0 m³/s acting as a weir or by utilizing the available pumping facilities at the dyke, the upstream land owners concerns can, for the most part, be addressed.

Two locations are considered viable for installation of lake level control devices:

- .1 Upstream of Lougheed Highway Bridge
- .2 Upstream side of the floodboxes (existing and proposed) at the Fraser River Dyke.

A structure at the upstream side of the Lougheed Highway Bridge has to be a specially built structure. This structure can also be utilized to control the debris which is reported to obstruct the flow through the CPR culverts.

The proposed structure would consist of five bays each 2.35 m wide formed with steel pipe piles driven in pairs spaced at 2.7 m centres. The piles in each pair are spaced at 2.8 m centres with steel plate stiffener diaphragms. The control sill elevation is at elevation -1.0 m. The two sides of the structure are constructed up to high water level using steel sheet piles driven to stable depths. A sheet pile cut-off wall is also provided for the lake control section to prevent piping and erosion below the control sill. The debris control screens are on the upstream face and the lake level control stop-logs are on the downstream face, which are installed each August and later removed immediately following Labour Day. A fixed walkway provides access to operate stop-logs and for maintaining debris screens. The structure is shown on the attached Drawing No's 4.17 and 4.18. The estimated cost of this structure is \$228,600 including engineering and contingencies.

This option is provided for consideration as it appeared potentially attractive when combined with a debris control structure at this location. However, the cost of this combined structure is more than the cost of a separate debris control structure at this location and a lake level control structure at the existing and proposed floodboxes and therefore this cannot be considered the preferred option. In addition should an August storm occur and the stop-logs at this location not be removed for any reason, the water level in the lake would rise to an elevation of 2.94 m (based on the top of the stop-logs being at 2.44 m). This lake level would, when combined with backwater effects in

the Hatzic Slough and Lagace Creek, undoubtedly result in some minor flooding in the prairie area.

An alternative method of controlling the lake level is by controlling the flow through the floodboxes. This could be achieved by inserting stop-logs at the inlet end of the existing floodboxes. A lifting beam with a pulley block supported by A-frames at each end over the roof of the floodbox would be installed to operate the stop-logs. A simple log-boom chain held by two piles at the entrance of the intake channel to the floodbox-pump station structure will control the debris at this location. The stop-log controls and the lifting beam for both the existing floodboxes and the proposed floodboxes are estimated at \$92,000, including the log boom chain, engineering and contingencies.

If the new floodbox is not built and the stop-logs are inserted in the existing flood-boxes only, the rise in water level would occur more quickly and reach higher elevations than for a new structure upstream of the Loughheed Highway. This results as the possible weir width is only 7.32 m at the existing floodboxes compared to the proposed control structure upstream of Loughheed Highway where the weir width is 11.88 m. Once the new floodbox is built, the weir width could be increased to 14.64 m which would greatly improve this situation. For the existing floodbox only if stop-logs are placed to an elevation of 2.4 m, the lake level would rise to 3.1 m in a 10 year - 2 day August storm if the stop logs are not removed and no pumping is carried out.

One advantage of installing lake level controls at the existing and proposed flood boxes is the location of the pump station(s). Should a storm occur, and the stop logs be left in place, the pumps would operate to help maintain lake levels at, or close to, 2.44 m (8.0 ft) thereby reducing the likelihood of flooding but increasing the operating costs of the pump station(s). Here again, the stop-logs could be lowered to account for weather patterns, and Fraser River levels. Should the stop-logs not be removed for any reason and the 10-year return period storm occur, the existing pump station could handle the entire flow and would pump for approximately 46 hours at an estimated cost of \$4,300. If the new pump station were constructed, the new pumps only would be required to operate for approximately 40 hours at an estimated cost of \$4,260. Should both pump stations operate they would be required to pump for only approximately 23 hours within the 48 hour period which leaves a significant pumping capacity in reserve should a more extreme event occur. These costs reflect a 10-year return period event with total rainfall of 56 mm in two days. These costs are also based on pumping the entire estimated flow at $7.0 \text{ m}^3/\text{s}$. As the weirs and pumps would be set up to allow the summer base flow of approximately $2.5 \text{ m}^3/\text{s}$ to pass without activating the pumps, the costs and hours of pumping are conservative and most likely would be

reduced by approximately 35%. Based on records from Atmospheric Environment Service (AES) the average total rainfall in August over the last 23 years has been 65 mm per August.

In addition an analysis has been completed to determine the approximate time required to lower the lake level in advance of an anticipated storm. The results of this analysis indicated that using only floodbox relief the lake level would require approximately 33 hours to drop from 2.44 m to 2.13 m and after 48 hours the lake level would be approximately 1.9 m. During August the Fraser River is tidal and on an average day the Fraser River level may exceed 2.0 m. Therefore the precise time required to lower lake levels is dependant on river levels in the Fraser River and as a result these times must be considered to be approximate.

Some risk of additional flooding will result if the lake level is held artificially high, at 2.44 m through August, rather than allowing lake levels to drop with the Fraser River levels. Depending on the capital works completed and the operational measures put in place, the lake levels can be controlled with only a low risk of additional flooding. This increase in risk is considered to be low if the new pumping station is constructed. Additional operational measures should include monitoring of lake levels during the control period, keeping a check on advanced weather forecasts and monitoring the Fraser River levels on a regular basis. These checks should be carried out by a responsible person of an Institutional authority and level control instructions carried out in accordance to an Operations and Maintenance Manual specifically prepared for the structure.

Any structure to be installed on the upstream side of the Loughheed Highway would require a vehicular access to the highway and therefore a permit from the Ministry of Transportation and Highways. In addition any work in the wetted perimeter of the channel and any scheme to control water flowing from the lake would require a permit from the BC Environment Water Management Branch.

Yearly drops in water levels which expose the mudflats on Hatzic Lake are believed to act as a natural control for submerged vegetation such as milfoil near shore areas. As mudflat exposure deters the proliferation of submerged vegetation, maintaining the lake levels high through August may encourage submerged vegetation to colonize some near shore areas. As the lake levels would be maintained for a relatively short time period during the year, this potential should be minimal.

Lake water quality is beyond the scope of this study however it is understood samples are taken through the summer to establish the water quality in the

lake and that during those years when the lake level remains high in August, as occurred in 1991, the quality of the lake water can become a concern.

No environmental concerns have been identified which would prevent a lake level control structure from being installed each August.

.2 Erosion of the Property Downstream of CPR Culverts

From historical records it can be seen that the Hatzic Slough down-stream of Loughheed Highway was part of the lake prior to 1882. The railroad and the Loughheed Highway (formerly a wagon road) were actually man-made land bridges across the lake. The first railway trestle was 398 m (1305 ft) long and in 1893 this gap was filled to form a dyke against the Fraser River high flows.

Prior to the construction of the dyke, Fraser River flows circulated back and forth through Hatzic Lake.

The land reported to have been eroded is downstream of the present CPR culverts on the left bank of the Hatzic Slough. This whole area was subjected to the two worst recorded floods in the Lower Fraser Valley, the first in 1894 and the second in 1948. The land in the downstream region of the Hatzic Lake is formed of alluvial deposits of fine sands and silts.

The CPR culverts were installed in 1949 but it is not known when it was first reported that the land opposite the culverts were subject to erosion. Former Inspector of Dykes, K.J. Chisholm, in a letter dated October 15, 1982 addressed to the Canadian Transport Commission, states that since 1969 approximately 200 feet of Lorenzen's property had been lost to erosion. Also he reports that "the rate of erosion is approximately 10 feet per year" and that "the original point of land was almost to the tracks at the time of 1948 flood. This particular flood took off the point of land closest to the railroad and I estimate that at that time 50 to 100 feet of the property was removed when the dyke collapsed in 1948."

From site observations it is quite evident that part of the erosion in this area has been caused by the water flow current impinging directly on the bank of the slough. As noted above the soils in this area are fine sands and silts, and therefore are readily subject to erosion. Also the culverts under the CPR tracks direct the flow such that it impinges directly onto the slough left bank south of the railway tracks.

In order to arrest any possible further erosion, the eroded section of the bank could be stabilized by cutting it to a stable slope and protecting the slope with a layer of filter material and rock riprap.

The eroded bank is approximately 40 m long and the slope depth is approximately 5 m, including a toe key for the rock slope. Allowing the rock protection height to elevation 3.0 m, the slope length is approximately 7.0 m at 2:1 slope.

With 300 mm head at the culvert, the discharge through the 3 m diameter culvert is 12.75 m³/s and the velocity is 1.75 m/s. For direct impingement on the bank, the design current velocity is 3.5 m/s assuming no dissipation of velocity which is conservative. Based on this assumption rock size required would be 300 kg minus (D50 = 50 kg). The estimated cost of completing this work is approximately \$16,000 including an allowance for engineering and contingencies.

Any work in this area would require an approval from BC Environment under Section 7 of the Water Act.

Buildings in the area of bank erosion are set back a substantial distance from the left bank of the Lower Hatzic Slough and are not presently in any danger as a result of the reported erosion. In addition, as erosion continues, the Slough bank will become located further away from the CPR rail culverts. This will result in the velocity of water impinging on the Slough bank becoming less and less with time, to a point where erosion no longer occurs. This point may already be approaching although the bank area in question is not vegetated and some erosion still appears to be occurring. If nothing is done, the area will stabilize and no costs will be incurred.

.3 Local Drainage Problems

Nine landowners described local drainage problems or poor maintenance of ditches as a possible reason for flooding. Two areas had complaints from two parties. These were the Sward Road subdivision and the area south of the CPR tracks. The concerns in these two areas have been combined in the following discussions. After visiting each site and interviewing the local residents, each problem is described in detail and recommendations for improvements where appropriate are provided.

.1 Culvert Size at the Intersection of Seux Rd and Sylvester Rd

The resident of 11599 Seux Road stated in the questionnaire that during fall and spring rains, there had been flooding through her yard. In discussions it was stated that the flood water comes across Seux Road in front of the Mission School property which originates from high ground across Sylvester Road. The flows in the ditch on the west side of Sylvester Road overflow due reportedly to inadequate size of

the culvert. The culvert is 600 mm diameter CSP and is 18 m long. A shallow swale along Seux Road picks up the overflow from the culvert and channels it towards the property in question. Downstream of the culvert, the ditch continues along the west side of Sylvester Road discharging into Hereford Creek. One new residence along Sylvester Road has a driveway crossing the ditch. The culvert size at this location is 760 mm diameter but the ditch is deeper and the culvert will flow under sur-charge conditions during high flows. There are no signs of erosion at this crossing.

From the contour mapping, the catchment area for the culvert under question is 60.5 ha. Peak runoff for a 1 in 10 year storm for this area is approximately $0.8 \text{ m}^3/\text{s}$. The size of culvert to adequately handle this flow is 900 mm diameter. It would appear the complaint of an inadequate size culvert is reasonable and it is recommended that a new culvert of 900 mm diameter be installed. The cost for culvert installation and restoration of the road is estimated at \$7,000 including contingencies.

.2 Oru Creek Culverts at Durieu Road

The resident of 35584 Durieu Road, the owner of Cedar Acres Trailer Park, stated in his questionnaire that the culverts crossing Durieu Road are not aligned with the flow of Oru Creek. On site investigation confirmed that the alignment of the culverts is approximately 70° opposite to the creek flow. There now exist two concrete culverts each 1200 mm diameter. The resident stated that during storm flows the water level can be up to 0.6 m higher on the upstream side than at the outlet of the culverts. However, the water levels have not overflowed Durieu Road. It is understood the upstream property owner had a flooded basement for the last two years. She did not however submit a questionnaire and was unavailable for comment. Downstream of the culvert crossing there is apparently no flooding problem. The downstream area has considerable vegetation with some minor erosion but on the whole is not subject to flooding.

It was suggested installing an additional culvert west of the present culverts as this alternative could be viable to accommodate high flows during peak periods.

The catchment area of Oru Creek upstream of the culverts is approximately 125 ha. Peak flow runoff for a 1 in 10 year storm event is approximately $1.6 \text{ m}^3/\text{s}$. The two culverts have sufficient capacity to handle this peak flow.

The realignment of the present culverts is not recommended. Improvement to the inlet hydraulics however is suggested by constructing a concrete inlet wall to direct flow more smoothly through the culvert. The estimated cost of constructing these new inlet works, including contingency, is \$6,000.

.3 Flooding at Madaris Road and Alberta Road

The resident of 8829 Madaris Road described flooding from a field on the east side of Madaris Road. This flooding occurs particularly in winter when fields are frozen. On site inspection and interview with the land owner confirmed the subject property on the west side of Madaris Road is at a lower elevation than the road as well as the field west of the property. There are no ditches or swales in the area. It is understood, during the November 1990 storm, the resident utilized five pumps to combat floodwaters in his basement. Despite these measures, some minor flood damage did occur and it is understood a damage claim was submitted. The pump outlet pipe had to be extended through the field east of the subject property to a lower area some 60 m away. The property owner stated that the barn adjacent to his house was also flooded and he would like to see a ditch constructed along the west side of Madaris Road to accommodate the flows.

From a review of the contour plan the catchment area is approximately 1.5 ha and the peak flow runoff is $0.02 \text{ m}^3/\text{s}$ for a 1 in 10 year storm event. Flow is directed toward the subject property from both the east and south sides. These flows are not large however since the property is in a depression, water ponds in this area. Storm drainage is required all along Madaris Road, however from observation of the buildings south of Alberta Street, it appears these are very close to the road allowance. It is therefore not feasible without considerable cost, to extend drainage beyond the property south of Alberta Street.

Alternative 1: One alternative would be to construct a ditch along the east side of Madaris Road flowing north towards Chilqua Slough.

Chilqua Slough is approximately 700 m from the subject property and the ditch should flow into this waterway. Quantities for excavation of the ditch along the east side of Madaris Road right-of-way are estimated to be 1400 m³ at an approximate cost of \$22,500, including contingencies. Right-of-way for this ditch already exists all of the way between the Robson property and Chilqua Slough.

Alternative 2: A second alternative is to install 300 mm dia drainage tile near the house running to the north west into a nearby pond on an adjacent property. The drainage tile would be backfilled with drain rock around the subject property and the adjacent property's barn only. The drainage tile would benefit both this land owner and the adjacent landowner. Permission from the adjacent landowner will be required. Costs including contingencies for this alternative is estimated at \$15,500.

.4 Sward Road Subdivision

The residents of 35048 North Sward Road and 35021 Sward Road, located near the north west side of Hatzic Lake both complained of poor drainage in their yards. The site was visited and site observations indicate that 35048 North Sward Road is lower than the adjacent properties to both west and south. These two adjacent properties are not developed and appear to have fill dumped on these lots. There are no ditches or swales along either North Sward Road or Sward Road. In effect this is a subdivision constructed without any allowance for drainage. At 35021 Sward Road, the flooding problem was recently relieved somewhat by building up the grade of the backyard. The flooding problem is now transferred to the adjacent lots. Since the homes in this area were apparently built without proper provision for drainage, the result is inadequate drainage overall.

Alternative 1: To alleviate the problems in this subdivision, storm sewers need to be installed along both streets. Drainage could be directed to Hatzic Lake. The problem is localized as all residents are above the 4 m contour and therefore not subject to flooding from high lake levels. A simple gravity storm water network was sized with costs estimated at \$230,000, including contingencies.

Alternative 2: A second alternative is to install a simple drainage tile system with a similar network as the storm sewer in Alternative 1. There would be no house connections in this system and the excavated area would be filled with drain rock. This alternative with contingencies is estimated at \$105,000.

.5 Front Field Along Sylvester Road Near Davis Road

The resident of 8689 Sylvester Road submitted a questionnaire stating they were having flooding problems in their front field each year. The water was highest during the November 1990 flood. The area was visited and the resident was interviewed. The resident considered the flooding was due to inadequate or no maintenance of the channel in their front field. He also indicated that their neighbour, who are north and downstream of them were not having problems because they had cleaned out their ditch.

From the contour mapping, a large portion of this field is below the 4 m contour elevation. Land below this elevation has been inundated by water during flood events. Therefore no amount of clearing or cleaning out of channels would help in lowering the water levels when lake levels are also high. Clearing the channel would assist flow to recede at a faster rate as lake levels recede. The drainage area is similar to the adjacent property which apparently does not have a problem.

An increase in the size of the culvert and changing the grade of driveway would result in better access to the house. Cleaning out the ditch would help the local drainage problem if lake levels are held lower than 4 m. The estimated cost of installation of a larger culvert, raising the driveway and cleaning out the ditch is \$13,500.

.6 Ditches and Culverts at McKamie and Newton Roads South of CPR Tracks

Two local residents stated in their questionnaires that local ditches and culverts were not maintained.

During the period since questionnaires were submitted, the ownership of the property on Newton Road has changed. The new owner was interviewed and indicated that water levels in their ditch along Newton Road had already reached high levels in August. The ditch along Newton Road has not been maintained and further inspection showed the culvert crossing Newton Road north of the property is in a state of collapse.

The McKamie Road resident stated that the culvert crossing McKamie Road just south of the CPR tracks was not maintained and its invert was too high. This area is completely overgrown with trees and brush and the culvert near CPR tracks cannot be seen. CPR was contacted

to verify the locations of culverts in the area. CPR indicated there was a 0.6 m x 0.9 m culvert 1.1 km east of the Hatzic Slough crossing which is in the same vicinity.

Contour maps indicate that the ditches in this area are below the 4 m contour level. At high lake levels they would flood. If the ditches were properly maintained the flood levels would recede quicker. In discussing water levels, the resident on McKamie Road indicated that the flood levels had reached the 5 metre contour or above in November 1990.

The recommended course of action includes the clearing of the ditches and culverts for the area and the installation of a new culvert at Newton Road. The cost of the work along the road right-of-way is estimated at \$12,000. Cost estimates for cleaning ditches and culverts along CPR right-of-way is \$6,600. Cleaning ditches in the area located in private property is estimated at \$9,000. All prices include a 35% contingency allowance.

.7 Flooding in Benbow Subdivision

The resident of 8413 McKenzie Street in Benbow subdivision had complained of front lawn flooding due to the ditch not being deep enough.

The subject property was visited twice and a letter sent once to inquire further into the problem. The resident was not available on both visits and did not reply to the letter. A telephone interview was conducted with the resident who stated the ditch along McKenzie Street cannot drain properly due to no maintenance. On inspection, the ditch is practically non-existent and it could not be determined in which direction storm water would flow. A large property west of the subject property is in a low lying area adjacent to the lake which floods during high lake levels and forms an extension of the lake.

The best solution to this local drainage problem would be to create a swale along the southern property boundary. The swale would start at the property's front yard at McKenzie then proceed westward into the low-lying area which is presently undevelopable. A cost estimate for the swale is \$1,400, including a contingency allowance.

.8 General

The possible improvements suggested above will certainly improve the local drainage problems encountered in the past during high lake levels. However, we are unable to comment on the benefit/cost aspect of these improvements as they affect only few local properties and do not contribute in any way to alleviate the large scale flooding encountered elsewhere in the study area.

Within an organized area the problems identified in item 1, and partly in item 6 above, would be corrected by the local authority having jurisdiction. It would appear that Canadian Pacific Railway and BC Ministry of Transportation and Highways are the organizations which have some responsibility for drainage in these specific areas. The Regional District may wish to consider approaching these organizations to establish whether they are willing to participate in the necessary corrective measures.

.4 Local Dyking and Flood Protection

After the November 1990 flood, claims were received from twelve properties for major flood damage. A review of the location of these properties indicates they are primarily grouped into two local areas where potentially some localized flood proofing or protection could be put in place to provide protection to the homes. The areas in question are the Benbow subdivision, on the south west side of Hatzic Lake, and the Riverside subdivision which lies between Farms Road and Hatzic Slough south of Dale Road. Recorded flood water elevations for the 1990 flood were:

Location	Water Elevations
Southwest corner of Everglades on Hatzic Lake Island	4.254 m
Sward Road on the north west corner of the Lake	4.318 m
Farms Road Bridge	4.256 m

These recorded high water elevations, which are based on anecdotal information, are considerably higher than the lowest floor elevations surveyed at Benbow Street of 3.85 m and Riverside Road of 4.04 m.

Flood protection could be provided by a variety of means. Which ever option is selected, it is recommended the protection be designed to protect the dwellings against the greater of the highest recorded high water or the highest calculated high water level as seen on the flood routing results. For either criteria a freeboard allowance of 600 mm should also be incorporated. This approach would provide a good level of protection against the recurring floods caused by internal drainage problems, but only minimal protection against a possible breach of the Fraser River dyke during freshet periods.

In general terms, flood protection can be provided by either enclosing the properties behind a dyke or raising the properties above flood levels. The former approach is viable at all locations however it does require consensus to be developed by all those affected. Should one property owner object to the dyke construction this would leave a gap in the dyke and make this approach ineffective. The latter approach may be adopted by individual property owners, however, it may be expensive depending upon the forms of house construction and each property would need to be inspected to confirm whether this approach would be attractive. The estimates provided later in this section are based on the assumption that each house could successfully be raised and that no unusual or unforeseen circumstances exist at the property.

.1 Benbow Subdivision

Out of the twelve properties where major house flooding was reported in November 1990, nine properties are in the Benbow Subdivision. Two more property owners reported minor house flooding in this subdivision.

Reviewing the flood photographs of the November 1990 flooding, it is quite evident that all the properties on the northern side of the subdivision and the northwest corner were flooded. Some have reported as much as 0.6 m (2 feet) of floodwater above the floor level.

This area could be protected from flooding by constructing a low dyke along the northern boundary and around the northwest corner of the subdivision (see Drawing No. 4.19). An earthfill dyke could be constructed with proposed dyke crest of 2 m wide with 2:1 side slopes. The dyke crest elevation would be 4.9 m. Drainage culverts would be provided with flap gates for surface runoff from the subdivision. The length of dyking required is 520 m and is estimated to cost \$147,000.

A cross section is provided indicating the relationship of the proposed earthfill dyke on the adjacent properties. If an earthfill dyke is

considered undesirable, a reinforced concrete wall or granite landscape wall could be built instead. The cost for each of these alternatives is estimated to be \$600/m and \$770/m respectively which would increase the cost of the dyke to \$312,000 or \$400,400 respectively if this approach were used throughout.

.2 Riverside Road - Youngston Road

Another area where the flooding affected more than one party is the Riverside Road-Youngston Road area. It was reported that the properties to the south of Youngston Road and west of Farms Road were flooded up to 1.2 m depth and up to 0.5 m inside the houses. It is reported that the Youngston Road access to the properties was cut off from November 10th to 16th.

Floodwater overflows the banks of the Hatzic Slough, flows south across Youngston Road into the north end of Riverside Subdivision. Also floodwater overflows into the Riverside Subdivision at the west end.

This area could be protected by constructing a dyke around the entire perimeter of the subdivision. On the east and north sides of the subdivision this could be achieved by rebuilding Farms Road and Youngston Road to the required dyke crest elevation. The roadwork would extend to the mid point of Youngston Road at this crest elevation and then drop down to the bridge over the slough to avoid the necessity of raising this bridge. From the mid point of Youngston Road to the slough and all around the west and south edge of the subdivision, the dyke would be formed using a reinforced concrete wall to overcome difficulties caused by the proximity of the buildings to the dyke. Ramps will be required to grade from existing road elevations on Riverside Road, Youngston Road, Farms Road and some driveways to the new dyke crest elevation (see Drawing No. 4.21). This approach would require the reconstruction of Youngston Road to provide accessible ramps over the dyke which would raise the road at the highest point by 1.6 m. The dyke would be built to an elevation of 5.2 m around the entire circumference of this subdivision. The estimated cost of the dyke and road works is \$368,000 based on the use of local dredged materials for road subbase and fill.

.3 Other Isolated Properties

There were several other isolated properties where major flooding of houses, barns or outbuilding have been reported.

They are listed below with proposed lengths of local dyking and costs.

Property Location	Length of Dyking Required	Estimated Costs
10033 Farms Road	220 m	\$74,000
10620 Farms Road	150 m	\$51,000
10814 Farms Road	150 m	\$51,000
11338 Farms Road	365 m	\$123,000
11475 Farms Road	250 m	\$84,000
11744 Stave Lake Road	210 m	\$71,000
12172 Sylvester Road	100 m	\$59,000

From the aerial flood photographs of the November 1990 flood, there appear to be many more properties that were flooded along Farms Road and Dale Road areas. Some have submitted claims and some have not. Flood protection for single dwellings or buildings is very expensive. Individual property owners may wish to consider completing these works over longer periods of time, using their own equipment if available, as fill materials come available at possibly little or no charge.

Should individual owners wish to raise their property above flood levels, the anticipated cost of raising individual buildings lies in the range of \$26,000 to \$95,000 per building. This cost range assumes the bottom of floor stringers is raised to the recommended flood proofing elevation or alternatively new concrete basement floor elevations would be set at this level. These estimates included the cost of raising the building, disconnecting and reconnecting services and modifying the building as required to match the new grades including some landscaping.

The possible range of costs is very high and is dependant on the size of building and type of construction among other considerations. A typical estimate per building could be taken as \$40,000. The high end of this estimate range would be associated with houses constructed on

a concrete floor slab with no foundation walls. For this form of construction consideration must be given to entirely rebuilding the house depending on its existing condition.

From topographic and planimetric mapping it would appear 19 houses are below the established floodproofing elevation in the Benbow Street area and 24 homes are below the required elevation in the Riverside Road Subdivision. Based on a most probable cost of \$40,000 to raise each house, the cost of providing flood proofing in these areas by this method would be \$760,000 in the Benbow Street area and \$960,000 in the Riverside Road area. Should protection be provided only to those properties which submitted damage claims following the 1990 flood these costs would be reduced to \$360,000 for the Benbow Street subdivision. No claims were received by PEP for damage to houses in the Riverside road subdivision although some flooding did occur. Other isolated properties also exist in low lying areas which are not included in these estimates.

This approach to provide flood protection has been successfully implemented in other regions in British Columbia including Marler Drive adjacent to Craigflower Creek near Victoria, B.C. At this site four homes were raised and Photographs 4.1 and 4.2 have been included to illustrate the homes before and after raising and to demonstrate that, with proper planning and design, the buildings can be modified without adversely affecting their appearances. Of the seven houses that were offered cost sharing assistance on Marler Drive, four chose to participate in the program. The average cost of raising these four homes was \$27,556.50 in 1983. The cost of raising the homes was shared between the Province and the home owner. (75% province/25% homeowner).

One advantage of raising homes to provide flood protection rather than dyking local areas is that individual homeowners may proceed with the required work independently. It is therefore not necessary for any homeowners to rely on obtaining consensus with neighbours or to rely on action by the local community as a group.

.5 Land Zoning and Avulsions on Alluvial Fans

As noted in the Terms of Reference, the consequences of a 200 year flood will depend largely on the associated sediment supply - a major flood event occurring simultaneously with a landslide and subsequent

debris torrent will result in sediment deposition and may produce avulsions and much greater flood damage than a flood event in a stable basin.

Avulsions are common on active alluvial fans because fans are created by sediment deposition. The sediment is deposited in the stream channels which reduces the capacity of the stream to retain the flow within its banks. As the channel is filled with sediment the water surface of the creek will rise which ultimately may cause the flow to break out (avulsions). Flow down steep stable channels generally does not cause flooding because the flow is supercritical and has a shallow depth. Flooding and damage to structures adjacent to the steep channels occurs when an avulsion occurs but this frequency of occurrence is difficult to quantify. In fact, as pointed out by Kellerhals and Church (1990), - "there are no broadly accepted standards for classifying hazards on fans". However, attempts have been made by Thurber Consultants (1983) to classify fan surfaces into the following three subareas:

- very active;
- moderately active; and
- slightly active.

Because of the intervention by man into the Lagace/Pattison alluvial fan resulting in roads, bridges and channel relocations, the classification of the fan surface into distinctive subareas becomes difficult. Of concern, however, is the location of a potential avulsion path should this occur. From examination of computations of flood levels and from assessment of deposition zones for debris torrents, as described in the sediment budget, it appears that an avulsion could occur due to the obstruction created by the Sylvester Road Bridge. The avulsion would be caused by the deposition of debris at or just upstream from the bridge resulting in a rapid rise of the creek bed and an overspill to the south, along the east side of Sylvester Road.

The "McConnell Creek - Hatzic Prairie Official Community Plan" shows land zoned as Rural Residential adjacent to Lagace Creek downstream of the junction with Pattison Creek. The BC Environment is withholding approval of development on this land until the risks due to debris torrents and flooding are clarified.

In order for the area to be utilized for rural residential development, the potential for an avulsion should be reduced. One means of reducing this potential is lengthening the Sylvester Road bridge. This would enable the channel at this location to be widened thereby significantly reducing potential for blockage by debris. The bridge should be lengthened to about 50 m, twice its existing length. In addition the channel should be widened by cutting into the left (south) bank of Pattison Creek and lining this bank with riprap. The cost for this option is estimated to be \$1,157,000.

As shown in Appendix I the 200 year instantaneous flow level is below the left (south) bank elevation of Pattison and Lagace Creeks upstream of Suex Road Bridge. Downstream from this structure however the flow would break out of the existing stream channel. In addition at sta. 3486 and 3588 the right bank level is below the 200 year flow level and the left bank is only slightly higher than the calculated water surface elevation. Some local raising of the bank levels at this location is recommended.

In addition it is necessary to establish a local organization or authority with responsibility to maintain the channel. This organization would be responsible to ensure debris and sediment do not build up in the channel which would reduce the flow carrying capacity of the channel. The creek invert and cross section which resulted from the PEP work completed in 1991 provide a good channel to convey the flow down to Hatzic Lake. This invert, as detailed in Appendix I and cross section should be adopted as a design for the creek and a monitoring and maintenance program established to ensure sediment is not permitted to build up in this channel.

In summary, therefore, to enable this area to develop it is recommended the Sylvester Road bridge is lengthened to 50 m, a maintenance organization is established to remove sediment and debris from the channel and the left bank of Lagace Creek is raised as required locally to provide reasonable freeboard.

.6 Existing Pump Station Improvements

As several electrical components have already been replaced and the electrical works are generally in good condition, there is no immediate need for upgrading. The secondary, 480 volt, wiring from the transformers to the pump house is, however in generally poor condition and the insulation on these overhead cables is virtually non-existent in many places. We understand however that BC Hydro have

judged the wiring to be still adequate for its purpose. Based on this assessment, no work has been identified with respect to the electrical installation and with continued maintenance, the installation should provide adequate service into the future.

For the mechanical components of the pump station any recommendations would vary depending upon what other work may be proposed for the catchment area. The pump station currently operates at a very low efficiency, in addition, cavitation occurs. This situation could be rectified by limiting the pump operation to periods when high water levels occur on the Fraser River side of the pump station and high water levels occur within the Hatzic Slough. This is obviously impractical unless a second pump station is constructed to service the Hatzic Slough catchment. Should a new pump station be constructed, it should be designated the primary pump station and designed to pump efficiently at low water levels, both in the Hatzic Slough and the Fraser River. The existing pump station would then act as a secondary pump station during periods of high water levels in the Fraser River and peak flows in Hatzic Slough.

The existing pump station, despite the fact that it is operating outside of its recommended service range, has performed satisfactorily for a long period of time and there is no reason to believe it would not continue to do so. In the event that no additional pump station is constructed, the impellers on the existing pumps could be replaced with new impellers. The new impellers would be designed to operate at peak efficiency within the normal operating range of the pumps. This modification to the existing pumps would reduce the power charges for this installation.

A preliminary cost estimate for replacing the impellers indicates the cost of new impellers would be approximately \$85,000. The potential cost savings in power charges at the existing station, assuming no new station is constructed, would be approximately \$19,500/year where minimum monthly charges are incurred which it is understood occurred in six of the last eleven years, or \$6,000/year when no minimum monthly charges are incurred. In the event a new pump station is constructed, these cost savings would be expected to be reduced to approximately \$5,000/year and \$2,100/year depending on the application of minimum monthly charges.

It is recommended the pump station logs be expanded to include daily pump clock records and that the time of all stage readings taken at the pump station be recorded.

.7 Local Creek Diversion

A creek starting about 800 m north of Scorey Creek on Sylvester Road runs north west to Hatzic Slough. A suggestion has been made that this flow should be directed to the south west to lower Hatzic Slough in order to minimize flooding in the area north of Dale Road.

The unnamed creek is not a well defined creek. It crosses Sylvester Road through a 450 mm culvert and enters the floodplain about 500 m to the west of Sylvester Road (See Drawing No. 4.22).

This creek has a catchment area of approximately 104 ha. Compared to this, the local catchment area of Hatzic Slough north of Dale Road and east of Sylvester Road is 896 ha. (Note: This is not the total catchment of Hatzic Slough). Therefore, the unnamed creek contributes only 11.6 percent of the runoff to the local area.

It has been suggested the unnamed creek be diverted to flow south west, instead of its present flowpath to the northwest, in an attempt to alleviate part of the present flooding problems north of Dale Road.

During flood periods the backwater from Hatzic Lake extends a considerable distance north of Dale Road and the entire area becomes one continuous lake up to an elevation in excess of 4 m (13 ft). During these extreme events, which often occur during periods when the Fraser River is high and the gravity flow outlets to the Fraser River are closed, there would be no benefit obtained by rerouting this minor catchment flow to the south. The flow from this creek would simply combine with the total flood water and produce the same water surface levels as with the existing situation.

During less extreme events when backwater from the lake is not a factor, for example the storm in February 1990, some benefit may be achieved by rerouting the flow from this creek south and west to enter the main drainage system closer to the outlet. This modification would enable the flow from this small catchment to be discharged more quickly and potentially released into Hatzic Lake before the major component of the flow, from the northern section of the catchment reaches this area, thereby potentially reducing the instantaneous peak flow. Considering the size of the subcatchment in relation to the overall catchment this potential benefit is considered to be small.

Diversion of the unnamed creek to the southwest would require construction of a ditch of flow capacity $0.70 \text{ m}^3/\text{s}$ (24.8 cfs). A feasible

ditch alignment is shown on the Drawing No. 4.13 and the length of the diversion ditch is estimated to be 1100 m (3600 ft). This alignment runs along an existing ditch flowing west up to Farms Road and into Hatzic Slough. Another feasible alignment is to excavate a new ditch flowing south to join Hatzic Slough. Obviously this second alignment will cost more than the first.

Approximate costs of the first alignment for enlarging the existing ditch to carry the additional flow are estimated at \$71,500, including an allowance for engineering and contingencies. To this figure, the cost of land acquisition must be added. The proposed ditch requires a bed width of 0.9 m; for a depth of 2.0 m and 2:1 side slopes, the top width is approximately 9 m. Allowing 1 m on either bank requires a ditch right-of-way of 1.21 ha (3.0 Ac). Land acquisition is estimated to cost approximately \$10,000 and therefore the total costs are estimated to be \$81,500 or \$71,500 without compensation for right-of-way.

In order to divert the flows from one creek to another, a permit from BC Environment would be required. Also approvals will be necessary from Federal and Provincial environmental agencies. Insufficient data exists to be able to determine the value of any environmental resource in the existing ditch and therefore it is not possible to predict whether these environmental agencies would have any concerns regarding the proposed work.

5.0 POTENTIAL COMBINATIONS OF MITIGATION WORKS

The preceding sections of the report have provided a description of the project area, the problems which exist regarding flooding and sedimentation and potential mitigation works which, if implemented, could reduce the severity of the problems experienced by local residents. Within the preceding sections the mitigation works were treated separately and the interrelationship of options may not always be apparent. This section of the report is intended to describe various combinations of capital works which could logically be combined within the study area. These combinations are developed simply to provide examples of the approach the community may wish to select and are not intended to represent the only possible combinations or the most beneficial combinations. Numerous other combinations of improvements exist and each combination would be associated with a different cost and resulting benefit. Ultimately the community must determine the combination of works they will seek to implement.

The community must also decide how costs would be distributed within the community. In some cases costs may be borne directly by the landowner receiving the benefit of the improvement works. In other cases, the cost may be a community charge paid by an improvement tax. In the sections that follow, the total cost of the proposed improvements is given without regard for the distribution of costs in the community or to any cost sharing arrangements that might be made with the Provincial and Federal Governments.

5.1 DUPLICATING IMPROVEMENT WORKS

The Terms of Reference call for the investigation of a large number of individual improvement works. Some of these works are mutually exclusive and therefore should one option be constructed this would remove the need for one of the other options investigated. These mutually exclusive items are described briefly below.

.1 Debris Control Structures

Debris control structures were investigated upstream of the existing CPR culverts and at the pump station location, close to the Fraser River. There is no requirement to construct both of these structures. Should the existing CPR culverts remain in service with or without additional culverts through this embankment, the appropriate location for a debris control structure is upstream of these culverts. As this structure would intercept any floating debris accumulated from north of the Lougheed Highway there would be no need to install additional works adjacent to the existing pump station. However, should the existing culverts at the CPR embankment be removed and a bridge installed the need to control debris at this location is eliminated.

The debris under these circumstances would pass through the two bridges at the Lougheed Highway and CPR tracks and continue downstream to the pump station location. Therefore should a bridge be constructed to support the CPR tracks, a debris control structure should be constructed adjacent to the existing pump station.

.2 CPR Culverts

A number of residents in the Hatzic Prairie area have indicated they consider the CPR culverts to be undersized, and as a result these contribute to the flooding which has occurred in the catchment area north of the railway. A review of the hydraulic capacity of these structures indicates that under present conditions, with no improvement to the existing outlet works adjacent to the Fraser River, these culverts are adequately sized. Therefore, unless any proposed improvement works include new floodboxes through the Fraser River dyke there has been no requirement to replace the existing culverts at the CPR embankment identified. This conclusion is based on the understanding that proper measures would be put in place to control floating debris at the existing culverts.

Conversely, should new floodboxes be constructed adjacent to the Fraser River, these existing culverts would not be sufficiently large to enable the increased outlet flow to reach the floodboxes unimpeded. Therefore, under these circumstances additional cross track drainage facilities should be installed at the CPR embankment.

.3 Sediment Traps

Two potential locations have been identified for sediment traps within Pattison and Lagace Creeks. The two locations are identified as within Amber Lake, east of Sylvester Road, and within Lagace Creek itself adjacent to Farms Road. Both of these sediment traps fulfill the same function with some variation in cost, benefit and maintenance requirements. Therefore, should the community wish to implement a sediment trap only one of these locations should be selected.

.4 Dyking at Riverside Road Subdivision

Flood protection for the Riverside Road subdivision is included within two separate capital works estimates; local dyking at this location and dyking of Lagace Creek and Hatzic Slough north of Hatzic Lake. Either of these projects would provide the same benefit to this subdivision and therefore, once again, only one of these improvements works should be implemented.

5.2 OPTION 1 - LOW COST OPTION

Despite the considerable damage to local property and crops which has occurred within the study area as a result of flooding and sedimentation, one option which exists for the community is to complete no capital works. This approach clearly requires the least investment of funds and the local community would have to accept that flooding in the future will be much the same as it has been in the past. Therefore if this approach was to be adopted no section of the community would receive any benefits.

In addition, for the "Low Cost" option to be viable the clean out of Lagace Creek should be completed on an ongoing basis at an estimated cost of approximately \$200,000 per year. To date it is understood this work has been primarily completed by the Provincial Emergency Program at no cost to the local community. However, as noted in a letter dated February 21, 1983 from the then Minister of Environment, Stephen Rogers to the Dewdney Alouette Regional District, the Ministry is not typically involved in recurring maintenance work. The potential exists that whether or not any improvement work is completed, the Provincial Government may conclude the regular clean out of Lagace Creek is a maintenance task and as such is not covered by the Provincial Emergency Program or any other provincial program.

Should the community elect to construct no capital works the Regional District may wish to approach the Ministry of Transportation and Highways (MOTH) and CP Rail (CPR) to request them to complete any works which would appear to fall under their jurisdiction. These works would include:

- Replacing a culvert at the intersection of Seux Road and Sylvester Road.
- Cleaning out culverts under the CP Rail tracks at McKamie Road.
- Cleaning culverts under McKamie Road and replacing a collapsed culvert on Newton Road.
- Replacement bridge on Sylvester Road over Pattison Creek including raising road.

These improvements would result in some benefit to local areas in the community by addressing some of the local drainage problems and potentially allowing development to proceed on land currently zoned Rural Residential.

The cost of the improvements included in this option in this option may be summarized as follows:

- Replace culvert at Suex Road and Sylvester Road \$ 7,000

• Drainage work on MOTH right-of-way south of CPR	\$ 12,000
• Drainage work on CPR right-of-way	\$ 6,600
• Replace bridge over Pattison Creek on Sylvester Road	\$1,157,000
TOTAL ESTIMATED COST	\$1,182,600

5.3 OPTION 2 - FLOODPROOFING HOUSES AND MINOR WORKS

A combination of improvements works which would give some benefit to localized areas in the community and DAID would include a replacement bridge over Pattison Creek, local floodproofing for Benbow Street and Riverside Road subdivisions, improvements to address local drainage problems, construction of water bars in the Pattison Creek catchment and improvements to the existing pump station and installation of a debris control boom chain upstream of the Loughheed Highway.

The cost of these improvements will vary depending on the approach finally adopted. Based on raising nine houses in the Benbow Street subdivision the estimated cost of this work is \$2,051,000.

Based on constructing earth fill dykes around the Benbow and Riverside Road subdivisions the estimated cost of this work would be \$2,206,000.

This work would reduce pump station operating costs, effectively remove local drainage problems and eliminate flooding of habitable areas in participating homes due to internal floods. The beneficiaries of this work would include the Dewdney Area Improvement District (DAID), landowners with land zoned Rural Residential, and home owners who decide to participate in the projects.

These works would provide only a small benefit to the overall community as flooding will continue to occur with much the same pattern and frequency as has occurred in the past. Therefore the low lying areas will continue to experience flooding during both summer and winter storms with the associated damage to crops and buildings which have not been floodproofed. In addition inconvenience will continue due flooding of roads and it would not be possible to control the lake level for recreational users during August.

This combination of options also requires an organization to complete clean out operations in Lagace Creek. Should this become the responsibility of the local community this would result in an additional maintenance cost of approximately \$200,000 per year. The construction of water bars in the Pattison Creek catchment,

for an estimated cost of approximately \$55,000, may provide some reduction in the risk of further debris torrents from this catchment. This investment in capital work at this time may therefore reduce the potential ongoing costs of removing sediment from Lagace Creek should further debris torrents occur in the future.

The existing institutional arrangements appear to be satisfactory if these improvements works are implemented with the exception of maintenance required for drainage works and local dyking in residential subdivisions, and the clean out of Lagace Creek. This work could be completed either by the Dewdney Area Improvement District (DAID) or by the Dewdney Alouette Regional District (DARD) under a specified area function.

The latter approach would appear to be the most logical for two reasons: the benefitting area is relatively small and occupies only a small portion of DAID; in addition, should DARD accept the responsibility under a specified area function it may be possible to issue a bond to cover the cost of constructing these works. Taxes could then be collected over an extended period from the benefitting area to pay for the capital works and for ongoing maintenance.

The improvements to the existing pump station would appear to benefit only the DAID as these improvements would result in reduced operating costs for the District. The DAID could finance these works from their existing reserve fund and would continue to operate and maintain the improved pump station as in the past.

The cost of the improvements included in this option may be summarized as follows:

• Replace culvert at Suex Road and Sylvester Road	\$ 7,000
• Culvert west of 35584 Durieu Road	\$ 6,000
• Drainage of Robson property	\$ 15,500
• Drainage of Sward Road Subdivision	\$ 230,000
• Drainage at 8689 Sylvester Road	\$ 13,500
• Drainage work on MOTH right-of-way south of CPR on McKamie and Newton Roads	\$ 12,000
• Drainage work on CPR right-of-way	\$ 6,600
• Drainage work on private property south of CPR on McKamie and Newton Roads	\$ 9,000
• Drainage at 8413 McKenzie Street	\$ 1,400
• Replace bridge over Pattison Creek on Sylvester Road	\$1,157,000
• Water bars in Pattison Creek catchment	\$ 55,000

- Raise nine houses for floodproofing at Benbow Street \$ 360,000
- or construct earthfill dyke at Benbow Street \$ 147,000
- with dyke at Riverside Road subdivision \$ 368,000
- Improvements to existing pump station \$ 85,000
- Debris control boom chain at Loughheed Highway \$ 93,000

5.4 OPTION 3 - PUMP STATION, CONTROL GATES, SEDIMENT TRAP, FLOODPROOFING HOUSES AND MINOR WORKS

This option would be identical to Option 2 identified above except for the deletion of improvements to the existing pump station and the addition of a sediment trap within Lagace Creek adjacent to Farms Road, a new pump station and control gates at the pump station to enable lake levels to be maintained at 2.4 m throughout August. The cost of this work is estimated to be \$4,332,600 or \$4,487,600 depending on the approach for floodproofing houses in Benbow and Riverside Road subdivisions.

The beneficiaries from this combination of improvement works would include landowners with land zoned Rural Residential, landowners adjacent to Lagace Creek downstream of the sediment trap, the homeowners who participate in the local drainage and floodproofing schemes, farmers with land or buildings in low lying areas, the Everglades Resort, and recreational users of the lake. This group of beneficiaries is therefore quite extensive and includes a large proportion of the people and property in the catchment area.

The new pump station would increase the existing discharge capacity of the pumps from 6.06 cms to 14.4 cms. Should the new pump station be constructed, which would then become the lead pump station for the catchment area, it would appear that improvements to the impellers on the existing pumps would no longer be worthwhile.

The advantages of constructing a new pump station are numerous. During summer storms the increased pumping capacity would significantly reduce the level of flooding which has been experienced in the catchment area. This is demonstrated on the forward flood routing diagrams for June 1981 and July 1983. During the summer months, when the Fraser River levels are high, the catchment is entirely dependant on the pumps to discharge storm water flows from Lagace Creek and Hatzic Slough. The June 1981 storm is fairly representative of a summer storm and the additional pumping capacity is able to generally maintain water levels below those elevations where flooding typically occurs throughout this storm.

An exception to this is in the Hatzic Prairie area where water levels would rise to approximately 3.1 m and therefore some minor flooding would occur. However, the duration over which the high waters rise above 3 m, is reduced to approximately two days which would satisfy the ARDSA criteria for the duration of the flooding, but it would not however satisfy the ARDSA criteria for freeboard of 1.2 m.

For the more extreme event of July 1983 some flooding would have occurred within Hatzic Prairie and at the Everglades Resort if the new pump station had been in operation. However, flood levels are controlled below those elevations where flooding would occur in residential subdivisions and the duration of flooding within the prairie and the Everglades Resort is significantly reduced.

During winter storms however, if the Fraser River levels are high and the floodboxes are closed, the new pump station would not have sufficient capacity to handle the storm runoff. Extensive flooding would still occur during winter months although the peak flooding elevation in Hatzic Lake would be reduced and the duration of flooding throughout the catchment would also be reduced.

Construction of the new pump station and control gates would also enable lake levels to be controlled through August at an elevation of 2.4 m. With the additional pump station in place the risk of flooding, which results from maintaining water levels at 2.4 m, is considered to be low as the pump station has capacity well in excess of the 10 year 2 day August storm. From an operational point of view the provision of three additional pumps provides DAID with significant extra flexibility. Should one pump require maintenance or malfunction for any reason, the impact on the overall pumping capacity is relatively small.

In addition, as the existing pump station has provided 42 years of service, and is now quite old, the addition of a new facility would provide the community with a reliable pumping facility for many years to come.

The maintenance required for the sediment trap is essentially no different from the "Least Cost" approach where sediment is deposited throughout the existing stream. The sediment trap would provide a convenient means of concentrating some of the channel maintenance function to the area immediately upstream of Farms Road. The sediment trap would reduce the maintenance requirements downstream of Farms Road and would also reduce the risk of avulsion between Dale Road and the upper Farms Road bridge.

Additional operating and maintenance costs would also be incurred as a result of the new pump station. These costs have been estimated to be approximately \$21,000 per year for power consumption and \$3,000 per year for maintenance. These costs when added to the annual cost for sediment removal total an additional \$24,000 per year over and above the cost of operating and maintaining the existing pump station.

The existing institutional arrangements for these improvements would appear to be inadequate and therefore DARD may wish to consider accepting the overall responsibility for drainage in the area under a specified area function. This would enable the ongoing operation and maintenance of the drainage facilities to be the responsibility of one organization and would also enable DARD to consider obtaining funding using a bond issued through the Municipal Finance Authority.

As the removal of gravels from a sediment trap is not considered to be commercially viable it is recommended a site for disposal of the sediment be identified to assist with the long term disposal requirements of creek maintenance. The raising of roads within the prairie area would appear to provide the most beneficial use of this material for the entire community. Alternatively, the material could be donated to land owners within the vicinity who appear willing to accept this material provided it is delivered at no charge.

The cost of the improvements included in this option may be summarized as follows:

• Replace culvert at Suex Road and Sylvester Road	\$ 7,000
• Culvert west of 35584 Durieu Road	\$ 6,000
• Drainage of Robson property	\$ 15,500
• Drainage of Sward Road Subdivision	\$ 230,000
• Drainage at 8689 Sylvester Road	\$ 13,500
• Drainage work on MOTH right-of-way south of CPR on McKamie and Newton Roads	\$ 12,000
• Drainage work on CPR right-of-way	\$ 6,600
• Drainage work on private property south of CPR on McKamie and Newton Roads	\$ 9,000
• Drainage at 8413 McKenzie Street	\$ 1,400
• Replace bridge over Pattison Creek on Sylvester Road	\$1,157,000
• Water bars in Pattison Creek catchment	\$ 55,000
• Raise nine houses for floodproofing at Benbow Street	\$ 360,000
• or construct earthfill dyke at Benbow Street	\$ 147,000
• with dyke at Riverside Road subdivision	\$ 368,000
• Debris control boom chain at Lougheed Highway	\$ 93,000
• Sediment trap in Lagace Creek	\$ 366,300
• Control gates at pump stations	\$ 92,000
• Pump station	\$1,908,300

5.5 OPTION 4 - FLOODBOXES, CPR CROSSING, PUMP STATION, CONTROL GATES, SEDIMENT TRAP, FLOODPROOFING HOUSES AND MINOR WORKS

This option is very similar to Option 3 with the addition of extra floodboxes at the outlet works and cross-track drainage structures at CP Rail to allow water to discharge more quickly to the Fraser River when the water levels in the Fraser River are low and the floodboxes are open. This combination of options would not require the boom chain to be installed at the Loughheed Highway unless additional cross track drainage is provided by means of culverts rather than a bridge at this location. The cost of these improvements is estimated to be \$6,160,100 or \$6,315,100 depending on the approach adopted in the Benbow and Riverside Road subdivisions for floodproofing houses. The estimated cost of operating and maintaining the system is essentially the same as Option 3 which is approximately \$224,000 per year over and above the cost of operating and maintaining the existing pump station.

The beneficiaries of this combination of works are essentially the same as in Option 3. The added benefit to the community of these extra outlet works is quite small. As shown on the forward flood routing diagrams for winter storms some floodbox relief is realized on the falling leg of the flood hydrograph. These improved outlet works would have effectively no impact on the maximum flood stage, however the duration of flooding for winter storms would be reduced.

As more water would be discharged by gravity flow during winter months the power consumption of the pump station would be reduced with an associated reduction in power consumption costs. This reduction in power consumption costs would approximately offset the additional operating and maintenance cost associated with the improved outlet works. The reduction in power consumption costs however would appear not to be sufficient to justify the expenditure of \$1,920,500 on capital works. If however senior levels of government and CP Rail were to fund the construction of these improved outlet works the community would receive an ongoing benefit from this reduction in operating costs.

The floodboxes provide no benefit during the summer storms when the Fraser River levels are high as they will not operate during these periods.

The cost of the improvements in this option may be summarized as follows:

• Replace culvert at Suex Road and Sylvester Road	\$ 7,000
• Culvert west of 35584 Durieu Road	\$ 6,000
• Drainage of Robson property	\$ 15,500
• Drainage of Sward Road Subdivision	\$ 230,000
• Drainage at 8689 Sylvester Road	\$ 13,500

• Drainage work on MOTH right-of-way south of CPR on McKamie and Newton Roads	\$ 12,000
• Drainage work on CPR right-of-way	\$ 6,600
• Drainage work on private property south of CPR on McKamie and Newton Roads	\$ 9,000
• Drainage at 8413 McKenzie Street	\$ 1,400
• Replace bridge over Pattison Creek on Sylvester Road	\$1,157,000
• Water bars in Pattison Creek catchment	\$ 55,000
• Raise nine houses for floodproofing at Benbow Street	\$ 360,000
• or construct earthfill dyke at Benbow Street	\$ 147,000
• with dyke at Riverside Road subdivision	\$ 368,000
• Sediment trap in Lagace Creek	\$ 366,300
• Control gates at pump stations	\$ 92,000
• Pump station	\$1,908,300
• Bridge at CP Rail crossing	\$1,105,000
• Floodboxes (built in conjunction with pump station)	\$ 815,500

5.6 OTHER MAJOR WORKS

Major work items which have not been included within any of the options described above include the diversion of sediment into Amber Lake, dredging Hatzic Lake, dyking Lagace Creek and Hatzic Slough north of the lake and raising roads in the low lying areas. Each of these possible improvement works has been shown to be quite expensive and the resulting benefit to the community would not appear to justify this level of expenditure on these capital works projects.

6.0 CONCLUSIONS

6.1 PROJECT IMPLEMENTATION

The study examines the flooding and sedimentation problems within the Hatzic Prairie catchment area, and investigates a number of possible improvements for the area. Capital costs are provided for the improvement works together with an estimate of ongoing operating and maintenance costs where these are significant. The study provides a basis for further decisions to be made, and the completion of the study represents the first phase of a possible improvement project. The next phase is for the local community to review the available options and decide on what course of action it wishes to pursue. To assist in this process, it may be desirable for DARD to constitute a committee which would be responsible for defining a realistic combination of measures that would best meet the needs of the local community. The committee could also be responsible for identifying an approach that could be made to other levels of government to obtain assistance in cost sharing and project implementation.

The actual benefit of any one option will be perceived differently by the residents in the study area depending on their property location, the severity of existing problems, and the potential cost implications of completing any of the works identified. Prior to proceeding with any construction project, it is important that the local community review the alternative works included in this report and obtain an understanding of the resulting impact of completing these works. This review process, it is envisaged, would provide the community with an opportunity to build consensus regarding the most preferred combination of improvement works. Conceivably, this combination of improvements would contain works which would benefit only one area within the overall catchment. However, by selecting an appropriate combination of improvement works, it may be possible for the community to prepare a balanced plan of improvements which addresses the overall concerns of the community.

Having selected the desired set of improvements, the community will need to review existing institutional arrangements to determine whether the existing institutions are able to provide the ongoing operating and maintenance required by the new works or whether a new framework to encompass the whole benefiting area is required.

Finally, and perhaps most important, the community will need to establish a source of funding for the desired improvements. It is anticipated that senior levels of government will be approached to determine whether any contributions will be forthcoming from the Provincial and Federal Governments.

One option which is available to the community is for the Dewdney-Alouette Regional District to obtain authority to accept a specified area function for drainage and sediment management in this catchment. The specified area would incorporate all of those land owners who benefit from the improvement works selected by the community. The benefit of this approach is perhaps two-fold. Firstly, an institution is identified which would be responsible for constructing, operating and maintaining the works in the area. In addition, it is understood DARD may be able to approach the Municipal Finance Authority of the Provincial government to obtain assistance in long-term financing of any capital works. It is understood the most recent issue of municipal bonds provided an interest rate 9.5%. Therefore, should DARD wish to obtain for example \$1,000,000 in financing by issuing a municipal bond for 15 years at 9.5%, this would cost approximately \$140,000 per year depending on the conditions of bond repayment. DARD would then be required to cover the cost of this bond, and any on-going operating and maintenance costs, by obtaining taxes from local residents. In the example quoted above, the cost of \$140,000 per year represents an increase of 132% over the revenue taken in by DAID for the 1990 fiscal year of \$60,394. The actual cost would clearly vary with the cost of capital works constructed and any cost sharing obtained from senior levels of government, and this example is provided simply as a reference bench mark. The variation in cost per year will vary linearly with the amount of any borrowing. Annual operating and maintenance costs for the proposed improvements would be in addition to any capital costs.

It is also important to note that all construction cost estimates are expressed in 1991 Dollars. These cost estimates must be adjusted for inflation as required when project funding is being considered.

6.2 COST SHARING OPPORTUNITIES

Historically, a number of programs have existed for the improvement of drainage or protection against flooding. These programs have been either funded independently by the Province, or jointly by the Provincial Government, Federal Government and local groups. These programs, a number of which no longer exist, may provide the community with a valuable precedence with which to approach senior levels of government for cost sharing assistance. A brief description of these programs is therefore provided for background and information purposes.

Three programs which have been funded in part by the Provincial Government include the River Protection Assistance Program, the Flood Protection Assistance Program, and the Dyking Program. Each of these programs was intended to provide assistance to local governments and individuals to enable these groups to construct flood protection works and bank protection works.

Owners and the Province, and in each case the Province paid 75% of the construction costs. Following completion of construction, the Province required the

land owner or local community to accept the full responsibility for operating and maintaining the works.

These programs have now been amalgamated into a single Flood Protection Assistance Program which, it is understood, is still in effect. However, although this program still exists, it is understood no funding is currently set aside for the program.

The Ministry of Transportation and Highways have also been involved in at least one cost sharing program for flood protection works. This local program is described earlier in this report and is associated with the raising of buildings above a flood level on Craigflower Creek near Victoria. The cost sharing agreement for this project was based on the River Protection Assistance Program and therefore once again a precedent does exist where the Provincial Government has contributed 75% of the capital works. For this particular project, there was no on-going maintenance requirement involved, however as the agreement was based on the River Protection Assistance Program it is most probable that MOTH would not have accepted any responsibility for ongoing operating and maintenance costs.

The Provincial and Federal Governments jointly cost share with local municipalities to fund drainage improvement projects for agricultural lands under the Agriculture and Rural Development Subsidiary Agreement (ARDSA). Within this program, it is necessary for the benefitting area to satisfy a benefit/cost analysis which demonstrates the benefit achieved by constructing the works exceed the capital cost investment. ARDSA projects are funded with a 37.5% contribution from the provincial government, a 37.5% contribution from federal government, and a 25% contribution from municipalities. Once again, the local community is responsible for all ongoing operating and maintenance costs. It is understood the ARDSA program is currently being phased out.

The remaining program is the Fraser River Flood Control Program, which is jointly funded by the provincial and federal governments. Each level of government cost shares equally for the construction works, while the local municipality is required to provide all of the necessary right-of-way and to accept responsibility for all ongoing operation and maintenance.

The Fraser River Flood Control Program is limited to providing protection against flooding from the Fraser River and its main tributaries. Once again, the area is subjected to a benefit/cost analysis and for the project to proceed, the value of benefits must exceed the estimated construction costs. During the initial years of the Fraser River Flood Control Program (FRFCP), provision was made to accommodate internal drainage with the construction of pump stations. This provision was later deferred and currently, any project completed by the FRFCP will be limited to the construction of dykes with flood-boxes to allow gravity flow from internal rivers to flow through the dyke when river levels in the Fraser River are low. In areas where

a requirement for a pump station is identified, the local municipality would be responsible for constructing the pump station. Where pump stations are combined with floodboxes, the municipality is required to pay the incremental extra cost of the pump station structure over and above the cost of building the floodboxes in isolation. The Fraser River Flood Control Program is currently scheduled to expire in 1995 and at this time, it is understood no extension to the program is anticipated.

Therefore in conclusion, two programs are currently in existence where senior levels of government cost share with local municipalities for the provision of flood control works. These programs are the Fraser River Flood Control Program and the Flood Protection Assistance Program. These programs are currently either scheduled to expire in the next few years, or have no funding assigned to them. In addition, senior levels of government have not previously accepted any responsibility for ongoing operating and maintenance costs. These costs have traditionally been borne by the local community.

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