DRAFT REPORT

ELWHA RIVER WATER QUALITY MITIGATION PROJECT PLANNING REPORT

Prepared for
M&I Water Users Group Review

January 14, 2002

URS

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Mission Statements

U.S. Department of the Interior

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

Bureau of Reclamation

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

Tables

TABLE OF CONTENTS

Figures

TABLE OF CONTENTS

Appendices

List of Acronyms

BOR	Bureau of Reclamation
CFR	Code of Federal Register
cfs	Cubic feet per second
D/DBPR	Disinfection/Disinfectant By-Product Rule
DBP	Disinfection By-Products
DE	Diatomaceous Earth
DOE	Department of Ecology
ED	Electrodialysis
EDR	Electrodialysis Reversal
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FERC	Federal Energy Regulatory Commission
GWI	Ground Water Under The Influence of Surface Water
HAA	Haloacetic Acid
hp	Horsepower
ICR	Information Collection Rule
IESWTR	Interim Enhanced Surface Water Treatment Rule
INA	Information Not Available
IOC	Inorganic Chemicals
MCL	Maximum Contaminant Level
MCLG	Maximum Contaminant Level Goals
MDD	Maximum Day Demand
MF	Microfiltration
mg/L	Milligram per liter
mgd	Million gallons per day
MWH	Montgomery Watson Harza
MRDL	Maximum Radionuclide Detection Limit
MRDLG	Maximum Radionuclide Detection Limit Goal
NAVD	North American Vertical Datum
NF	Nanofiltration
NMFS	National Marine Fisheries Service

List of Acronyms

NPDES	National Pollution Discharge Elimination System
NPDWR	National Primary Drinking Water Regulations
NPS	National Park Service
NSDWR	National Secondary Drinking Water Regulations
NTU	Nephelometric Turbidity Units
pCi/L	Picocurie per liter
psi	Pounds per square inch
PUD	Public Utility District
RO	Reverse Osmosis
SDWA	Safe Drinking Water Act
SOC	Synthetic Organic Chemicals
SWTR	Surface Water Treatment Rule
TCE	Trichloroethylene
TCR	Total Coliform Rule
TNCWS	Transient Noncommunity Water System
TOC	Total Organic Carbon
TSS	Total Suspended Solids
TT	Treatment Technique
TTHM	Total Trihalomethanes
UF	Ultra Filtration
URS	URS Corporation
USACE	United States Army Corp of Engineers
USGS	United States Geological Survey
UV	Ultraviolet Light
VOC	Volatile Organic Compounds
WAC	Washington Administrative Code
WDFW	Washington State Department of Fish and Wildlife
WDOH	Washington State Department of Health

List of Elwha River Mile Landmarks

Landmark	River Mile
City of Port Angeles Ranney Collector	2.8
One Lane Bridge	3.0
Middle of WDFW Rearing Channel	3.1
Elwha River Road Bridge	3.2
City of Port Angeles Diversion Dam and Industrial Surface Water Intake	3.48
State Highway 112 Bridge	4.0
Elwha Dam	4.8
US 101 McDonald Bridge and USGS Gaging Station	7.6
Glines Canyon Dam	13.5

Executive Summary

This Project Planning Report presents the results of the Elwha River Restoration Project water mitigation evaluation and provides additional planning to further develop the findings and recommendation of the Final Environmental Impact Statement. Public Law 102-495, the Elwha River Ecosystem and Fisheries Restoration Act of 1992, authorized the Secretary of the Interior to fully restore the Elwha River ecosystem and native anadromous fisheries. The Elwha River is located on the Olympic Peninsula in northwest Washington State. In order to accomplish this goal, two hydroelectric dams on the river, Elwha and Glines Canyon, will be removed. The accumulated sediments deposited behind the dams will be allowed to wash downstream, significantly affecting river water quality for a period of time.

The following entities currently use the Elwha River as a water supply and are expected to be negatively impacted by the removal of the dams and subsequent release of sediments:

- City of Port Angeles Residents (Municipal Water Supply)
- City of Port Angeles Industrial Consumers
- The Washington Department of Fish and Wildlife (WDFW) Fish Rearing Facility
- Lower Elwha Klallam Tribe Fish Hatchery
- Dry Creek Water Association
- Elwha Place Homeowners Association

Public Law 102-495 states that the Secretary is to take actions as necessary to implement:

> "…protection of the existing quality and availability of water from the Elwha River for municipal and industrial uses from possible adverse impacts of dam removal."

The National Park Service, its technical support, the Bureau of Reclamation, and its engineering consultant, URS Corporation, have developed this Elwha River Water Quality Mitigation Project Planning Report (Project Planning Report) to address the protection of the City of Port Angeles residential and industrial consumers, the WDFW fish rearing facility, and the Lower Elwha Klallam Tribe fish hatchery. This Project Planning Report does not address possible mitigation measures for the other users of the Elwha River including, the Dry Creek Water Association, the Elwha Place Homeowners Association, and various individual private wells located along the river. The mitigation measures for these water users are being developed directly between each of these individual users and the National Park Service. The Dry Creek Water Association has expressed interest in potentially being included in any mitigation measures proposed for the City of Port Angeles municipal supply. Inclusion of the Dry Creek Water Association into any of the mitigation measures developed in this report would not significantly affect the feasibility or cost due to the limited water requirements of this community and its relative proximity to the City's water supply system.

This Project Planning Report has developed a series of alternatives to protect the City of Port Angeles' municipal supply, and another series of alternatives to protect the industrial and fisheries supply that includes Daishowa America, the WDFW fish rearing facility, the Lower Elwha Klallam Tribe fish hatchery, and the reserve industrial water supply capacity formerly available for use by the Rayonier Paper Mill. A description of the recommended mitigation measures follows.

Water Supply Requirements

The water supply needs for the entities evaluated as part of this report are presented in Table ES-1.

TABLE ES-1
WATER SUPPLY SUMMARY

	Current Water Right	Current Average Daily Demand	Current Maximum Daily Demand	Mitigation Period Maximum Daily Demand	Current Contract Amount with City
Port Angeles Municipal	32.2 mgd (50 cfs)	3.4 mgd (5.3 cfs)	10.0 mgd[1] (15.4 cfs)	10.6 mgd (16.4 cfs)	--
Port Angeles Industrial	96.8 mgd (150 cfs)	--	--	--	--
Daishowa	--	9 mgd (13.9 cfs)	14 mgd (21.7 cfs)	14 mgd (21.7 cfs)[2]	20 mgd (31 cfs)
WDFW	--	11.4 mgd (17.6 cfs)	21.3 mgd (33 cfs)[2]	14.2 mgd (22 cfs)[2]	32.3 mgd (50 cfs)
Reserve	--	--	--	--	44.5 mgd (69 cfs)
Tribal Fish Hatchery	Undefined	Varies by month	7.4 mgd (11.4 cfs)[2]	12.4 mgd (19.2 cfs)[2]	--
Totals	**129 mgd (200 cfs)[3]**	**23.8 mgd (36.8 cfs) [3]**	**52.7 mgd (81.5 cfs)**	**51.2 mgd (79.3 cfs)**	**96.8 mgd (150 cfs) [3]**

Notes:
1 The City of Port Angeles will use the storage water within its water distribution system to supply the current maximum daily demand.
2 Based on Maximum Monthly Demand
3 Does not include Tribal water

The Tribe has identified that their maximum long-term hatchery water supply need will be up to 18.6 mgd (28.8 cfs). With the addition of the Tribe's long-term hatchery water supply to the City's municipal and industrial water right amounts (129 mgd/200 cfs), the total long-term water supply need is 148 mgd (228.8 cfs).

Municipal Mitigation Measures

This Project Planning Report recommends the construction of a conventional treatment plant that utilizes a microsand ballasted flocculation process to treat water from the City's existing collector well (Ranney well) or surface water before distribution. The complete treatment process will include the following:

- Pump water from the existing collector well or surface water to a new treatment facility
- Add coagulation chemicals such as aluminum sulfate (alum) and polymer, adjust pH and alkalinity, if necessary
- Add a microsand to assist in the flocculation of suspended sediment
- Allow the flocculated solids and microsand to settle, separate microsand and recycle through the process, dispose of treatment residuals in settling basins

- Filter water through dual or multi-media gravity filter
- Use sodium hypochlorite as a primary disinfectant and if necessary for TTHM control, chloramines as a secondary disinfectant prior to distribution to consumers

This treatment process allows for the greatest flexibility and reliability for treating a source water of unknown or highly variable quality. This treatment process also meets all applicable Federal and Washington State drinking water treatment requirements.

There are recommended treatment plant locations at this time based on a cursory review of all the identified sites. Both the southern portion of the landfill site and the property immediately east are potential candidates for the development of a treatment plant. The landfill site is already owned by the City and thus would not require purchase of property from a private party whereas the second location would involve a purchase of property. For the purposes of this report, it was assumed that the City landfill site would be the recommended treatment plant location. A siting study to further examine the environmental issues, geological and geotechnical issues, and property acquirement and cost issues will be required before a recommendation can be finalized.

The City's existing Ranney collector has experienced a decrease in yield over the years that has been attributed to river migration. To mitigate possible further yield reduction, the City's existing collector well will be supplemented with a cross-connection to the proposed industrial and fisheries intake to provide added reliability for the municipal system. The cross connection will have the flexibility to draw water from the industrial and fisheries mitigation system described in the next section, and also draw surface water when the water quality impacts of dam removal have subsided.

Industrial and Fisheries Mitigation Measures

Surface water is recommended as the primary source for industrial and fishery use. The same system will also provide a secondary source for City municipal use. The water will be diverted from the river, clarified, and supplied to users for direct use (WDFW and Tribe fish facilities) or additional treatment (Daishowa and the City municipal). During periods of low turbidity, water for the Tribe and WDFW fish facilities may also be supplied or augmented using the existing Tribe infiltration gallery and the Tribe and WDFW vertical groundwater wells.

Surface water will be diverted using a new diversion and intake facility located at approximately the same location as the existing City diversion dam. The facilities will be designed to divert the long-term demand of 148 mgd (228.8 cfs). The water intake facility will provide limited gravity settling to promote removal of large suspended solids. Sediments trapped by the intake facilities will be periodically flushed downstream through gated openings. Both the intake and the diversion will be designed to meet fish protection and passage requirements.

51.2 mgd (79.3 cfs) of surface water that has been determined to supply the municipal, industrial and Tribal hatchery water supply needs will be treated using a conventional clarification process with chemical addition and settling before delivery to the individual users. Conventional chemical addition and clarification treatment will be provided to reduce turbidity to at least 20 NTU for use at the WDFW rearing channel and Tribe hatchery. Additional treatment for Daishowa and municipal drinking water will be required.

An inter-governmental agreement would be required to define the relationship between the City and the Tribe for operation of the shared intake. After the dam removal and sediment erosion period the Tribe would be able to continue using their existing infiltration gallery and well water supply with supplemental water available from the surface water intake, to satisfy additional Tribal hatchery water requirements.

An estimate of capital and operation and maintenance cost for the municipal, industrial and fisheries mitigation measures is presented in Table E-1.

Table E-1
WATER QUALITY MITIGATION MEASURES PROJECT COST

	Capital Cost	Annual Operation and Maintenance Cost
Municipal Water Quality Mitigation	$17,195,000	$958,000
Industrial and Fisheries Water Quality Mitigation with Treatment for Maximum Water Demand During Mitigation Period	$42,800,000	$1,000,000
Industrial and Fisheries Water Quality Mitigation with Treatment for Remaining Industrial Water Capacity	$72,200,000	$1,400,000

*Costs based on first quarter of year 2001

1.1 PURPOSE AND SCOPE

The purpose of this Elwha River Water Quality Mitigation Project Planning Report (Project Planning Report) is to evaluate various water intake, treatment and supply alternatives to ensure the preservation of water quality for the City of Port Angeles (City), Washington Department of Fish and Wildlife, and the Lower Elwha Klallam Tribe (Tribe) during the Elwha River Restoration Project. The Elwha River Restoration Project includes the removal of the Glines Canyon Dam and Elwha Dam from the Elwha River. The Elwha River is the current source of water for the City of Port Angeles' municipal and industrial needs, as well as the source of water for the Tribal fish hatchery and the State rearing channel. The removal of the dams will adversely impact the raw water quality of the Elwha River. Public Law 102-495, the Elwha River Ecosystem and Fisheries Restoration Act, passed in 1992, directs the Secretary of the Interior to restore the anadromous fisheries and the ecosystem of the Elwha River. The Act states that the Secretary is to take actions as necessary to implement:

> "…protection of the existing quality and availability of water from the Elwha River for municipal and industrial uses from possible adverse impacts of dam removal."

This Project Planning Report develops mitigation measures for the City of Port Angeles and the Lower Elwha Klallam Tribe to protect against the possible adverse impacts of dam removal. The City of Port Angeles and the Lower Elwha Klallam Tribe currently supply water for the following uses:

- City of Port Angeles Residents (Municipal Water Supply)
- City of Port Angeles Industrial Consumers
- Washington Department of Fish and Wildlife (WDFW) Fish Rearing Channel
- Lower Elwha Klallam Tribe Fish Hatchery

This Project Planning Report does not address possible mitigation measures for other users of the Elwha River, including, but not limited to, the Dry Creek Water Association, the Elwha Place Homeowners Association, and various individual private wells located along the river. The mitigation measures for these water users are being developed directly between each of these users and the Olympic National Park. The Dry Creek Water Association has expressed interest in potentially being included in any mitigation measures proposed for the City of Port Angeles municipal supply. Inclusion of the Dry Creek Water Association into any of the mitigation measures developed in this report would not significantly affect the feasibility or cost due to the small amount of water required by this entity and its relative proximity to the City's water supply system.

The Washington State Department of Health (WDOH) requires that water purveyors shall submit Project Reports for written approval prior to installation of any new water system, water system extension, or major improvement. The requirements for Project Reports are defined in Washington Administrative Code (WAC) 246-290-110. The Project Report for the municipal mitigation measures will be submitted to WDOH as a separate report.

1.2 PROJECT DESCRIPTION AND HISTORY

In the early 1900's, two hydroelectric dams were constructed on the Elwha River located on the Olympic Peninsula in Washington State. Construction of the Elwha Dam began in 1910 and formed Lake Aldwell. Construction of Glines Canyon Dam began in 1925 and formed Lake Mills. Both dams were constructed without fish passage facilities and block the migration of the anadromous fish that historically used the river for spawning and rearing.

In 1968, the owner and operator of the dams, Crown Zellerbach Corporation, submitted an application to the Federal Energy Regulatory Commission (FERC) to license Elwha Dam and, in 1973, applied to re-license Glines Canyon Dam. FERC proceeded with licensing activities and processed the two licenses together. Licensing the two dams became controversial for a number of reasons including a challenge in the 1980's that FERC did not have jurisdiction to re-license Glines Canyon Dam because the dam is located within the Olympic National Park.

Subsequent to license applications, the assets of Crown Zellerbach Corporation were purchased by James River Corporation (currently Fort James Corporation). These assets included the Elwha and Glines Canyon Dams and a pulp and paper mill located in the City of Port Angeles. The mill was later sold to Daishowa America Co., Ltd. (Daishowa). Until February 2000, the Fort James Corporation owned the two dams and Daishowa operated the two dams and associated power plants. Daishowa received power from the dams for the operation of the mill.

In 1992, Congress passed Public Law 102-495, the Elwha River Ecosystems and Fisheries Restoration Act (Act). The Act directs the Secretary of the Interior to study ways to fully restore the Elwha River ecosystem and native anadromous fisheries, including purchase and removal of the dams. The Secretary's report, *The Elwha Report* (Interior, et al., 1994), determined that removing the dams was feasible and necessary to fully restore the fisheries and ecosystem. In February 2000, the Federal Government purchased the dams and related facilities from the Fort James Corporation. The Bureau of Reclamation with National Park Service oversight currently operates the dams. Operation will continue until decommissioning.

The dam removal approach and environmental impacts associated with dam removal alternatives were discussed in the *Elwha River Ecosystem Restoration Final Environmental Impact Statement* (EIS, Olympic National Park Service, November 1996). This report also proposed water quality mitigation measures for all users of the Elwha River, including:

- Construct a new Ranney collector on west side of the Elwha River to mitigate possible river migration away from the current Ranney collector and possible reduction in the City's municipal water supply.
- Provide a temporary cartridge filtration plant to treat the municipal potable water supply.
- Construct a new industrial infiltration gallery in place of the current industrial surface water diversion to provide water for the City's industrial customers.
- Construct a temporary industrial pretreatment facility to be a primary sedimentation basin to remove particulates from the water supplied to industrial customers.
- Temporarily relocate the chinook salmon production from the WDFW fish rearing channel during dam removal activities.

- Construct dikes to flood proof the area containing the current Ranney collector, the proposed package in-line filtration plant, and the industrial pretreatment facility.
- Upgrade the existing Tribal hatchery infiltration gallery, restore existing wells, construct new infiltration gallery, drill two new wells, and add aeration facilities.

The Bureau of Reclamation (BOR) prepared a report entitled *Water Quality Analysis and Mitigation Measures* (BOR, March 1997). This report examined the impact to water quality in the Elwha River from the preferred alternative for dam removal presented in the 1996 EIS. The *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1997) modeled sediment transport and predicted both short-term and long-term changes in water quality within the Elwha River as a result of dam removal. The report also examined alternative mitigation measures and developed a conceptual design and cost estimate for the water quality mitigation measures initially presented in the EIS.

Since the release of the *Water Quality Analysis and Mitigation Measures Report* in 1997, additional new information has become available that requires a re-evaluation of the proposed mitigation measures. Additional new information relevant to the project includes:

- The cartridge filter manufacturer proposed no longer makes the proposed equipment for drinking water treatment, and cartridge filtration is not approved by the Environmental Protection Agency (EPA) for the removal of cryptosporidium.
- One of the industrial water users, the Rayonier Pulp Mill (Rayonier), no longer exists.
- Additional potential sources of a temporary water supply to utilize during periods of extremely poor water quality have been identified and need to be evaluated.
- Additional potential sites for a treatment facility have been identified.
- The water from the City of Port Angeles' current Ranney collector has been classified as groundwater under the influence of surface water (GWI) and must now meet the treatment requirements of the Enhanced Surface Water Treatment Rule (SWTR) and Disinfection/Disinfectant By-Product Rule (D/DBP).
- The proposed improvements to the Tribe's infiltration galley will not protect the hatchery's water supply from potential river mitigation and corresponding reduction in yield.
- The National Marine Fisheries Service (NMFS) has listed the chinook salmon currently raised in the WDFW fish rearing facility as a threatened species and will not allow the rearing channel to be shut down as planned during the period of dam removal.
- The U.S. Fish and Wildlife Service has listed bull trout in the Elwha River as a threatened species.

Based on this new project information, a revised set of mitigation alternatives have been developed and evaluated as part of this Project Planning Report.

1.3 REPORT ORGANIZATION

The contents of the Elwha River Water Quality Mitigation Project Planning Report are divided into an executive summary, nine sections, and appendices. Each section describes components of the study and is briefly summarized in the following.

Executive Summary

The executive summary is a brief description of the study findings and is intended to provide a summary of the development and recommendations of the report.

Section 1 – Introduction

The introduction provides the purpose and scope of this study and gives the description and history of the project.

Section 2 – Elwha River

Information on the Elwha River is given in this section. The water rights associated with the key entities affected by the dam demolition are provided. The present and anticipated river water quality during and after the dam removal based on modeling by the Bureau of Reclamation are presented.

Section 3 – Applicable Drinking Water Treatment Regulations

The required drinking water treatment regulations are presented in this section and in the appendices. The municipal water treatment must be provided to treat water that will satisfy the present and be adaptable to the anticipated future drinking water regulations.

Section 4 – Existing Water Intake, Treatment and Distribution Facilities

The existing water supply and treatment facilities for the City of Port Angeles, industrial supply for Daishowa and the WDFW rearing channel, and Tribe hatchery are presented in this section.

Section 5 – Projected Water Quantities

To enable mitigation plans to be developed, the projected water use quantities are presented for each user including the City's municipal use, WDFW rearing channel, Daishowa Industries, and the Tribal hatchery. Both present and projected water usage is given.

Section 6 – Municipal Water System Alternatives

Water supply and treatment alternatives are developed in this section to provide drinking water to the City of Port Angeles water distribution system. Multiple treatment alternatives are presented with process descriptions, capital and operating costs, and advantages and disadvantages for each alternative. A recommended supply and treatment alternative is made.

Section 7 – Industrial and Fisheries Mitigation Measures

Methods and options for obtaining water and treating water for the industrial and fisheries users are presented in this section. The measures also consider providing an alternate source of water for the City municipal water supply.

Section 8 – Proposed Schedule for Implementation

A schedule for the planning, engineering, and construction of the water mitigation facilities is presented in this section.

Section 9 –Value Engineering

In accordance with Olympic National Park and Bureau of Reclamation guidelines value engineering was completed during the time that the study was being developed. Three meetings were conducted and proposals for investigation were prepared then further evaluated by the team preparing this report.

Appendices

The appendices are provided to supplement the findings of the sections and present more detailed information.

2.1 WATER RIGHTS

As outlined in the *Comprehensive Water System Plan* (CH2M Hill, 1995), the City of Port Angeles maintains water rights on the Elwha River and Morse Creek. The water rights on the Elwha River are summarized below.

The City obtained an appropriation of 150 cfs (97 mgd) from the Elwha River on August 12, 1927. The 1927 permit, Permit Number 1397, was for manufacturing purposes. The water right was verified with a certificate of water right on April 15, 1940. In 1974, through a change of use permit, 50 cfs of the 150 cfs was to be used for fish rearing by the WDFW.

The City's current water right includes 100 cfs (65 mgd) for manufacturing or industrial water supply and 50 cfs (32 mgd) for fish rearing.

In 1975, the City obtained Permit No. G2-21950 for the City's municipal supply totaling 50 cfs (32.4 mgd). The original permit requires that the water permit be certified by July 1, 2000, and an extension was granted to allow the permit to be valid through July 1, 2020. The annual appropriation for this water right is 20,600 acre-ft/yr (28.4 cfs or 18.4 mgd). According to the *Comprehensive Water System Plan* (CH2M Hill, 1995), the City's projected future water usage, maximum day demand (MDD) is 9.37 mgd in 1999, 9.63 mgd in 2003 and 10.23 mgd in 2012; therefore the City of Port Angeles has adequate permitting in-place for its municipal supply to meet future demands.

2.2 PRESENT WATER QUALITY

The Elwha River, its tributaries, Lake Mills, and Lake Aldwell are classified by the Washington Department of Ecology as Class AA waters, signifying "extraordinary" quality. Overall, the Elwha river has relatively low concentrations of dissolved and suspended sediment loads, nutrients, and organics. Suspended sediment concentrations and turbidity of the lower river are currently related to the reservoir trapping efficiency, flood flows, logging, agricultural practices, and bank erosion.

The water quality of the Elwha River is excellent with the concentrations of many chemicals below detection limits and all detections are less than the maximums allowed by the EPA and Washington State Drinking Water regulations. The quality of the Elwha River and the groundwater from the alluvium aquifer are very similar because of their hydraulic connection. The watershed feeding the Elwha River is under National Park status in the upper reaches. There are no landfills or industrial discharges into the river. Because of the small number of farm operations and limited development, non-point sources of pollution from agricultural and urban run-off have a very minor influence on the water quality.

The *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1997) reports water quality data from the Elwha River measured from a United States Geological Survey (USGS) gauging station at McDonald Bridge at river mile 8.6. The data reported was collected from 1974 to 1986. The concentration ranges of water quality criteria most likely to be impacted from dam removal are summarized in Table 2.1.

Table 2.1

ELWHA RIVER WATER QUALITY SUMMARY

Parameter and Unit	Concentration
Dissolved Oxygen	95-110 Percent Saturation
Total Suspended Solids (TSS)	1 to 1,500 mg/L 1.3 to 531 tons/day
Turbidity	1 to 800 NTU
Total Iron	0.02 – 2.3 mg/L
Copper	0.02 – 0.43 mg/L
Total Manganese	0.004 - 0.21 mg/L
Peak Temperature	66 ^{0}F
pH	6.7 – 10
Total Alkalinity	21 – 44 mg/L
Total Organic Carbon	0 – 10 mg/L

1 Source- Water Quality Analysis and Mitigation Measures Report. BOR, March 1997.

As stated earlier, the City of Port Angeles receives water for municipal supply from a Ranney collector at Elwha river mile 2.8. The Ranney collector draws water from the alluvium of the Elwha River at a depth of approximately 60 feet. The water in the alluvium is hydraulically connected to the surface water in the river. The alluvium does an excellent job of filtering the surface water and provides water of excellent quality. Chemical analysis of the water from the Ranney collector is presented in Appendix A. All chemical analyses indicated the chemical concentrations of water in the collector were below all state and federal primary and secondary drinking water standards. An analysis of copper taken from the distribution system in 2001 showed a concentration of 1.6 mg/L while the copper concentration at the source was between 0.02 and 0.43 mg/L (Appendix G1, Water Quality Analysis and Mitigation Measures, BOR, March 1997). The state standard for copper in the distribution system is 1.3 mg/L, suggesting a potential issue with corrosion within the Port Angeles water distribution system. The possible copper sources in the water distribution system are corrosion of brass and copper pipes and related fixtures.

Figure 2.1 shows the removal efficiency of the river alluvium by comparing turbidity in the surface water verses turbidity in the Ranney collector according to City operating records. The data shows an average turbidity removal efficiency of 94%.

In addition, water quality data collected at Daishowa and the former Rayonier mill are presented in Appendices B and C, respectively. The data collected by Daishowa shows turbidity data collected three times daily before and after treatment for the years 1983 through 1993, and data collected in 1999. Table 2.2 shows the monthly maximum and average turbidity readings recorded at Daishowa for the data reviewed.

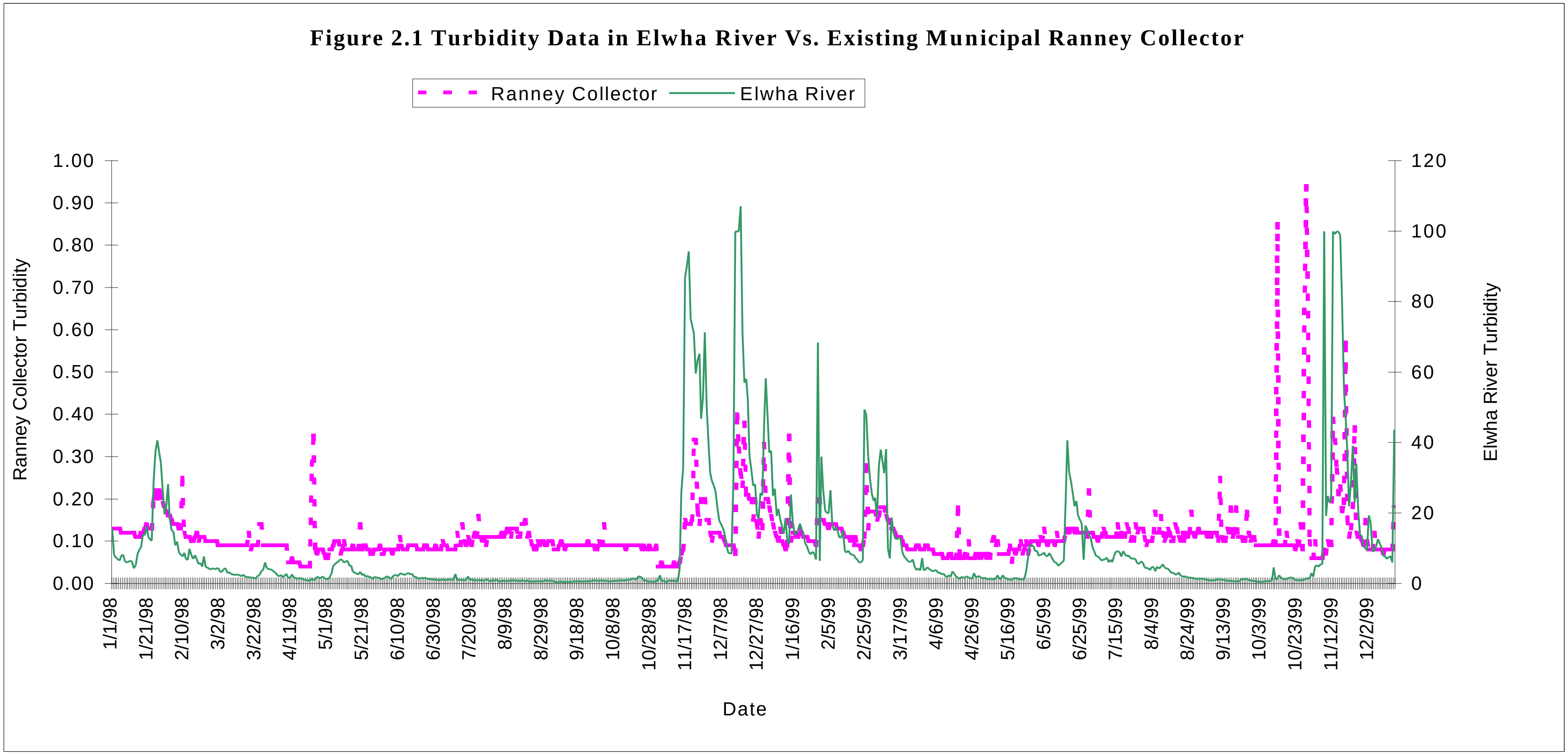

Figure 2.1 Turbidity Data in Elwha River Vs. Existing Municipal Ranney Collector
Ranney Collector
Elwha River
Ranney Collector Turbidity
Elwha River Turbidity
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
120
100
80
60
40
20
0
1/1/98
1/21/98
2/10/98
3/2/98
3/22/98
4/11/98
5/1/98
5/21/98
6/10/98
6/30/98
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12/7/98
12/27/98
1/16/99
2/5/99
2/25/99
3/17/99
4/6/99
4/26/99
5/16/99
6/5/99
6/25/99
7/15/99
8/4/99
8/24/99
9/13/99
10/3/99
10/23/99
11/12/99
12/2/99
Date

Table 2.2

MONTHLY MAXIMUM AND AVERAGE TURBIDITY READINGS AT DAISHOWA FOR YEAR 1983 – 1993 and 1999

Month	Maximum Turbidity Reading (NTU)	Average Turbidity Reading (NTU)
January	207	15
February	511	15
March	55	6
April	13	3
May	20	3
June	26	4
July	12	2
August	20	1
September	31	2
October	72	4
November	563	29
December	398	20

The data collected by Rayonier shows TSS concentrations measured daily from 1988 to 1991. Table 2.3 shows the monthly maximum and average turbidity readings recorded at Rayonier for the data reviewed.

Table 2.3

MONTHLY MAXIMUM AND AVERAGE TSS READINGS AT RAYONIER FROM 1988 - 1991

Month	Maximum TSS Reading (mg/L)	Average TSS Reading (mg/L)
January	41	9
February	14	2
March	15	3
April	12	3
May	12	2
June	13	3
July	41	6
August	45	6
September	55	5
October	16	4
November	846	46
December	159	34

2.3 ANTICIPATED WATER QUALITY DURING AND AFTER DAM REMOVAL

The removal of the Glines Canyon Dam and the Elwha Dam will allow sediment loads trapped behind the dams to move downstream to the ocean. In the short-term (a period of up to 5 years during and after removal of the dams), the water quality parameters that could be adversely affected include: suspended and dissolved solids, total organic carbon, turbidity, dissolved oxygen, iron, manganese, and temperature. Of these water quality parameters, the anticipated high concentrations of suspended solids present the greatest treatment challenge.

Suspended sediment concentrations will generally be much higher than currently experienced, and there will be intermittent periods of 1 to 3 days when large amounts of suspended sediment will be present in the Elwha River during dam demolition. High sediment loads may also occur after the dams are removed due to flood events until the sediments behind the dams are eroded away. The sediment transport model (BOR, October 1996) predict that concentrations of suspended sediments in the river could exceed 50,000 parts-per-million (5% solids) during individual high turbidity events.

Over the long-term, natural processes will be restored and water quality will return to natural conditions. The watershed in the reservoir area will have been returned to natural conditions, and the water quality associated with those natural conditions will be observed, whereas currently, the reservoirs behind the dam act as a buffering zone against substantial changes to water quality. Natural events such as landslides and forest fires may increase short term sediment loads in the river although they will have much less of an effect than the dam demolition.

A summary of current water quality conditions and anticipated water quality conditions within the Elwha River both in the short and long-term are presented in Table 2.4.

Table 2.4

ANTICIPATED IMPACTS TO WATER QUALITY

Water Quality Parameter (Units)	Current Conditions	Anticipated Short-Term Conditions	Anticipated Long-Term Conditions
Dissolved Oxygen (% saturation)	95-110	90-100	95-110
Total Suspended Solids (mg/L)	15[1]	15,000-51,000	69[1]
Turbidity (NTU)	1-800	2,000-25,000	1-1,000
Total Iron (μg/L)	20-2,300	30,000-50,000	10-5,000
Total Manganese (μg/L)	4-210	500-10,000	10-700
Peak Temperature (°F)	66	59-66	59-63
pH	6.7-10	5-9	6.5-8.5
Total Organic Carbon (mg/L)	0-10	100-1,000	10-200

Notes:

1. Represents average conditions.
2. All information from *Water Quality Analysis and Mitigation Measures Report*, Bureau of Reclamation, March 1997, Elwha Technical Series PN-95-8

3.1 PURPOSE AND INTRODUCTION

The purpose of this section is to discuss applicable drinking water treatment regulations, which must be met by the City of Port Angeles. The discussion includes regulations set by both WDOH and EPA and covers the following topics.

- The reclassification of the City's current source water from "groundwater" to "groundwater under the direct influence of surface water" (GWI).
- Drinking water treatment regulations for a groundwater source.
- Drinking water treatment regulations for a GWI source.

3.2 RECLASSIFICATION OF THE CITY'S MUNICIPAL SOURCE WATER

WDOH has informed the City of Port Angeles that their current source of water obtained through their Ranney collector has been designated as a GWI source in a letter dated April 25, 2000. The designation is based on testing results that suggest that the water from the City's Ranney collector is hydraulically connected to the surface water of the Elwha river. Furthermore, it is possible for pathogens to be transported from the river to the collection, and the City's source water may be at risk of cyst and virus contamination. The GWI classification is not attributed to the proposed dam removal activities.

A GWI determination means the source water is now subject to the requirements of the Surface Water Treatment Rule (SWTR) The SWTR requires public water systems using surface sources and GWI sources to provide treatment to achieve 3-log (99.9%) removal and/or inactivation of *Giardia lamblia* cysts and 4-log (99.99%) removal and/or removal of viruses. This can be accomplished by providing filtration, disinfection, and operation by qualified personnel.

Other options for avoiding the filtration requirement of the SWTR are the following:

- Develop an alternate WDOH-approved groundwater source.
- Re-construct the existing Ranney collector source, if possible, to eliminate surface water influence.
- Purchase water from a WDOH-approved public water system.
- Qualify for the WDOH criteria to remain unfiltered under WAC 246-290-690.
- Demonstrate the effectiveness of the existing Ranney collector as an alternative technology for filtration capable of removing cysts and viruses.
- Satisfy the WDOH criteria for unfiltered systems with a "limited alternative to filtration" under WAC 246-290-691;

The City is currently investigating all of their options as an independent and parallel process to this study. The City has entered into a Bilateral Compliance Agreement with WDOH that establishes a schedule for the City of Port Angeles to meet the requirements of the SWTR. According to the Bilateral Compliance Agreement, the City must provide a preliminary evaluation of their filtration alternatives by August 1, 2001, and provide a Project Report for the chosen alternative by July 1, 2002.

The municipal mitigation measures developed in this report will not only mitigate against the possible adverse affects of dam removal, but they also, by nature, meet the treatment requirements of the SWTR.

3.3 OVERVIEW OF REGULATORY AGENCIES AND REGULATIONS

Water quality from the City's Ranney collector is monitored and reported to both WDOH and EPA. The Safe Drinking Water Act (SDWA) was initially established in 1974 to set national enforceable standards. The SDWA allows the EPA to delegate primary enforcement authority to states, referred to as "primacy". As the EPA standards increase, the primacy states are required to adopt these new standards. The SDWA allows the EPA to set maximum contaminant levels (MCLs) for water quality parameters that have known negative health effects at varying levels. The SDWA also allows the EPA to set maximum contaminant level goals (MCLGs) which are non-enforceable health goals. The MCLG is the maximum level of contaminant at which no known or anticipated adverse health effects occur.

The SDWA was amended in 1986, and required EPA to set new regulations within 3 years, regulate 25 additional contaminants every 3 years, and developed a groundwater protection program. Regulation of groundwater sources replenished by surface water also began in the 1986 amendments, which required sources classified as Ground Water Under the Direct Influence (of surface water) or GWUDI (GWI) to meet the same regulation requirements as surface water sources. In 1996 additional amendments were added to the SDWA, that eliminated the requirement to regulate 25 additional contaminants every three years, allowed for enforcement flexibility, and provided the public with more information about their water quality.

National Primary Drinking Water Regulations (NPDWR) are standards that are applicable to public water systems and are enforceable whereas the National Secondary Drinking Water Regulations (NSDWR) are non-enforceable limits to contaminants that cause either cosmetic effects or aesthetic effects. Contaminants included in the NPDWR are inorganic chemicals, organic chemicals, radionuclides, and microorganisms. The EPA is currently in the process of revising regulations for microbials, disinfectants, disinfection byproducts (DBPs), radon, radionuclides, groundwater, and arsenic.

See Appendix D for the current National Primary Drinking Water Regulations.

3.4 DRINKING WATER REGULATIONS FOR GROUNDWATER AND SURFACE WATER SOURCES

Prior to April, 2000, the source water for Port Angeles was classified as a groundwater source. Under the classification as a groundwater source, the following drinking water regulations were applicable:

- Phase I (8 VOC Standards)
- Phase II/IIB and Phase V (IOC/SOC Standards)
- Total Coliform Rule
- Total Trihalomethane Standard
- Lead and Copper Rule

- Interim and Proposed Radionuclides
- Disinfectants/Disinfection By-Products Rule (D/DBPR) Stage I
- Disinfectants/Disinfection By-Products Rule (D/DBPR) Stage II

The above regulations are also applicable to the source water under the recent classification as a GWI.

Since the source of supply has been classified as GWI, additional regulations that now apply include the following:

- Surface Water Treatment Rule (SWTR)
- Information Collection Rule (ICR)
- Interim Enhanced Surface Water Treatment Rule (IESWTR)

3.4.1 Phase I (8 VOC Standards)

This Rule effective in 1989 required monitoring for eight volatile organic compounds (VOCs). These eight VOCs include the following:

- 1,3-Dichlorobenzene
- Vinyl chloride
- p-dichlorobenzene
- 1,2-Dichloroethane
- 1,1,1-Trichloroethane
- Tetrachloromethane
- Trichloroethylene (TCE)
- Benzene

This rule is applicable to both surface and groundwater, but groundwater sampling was initially only required once annually, whereas the surface water monitoring requirements are more stringent. If there are no detects for three years then sampling may be reduced to once every 36 months for groundwater. Systems that have surface water as the source of supply, initially need to sample for 8 consecutive quarters. If there is no VOC detection then monitoring may be reduced to once annually. VOCs are to be monitored at a representative entry point of each well for groundwater. The sampling location for surface water is a site in the distribution system representative of each source or an entry point to the distribution system after treatment.

3.4.2 Phase II/IIB and Phase V

Phase II of the SDWA was effective January 1991 and included the additional monitoring of 9 inorganic chemicals (IOCs), 10 VOCs, and 15 synthetic organic chemicals (SOCs) and 4 other organic chemicals, for a total of 38 contaminants that were either updated or new limits were created.

In July 1992, final National Primary Drinking Water Regulations (known as the Phase V Rule), identified 18 additional SOCs and five additional IOCs. IOCs include Cyanide, Antimony, Beryllium, Nickel, and Thallium. Monitoring frequency for ground water and surface water is equal for both the VOCs and the SOCs but stricter monitoring is required for IOCs for surface water.

Phase II/IIB	8 IOCs	(excluding asbestos, nitrate, and nitrite) • Initial- sw: annual; gw:tri-annual • Repeat- Same as initial, unless results are ≥ MCL, system goes on quarterly monitoring until R&C below MCL
	10 VOCs	• Initial- sw & gw: 4 consecutive quarterly samples • Repeat- sw&gw: quarterly if detects; annual if no detects & gw goes to triennial if 3 consecutive years without detects
	15 SOCs	• Initial-sw & gw: 4 consecutive quarterly samples • Repeat: sw & gw: if detects, monitor quarterly Sw & gw: if no detects for a system serving > 3,300, monitor 2x/3 years
	IOCs	Asbestos- Initial- 1 sample Repeat- if initial is = MCL, monitor during 1st 3 years of next 9-year cycle > MCL, monitor, quarterly until R&C below MCL Nitrate (no waivers allowed) CWSs/NTNCWSs: sw-quarterly; gw-annually Reduce from quarterly to annually if 4 consecutive quarters < ½ MCL Increase from annually to quarterly if ≥ ½ MCL TNCWSs: annual monitoring
Phase V	5 IOCs	Monitoring frequency is the same as for Phase II/IIB (excluding asbestos, nitrate, or nitrate)
	3 VOCs	Monitoring frequency is the same as for Phase II/IIB (excluding asbestos, nitrate, or nitrate)
	15 SOCs	Monitoring frequency is the same as for Phase II/IIB (excluding asbestos, nitrate, or nitrate)

sw- surface water source; gw- groundwater source; CWS- community water system; NTNCWS- non transient non community water system; TNCWS- transient non community water system; R&C- reliably and consistently below the MCL for sw=2 consecutive quarters below the MCL; for gw- 4 consecutive quarters below the MCL.

3.4.3 Total Coliform Rule (TCR)

The Total Coliform Rule (TCR), established in June 1989, sets the MCL for total coliforms based upon the classification of the source water. Although this rule was applicable when the City's water source was not considered GWI, new regulatory procedures are required since the classification as a GWI. The EPA set the following criteria:

- For systems that collect less than 40 samples per month, then no more than 1 sample may be total coliform positive.

A city the size of Port Angeles must sample 20 times per month according to EPA regulations. If the system violates these criteria then the public must be notified per EPA regulatory language. If the system has one positive sample then repeat samples must be taken within 24 hours of the initial positive sample.

3.4.4 Total Trihalomethane (TTHM) Rule

In November 1979, EPA set an interim MCL for total trihalomethanes (TTHM) of 0.1 mg/l as an annual average. The value is the sum of the measured concentrations of chloroform, bromodichloromethane, dibromochlorormethane and bromoform. This only applies to surface water or GWI sources for systems serving 10,000 people or greater, such as Port Angeles, that add chlorine to the potable water during any part of the treatment process.

3.4.5 Lead and Copper Rule

The Lead and Copper Rule was initially published in June of 1991 and updated in January 2000. The rule applies to any public water system. This rule primarily applies to levels of lead and copper in tap water as a result of the degradation and corrosion of lead and copper piping. The regulations include requirements for corrosion treatment, source water treatment, lead service line replacement, and public education. The lead action level is exceeded if the concentration of lead is greater than 0.015 mg/l (90^{th} percentile) in more than 10 percent of tap water samples. The copper action level is exceeded if the concentration of copper is in more than 10 percent of tap water samples are greater than 1.3 mg/l (90^{th} percentile). If any system exceeds the lead action level after implementation of corrosion control and source water treatment then lead service lines must be replaced.

The minor revisions of January 2000 included the optimal corrosion control, lead service line replacement requirements, public education requirements, monitoring requirements, analytical methods, reporting and record keeping requirements, and special primacy considerations. These minor revisions allowed for streamlining the requirements, promoting consistent national implementation, and in many cases, reducing the capital cost burden on water system operations.

3.4.6 Interim and Proposed Radionuclides

In 1976 the National Interim Primary Drinking Water Regulations were promulgated for radium-226 and –228, gross alpha particle radioactivity, and beta particle and photon radioactivity. The final Radionuclides Rule will become effective on December 8, 2003, three years after the publication date. This rule requires all community water systems which has at least 15 service connections or 25 residents to meet final MCLs and meet the requirements for monitoring and reporting of the following radionuclides:

- Combined radium-226/228
- Gross alpha
- Beta particle
- Photon radioactivity
- Uranium

The radionuclides rule does not include radon, this will have its own regulation which has not yet been finalized.

The final rule will require monitoring to be conducted at each entry point into the distribution system under a schedule in compliance with regulations. WDOH can determine the initial compliance by either using four quarterly samples or using appropriate previously collected data. This rules ensures that the water system meets the MCL standards for the entire system, the previous rule did not require monitoring at each entry point, only at a representative point in the system. This rule also promulgates a new standard for uranium.

The following table summarizes the 1976 rule and the 2000 rule:

Provision	1976 rule (current rule)	2000 Final Rule
MCLG for all radionuclides	No MCLG	MCLG of zero
Radium MCL	Combined Ra-226 +Ra-228 of 5 pCi/L	Maintain current MCL based on newly estimated risk level associated with 1991 proposed MCL of 20 pCi/L
Gross alpha MCL	15 pCi/L excluding U and Rn but including Ra-226	Maintain current MCL based on newly estimated risk level associated with 1991 proposed MCL of 15 pCi/L excluding Ra-226, radon, and uranium
Uranium MCL	Not Regulated	30 µ/L
Lead-210	Not Regulated	No changes
Ra-224	Part of gross alpha, but sample holing time too long to capture Ra-224	No changes, will collect national occurrence information; further action may be required at a later date.
Radium monitoring	Ra-226 linked to Ra-228; measure Ra-228 if Ra-226> 3 pCi/L and sum.	Measure Ra-226 and Ra-228 separately
Monitoring baseline	4 quarterly measurements. Monitoring reduction based on results: >50% of MCL required 4 samples every 4 yrs; <50% of MCL required 1 sample every 4 yrs.	Four initial consecutive quarterly samples in first cycle. If initial average level >50% of MCL: 1 sample every 3 years; <50% of MCL: 1 sample every 6 years; Non-detect: 1 sample every 9 years. States have discretion in data grandfathering for establishing initial monitoring baseline.

Provision	1976 rule (current rule)	2000 Final Rule
Beta particles and photon emitters monitoring	Surface water systems >100,000 population Screen at 50 pCi/L; vulnerable systems screen at 15 pCi/L	Community water systems determined to be vulnerable by the State screen at 50 pCi/L
Gross alpha monitoring	Analyze up to one year later	Six month holding time for gross alpha samples; annual compositing of samples allowed.

pCi/L = Picocurie per liter

3.4.7 Disinfectants/Disinfection By-Product Rule (D/DBPR)

3.4.7.1 Stage 1 D/DBP Rule

The purpose of the D/DBP Rule is to limit the levels of both disinfectants and disinfection by-products (DBPs) in drinking water. Disinfectants are effective in controlling microorganisms but they also react with organic matter to form disinfection by-products that are regulated carcinogens. The D/DBP Rule sets levels in order to reduce DBP formation.

The following tables from the EPA Guidance Manual Alternative Disinfectants and Oxidants, April 1999, are the regulations relating to the DBP/D Rule. Included are the MCLGs, MCLs, MRDLGs, and MRLDs:

Table 3.1

PRIMARY DRINKING WATER REGULATIONS RELATED TO MICROBIOLOGICAL CONTAMINANTS

Compound	MCLG (mg/L)	MCLG (mg/L)	Potential Health Effects	Sources of Drinking Water Contamination
Giardia lamblia	Zero	TT[1]	Gastroenteric disease	Human and animal fecal waste
Legionella	Zero	TT	Legionnaire’s disease	Common bacteria in natural waters; can proliferate in water heating systems
Heterotrophic Plate Count	N/A	TT	Indicates water quality, effectiveness of treatment	
Total Coliform	Zero	<5.0%[2]	Indicates potential presence of gastroenteric pathogens	Human and animal fecal waste
Turbidity	N/A	TT	Indicates potential water treatment failure and pathogens in drinking water	Particles from storm runoff, discharges into source water and erosion
Viruses	Zero	TT	Gastroenteric disease	Human and animal fecal waste

Source: AWWA Internet, 1997

1. TT = Treatment technique requirement in lieu of MCL as established in 40 CFR §141.70
2. No more than 5.0 percent positive if >40 samples/month. No more than 1 positive if <40 samples/month [40 CFR §141.63(a)].

Table 3.2

PRIMARY DRINKING WATER REGULATIONS RELATED TO DISINFECTION BYPRODUCTS

Compound	MCLG (mg/L)	MCLG (mg/L)	Potential Health Effects	Sources of Drinking Water Contamination
Bromate	Zero[3]	0.010[4]	Cancer	Ozonation byproduct
Bromodichloromethane	Zero[3]	See TTHMs	Cancer, liver, kidney, and reproductive effects	Drinking water ozonation, chloramination, and chlorination byproduct
Bromoform	Zero[3]	See TTHMs	Cancer, nervous system, liver and kidney effects	Drinking water ozonation, chloramination and chlorination byproduct
Chlorite	0.8[3]	1.0[4]	Hemolytic anemia	Chlorine dioxide disinfection byproduct
Chloroform	Zero[3]	See TTHMs	Cancer, liver, kidney, reproductive effects	Drinking water chlorination and chloramination byproduct
Dibromochloromethane	0.06[3]	See TTHMs	Nervous system, liver, kidney, reproductive effects	Drinking water chlorination and chloramination byproduct
Dichloroacetic Acid	Zero[3]	See HAA5	Cancer and other effects	Drinking water chlorination and chloramination byproduct
Haloacetic Acids[1] (HAA5)	NA	0.060[4]	Cancer and other effects	Drinking water chlorination and chloramination byproduct
Trichloroacetic Acid	0.3[3]	See HAA5	Possibly cancer and reproductive effects	Drinking water chlorination and chloramination byproduct
Total Trihalomethanes (TTHMs)	N/A	0.08[4]	Cancer and other effects	Drinking water chlorination and chloramination byproduct

Source: 63 FR 69390 (12/16/98)

[1] HAA5 is the sum of the concentrations of mono-, di-, and trichloroacetic acids and mon-and dibromoacetic acids.

[2] Total Trihalomethanes are the sum of the concentrations of bromodichloromethane, dibromochloromethane, bromoform, and chloroform.

[3] Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.53.

[4] Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.64.

Table 3.3

PRIMARY DRINKING WATER REGULATIONS RELATED TO RESIDUAL DISINFECTANTS

Disinfectant	MRDLG[3] (mg/L)	MRDL[4] (mg/L)
Chlorine[1]	4(as Cl_2)	4.0 (as Cl_2)
Chloramine[2]	4(as Cl_2)	4.0 (as Cl_2)
Chlorine Dioxide	0.8 (as CIO_2)	0.8 (as ClO_2)

[1]Measured as free chlorine

[2]Measured as total chlorine

[3]Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.54.

[4]Finalized on December 16, 1998 (63 FR 69390) as established in 40 CFR §141.65.

The Stage 1 Disinfectants and Disinfection Byproducts Rule (Stage 1 D/DBPR) applies to all community and non-transient non-community water systems that treat their water with a chemical disinfectant for either primary or residual treatment. All systems that use surface water or a source classified as GWI and serve 10,000 or more people must comply with Stage 1 D/DBPR by December 2001. Stage 1 D/DBPR establishes limits of exposure to three disinfectants and many disinfection by-products, such as the following:

- Lowered the existing MCL for Total Trihalomethanes (TTHMs) from 0.10 mg/L to 0.080 mg/L.
- Extended the MCL for TTHMs to all size systems.
- Requires enhanced coagulation or enhanced precipitative softening for certain systems.
- Established maximum contaminant disinfectant levels (MRDLs) and maximum contaminant disinfectant level goals (MRDLGs) for chlorine, chloramine, and chlorine dioxide.
- Established MCLs for haloacetic acid (five) (HAA5), bromate, and chlorite, see Table 3.4.
- Established MCLGs for eight disinfection byproducts.

Table 3.4

D/DBP COMPLIANCE CRITERIA

	MRDLG (mg/L)	MRDL (mg/l)	Compliance Based On
Disinfectant Residual			
Chlorine	4 (as Cl2)	4 (as Cl2)	Annual Average
Chloramine	4 (as Cl2)	4 (as Cl2)	Annual Average
Chlorine Dioxide	0.8 (as ClO2)	0.8 (as ClO2)	Daily Samples
Disinfection Byproducts			
Total Trihalomethanes (TTHM)1	N/A	0.080	Annual Average
Chloroform	0		
Bromodichloromethane	0		
Dibromochlorormethane	0.06		
Bromoform	0		
Haloacetic acids (five) (HAA5)2	N/A	0.060	Annual Average
Dichloracetic acid	0		
Trichloroacetic acid	0.3		
Chlorite	0.8	1.0	Monthly Average
Bromate	0	0.010	Annual Average

N/A- Not applicable because there are individual MCLGs for TTHMs or HAAs

Total trihalomethanes is the sum of the concentrations of chloroform, bromodichloromethane, dibromochloromethane, and bromoform.

Haloacetic acids (five) is the sum of the concentrations of mono-, di-, and trichloroacetic acids and mono- and dibromoacetic acids.

Systems using conventional filtration treatment must operate with enhanced coagulation or enhanced softening to optimize the removal of total organic carbon (TOC) to prevent the formation of DBPs. Table 3.5 gives the required removal based on TOC and alkalinity.

Table 3.5

D/DBP ENHANCED COAGULATION REQUIREMENTS

Source Water TOC (mg/l)	Required TOC Removal, percent		
	Source Water Alkalinity (mg/L as CaCO3)		
	0-60	>60-120	>120
>2.0-4.0	35	25	15
>4.0-8.0	45	35	25
>8.0	50	40	30

1. Systems meeting at least one of the alternative compliance criteria in the rule are not required to meet the removals in this table.
2. Systems practicing softening must meet the TOC removal requirements in the right column.

3.4.7.2 Stage II D/DBP Rule

The second phase, consisting of the Stage 2 DBPR and the Long-Term 2 (LT2) ESWTR, will be promulgated in the year 2002 and will revisit the regulations for the formation of DBPs for all systems and the inactivation and removal of pathogens for surface water systems, respectively. The intent of Stage II is to provide protection of public health from DBPs.

- Stage II D/DBPR (May 2005) – Lowers the MCL for TTHM levels to 0.08 mg/L and HAA5 levels to 0.06 mg/L
- LT2 treatment technique requirements to mitigate Cryptosporidium risks
- System compliance schedules
- Monitoring of microbial and disinfection byproducts that are site specific and based on risk.
- Compliance monitoring
- Long-term assessment of microbials

EPA anticipates the Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) and the Stage 2 Disinfectants and Disinfection Byproducts Rule will be finalized in mid 2001 and promulgated in 2002. The intent of LT2ESWTR is to protect the public health against microbial pathogens.

3.5 DRINKING WATER REGULATIONS FOR SURFACE WATER SOURCES

With the source water for Port Angeles now considered a GWI source, the City will be required to meet the same regulations that pertain to direct surface water sources. The following regulations will apply to the City's water system:

3.5.1 The Surface Water Treatment Rule (SWTR)

Under this Rule, which was established in June 1989, the EPA set MCLGs of zero for Giardia Lamblia, viruses, and Legionella. The SWTR includes treatment techniques for both filtered and unfiltered systems for the protection against pathogenic organisms. The following items are all components of the SWTR.

- SWTR protects against Giardia, viruses, and Legionella, as well as many other pathogenic organisms
- Requirements for maintenance of disinfectant residual in distribution systems
- Removal and/or inactivation of 3 log (99.9%) for Giardia and 4 log (99.99%) for viruses
- Combined filter effluent turbidity performance standard of 5 NTU as a maximum and 0.5 NTU at the 95^{th} percentile monthly.
- Watershed protection.

3.5.2 The Interim Enhanced Surface Water Treatment Rule

The Interim Enhanced Surface Water Treatment Rule (IESWTR) applies to all public water systems that use surface water, or ground water under the direct influence of surface water and that serve at least 10,000 people. The IESWTR updated the SWTR to strengthen microbial protection, including provisions to address Cryptosporidium, and to address other waterborne pathogens. In addition, water systems must continue to meet existing requirements for Giardia Lamblia and viruses. The IESWTR rule includes:

- Maximum contaminant level goal (MCLG) of zero for Cryptosporidium
- 2-log removal Cryptosporidium requirements for systems that filter
- Strengthened combined filter effluent turbidity performance standards (0.3 NTU in 95% of samples)
- Individual filter turbidity monitoring provisions
- Disinfection profiling and benchmarking provisions

3.5.3 Information Collection Rule (ICR)

The purpose of the ICR is to collect treatment information from treatment facilities in order to aid the EPA in updating the SWTR in regards to microbial treatment practices and help in evaluating the need for future regulation for disinfectants and disinfection by-products. This will allow for further evaluation of disinfectant by-products and disease causing microorganisms at a national level. An EPA committee concluded that more information was needed regarding microbials, disinfectants, and disinfectant by-products in order to assess the extent and severity of risk to make public health decisions. This rule also provides information to help determine how to cost effectively implement regulations. Additionally, the data could also be used to determine national costs for various treatment options.

4.1 MUNICIPAL FACILITIES

4.1.1 Existing Water Intake and Treatment

The City's municipal water supply is derived from the induced groundwater flow below the channel of the Elwha River. The City collects water for its municipal use through a Ranney collector located near river mile 2.8 on the east bank of the Elwha River. The Ranney collector was constructed in 1977 and consists of a 13-foot diameter concrete caisson extending to a depth of 62 feet below ground surface. From the bottom of the caisson, seven 10.75-inch outside diameter, horizontal screen laterals extend outward providing a total screen length of 528 feet. Pumping facilities in the Ranney collector consist of two 600-horsepower pumps rated at 3,700 gpm each at 530 feet total dynamic head. The water from the Ranney collector is disinfected by gaseous chlorine that is fed from 1-ton cylinders to the water before release to the City of Port Angeles' municipal water distribution system.

4.1.2 Distribution System

Water from the City's Ranney collector is pumped directly into the medium-level service zone. The high level service zone is fed by water pumped from the medium-level service zone. The low-level service zone is fed by gravity primarily from the medium zone, but the eastern part of the system can also be fed directly from the high zone during an emergency.

The City has five storage reservoirs located within its distribution system. The total storage capacity of the reservoirs is 18 million gallons. Three of the reservoirs are covered and two are not. Each of the uncovered reservoirs and one of the covered reservoirs have re-chlorination stations at the reservoir outlet. Re-chlorination uses sodium hypochlorite. A schematic of the City's current water distribution and wastewater systems is included as Figure 4.1. A more detailed description of the City's distribution facilities may be found in the *Comprehensive Water System Plan* (CH2M Hill, 1995).

In 1974, before construction of the Ranney collector, a thorough hydrogeological investigation of the existing collector site was completed. From the results of test pumping, the yield of one Ranney collector was estimated to be 14.4 million-gallons-per-day (mgd) or 22.3 cubic feet per second (cfs) under average conditions of river stage and water temperature, decreasing to 11.7 mgd (18.1 cfs) under minimum flow conditions. The results of the 1977 initial performance test of the completed Ranney collector calculated an average yield of 15.7 mgd (24.3 cfs) and a minimum yield of 12.8 mgd (19.8 cfs) with a pumping level elevation of 20 feet (Ranney, 1977).

In 1985, the City observed a decline in collector capacity. The water level would decline with both pumps running until the low-level alarm was reached and one pump would automatically shut-off. Ranney reviewed the City data and concluded that the capacity in 1985 had declined to about 11.0 mgd (17 cfs) compared to the initial capacity of 15.7 mgd (24.3 cfs) (Ranney, 1985). The decline appeared to be due to migration of the river channel away from the collector and not due to clogging of the laterals. Ranney suggested that the City take measures to divert more river water into the side channel near the collector to increase recharge to the aquifer in that area. Although there was no evidence of clogging of the horizontal screen laterals, Ranney also suggested that a pumping test be performed to further evaluate possible clogging of the laterals.

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FIGURE PROVIDED COURTESY OF THE CITY OF PORT ANGELES

URS

PORT ANGELES
WATER/WASTWATER
UTILITIES

FIGURE 4.1

In April 1994, the City authorized Ranney to conduct a controlled pumping test to determine the cause of continued capacity decline. An 8-inch diameter observation well was constructed adjacent to the collector and was used to measure the water level in the aquifer during pumping of the collector. The test measured an operating efficiency of the Ranney collector of 94.9-96.0 percent, which indicated that the screen laterals were not clogged. The test further computed the effective distance to the line of infiltration and found that a significant change in the recharge conditions had occurred since the initial testing in 1977 (Ranney, 1994). The report to the City attributed the changed condition to the loss of river flow in the side channel near the collector and the increased distance to the main channel of the river. The 1994 yield of the collector was calculated at 10.7 mgd (16.6 cfs).

In April, 2000, the source water for the Ranney collector was classified as GWI by WDOH. The re-classification of the City's municipal water source as GWI means that the source is now subject to the requirements of the SWTR and will require further treatment and monitoring than what has been previously required. The GWI classification of the City's water supply is not a result of the proposed ecosystem restoration plan, and the City would be responsible for the additional treatment requirements even if the dams were to remain in place. A description of the applicable drinking water treatment regulations and treated water quality criteria are presented in Section 3.

4.2 INDUSTRIAL FACILITIES

The City has a water right to divert 96.9 mgd (150 cfs) of river flow for industrial use and owns an industrial water delivery system. Specifically, the City has a water right for 32.5 mgd (50 cfs) for the WDFW fish rearing channel and 64.6 mgd (100 cfs) for industrial use. Currently, this system supplies untreated surface water by gravity to the Daishowa pulp and paper mill and the WDFW fish rearing channel. A rockfill diversion dam across the river at river mile 3.48 directs surface water flow to a diversion tunnel. At the tunnel outlet, the flow is divided to supply to the WDFW fish rearing channel and to the industrial diversion channel. The diversion channel leads to a weir and screen house that controls flow into the industrial pipeline. The surface water diversion tunnel and industrial diversion channel can handle a maximum capacity of approximately 194 mgd (300 cfs).

Daishowa currently has a contract with the City for 20 mgd (31 cfs). Excess flow is returned to the river through a by-pass channel to the river. The transmission pipeline was originally constructed as a combination of concrete and wood stave pipe ranging in diameter from 72 to 57 inches with a 48-inch pipe extending through the City. Most of the wood stave pipe has been replaced with concrete pipe.

According to Daishowa staff, the industrial pipeline had a design capacity of approximately 65 mgd (100 cfs). A flow survey of the industrial pipeline, conducted by Rayonier in 1957 shows that the max daily flow into the pipeline was 58 mgd (90 cfs) with a peak instantaneous flow of 64 mgd (99 cfs). In 1999, a natural landslide took out a section of the pipeline just downstream of the fish screen house. Approximately 60-feet of 72-inch diameter pipe was replaced with 57-inch diameter pipe. This restriction in the pipeline is located approximately 100 yards downstream of the fish screen house. The current capacity of the industrial pipeline with this new restricted section is unknown.

Daishowa treats the water at the plant site primarily to reduce sediment and turbidity prior to use. Daishowa uses a conventional treatment process consisting of chemical addition, flocculation, sedimentation, and filtration to treat the water. The water treatment plant was constructed in 1929. Treatment residuals from the coagulation and sedimentation process are currently conveyed to the strait shoreline adjacent to the plant and discharged as allowed by Daishowa's National Pollution Discharge Elimination System (NPDES) permit.

Review of Daishowa water treatment plant data from December 1998 to November 1999 indicated that untreated water turbidity to the plant ranged from less than 1 NTU to 140 NTU. The filtered water turbidity from the plant ranged from 0.11 NTU to 8.66 NTU. Both untreated and treated waster data for the Daishowa water treatment plant are listed in Appendix B. Daishowa's treatment plant does not treat water to potable water quality standards as plant. Staff have indicated that water turbidity not greater than 2 NTU is acceptable for mill operations.

4.3 FISHERIES

4.3.1 Washington Department of Fish and Wildlife Fish Rearing Channel

The rearing channel was constructed in 1975 pursuant to an agreement between the State of Washington Department of Fisheries and the Crown Zellerbach Corporation. The facility is located on the east bank of the river near the industrial diversion channel and existing Ranney collector at river mile 2.8. The facility consists of a fish rearing trapezoidal channel with a total top width of about 80 feet, bottom width of 20 feet, depth of 6 feet, and length of 1,400 feet. The total volume is 415,500 cubic feet. An adult holding pond is located at the downstream end of the rearing channel. The facility also includes support buildings and living quarters.

Water for the WDFW fish rearing channel is collected through the industrial diversion and split to the channel upstream of the industrial diversion channel. Although the WDFW contract with the City of Port Angeles is for 50 cfs, the maximum flow required for the fish rearing program anticipated during the dam removal and delta sediment erosion period is approximately 14.2 mgd (22 cfs). Outflow from the facility returns to the river by an open channel and is used to attract returning anadromous fish augmented by overflow from the industrial diversion channel. Water to the facility is metered using a sharp crested weir. The projected water quantities required by the WDFW fish rearing channel are discussed in the next section.

The WDFW facility currently utilizes four groundwater wells to provide cooler, pathogen-free water for the rearing channel and adult holding pond. One well was drilled in 1992 and the other three were drilled in 1995. Three of the wells supply the upper end of the rearing channel, and one well supplies the adult holding pond. The total rated output for the wells is approximately 3.6 mgd (5.6 cfs), but WDFW staff report considerably less output. The wells are currently operated with 5-10 pounds per square inch (psi) back pressure providing a total output of approximately 1.7 mgd (2.7 cfs).

4.3.2 Lower Elwha Klallam Tribe Fish Hatchery

The Tribe operates a fish hatchery on Tribal land near the mouth of the Elwha River. Current maximum water demand for the hatchery is about 7.4 mgd (11.5 cfs). Expansion plans as part of the Elwha River Ecosystem Restoration Project call for increasing the hatchery water use to approximately 18.6 mgd (28.8 cfs). During the dam removal and delta sediment erosion period the maximum water use by the hatchery is projected to be 12.4 mgd (19.2 cfs) (*Water Quality and Quantity Parameters, Lower Elwha Klallam Tribal Fish Hatchery, MWH, October 2001).*

An infiltration gallery, located about ½-mile upstream from the hatchery was upgraded in 1989 and previously had delivered approximately 4.5 to 5.1 mgd (7 to 8 cfs) of untreated river water to the hatchery by gravity flow. The infiltration gallery consists of approximately 600 linear feet of 18-inch diameter, perforated, polyvinyl chloride pipe buried about 10 feet in the riverbed.

In 1997, a natural log jam located upstream of the Tribal infiltration gallery was washed down river. As a result, the quantity of water flowing to the channel where the Tribal infiltration gallery is located has substantially decreased along with the available yield from the existing infiltration gallery. The infiltration gallery yield has decreased approximately 90% since the release of the log jam. Based on conversations with Tribal hatchery staff, the current yield from the infiltration gallery can be as low as 0.43 mgd (0.67 cfs).

River water is mixed with approximately 2.9 mgd (4.5 cfs) of groundwater supplied by two wells. Aquifer testing indicates that a steady-state yield of 5.2 mgd (8 cfs) from a well field in this area is possible without the risk of salt water intrusion (Dames and Moore, 1989). This yield has not been verified based on any new or future wells that are present in the area since the 1989 study.

In 1998, a study was conducted to evaluate the performance of the Tribal infiltration gallery (Gathard Engineering and Consulting, 1998). The study determined that there was good correlation between the turbidity in the river and the turbidity of water collected through the infiltration gallery. The study indicated that approximately 85-92% of the river water turbidity was removed through the infiltration gallery. The study also attempted to correlate the total suspended solids (TSS) concentration in the river to the TSS concentration collected through the infiltration gallery. Although no direct correlation was established, it was concluded from the data that up to 90% of the river water TSS may be removed through the infiltration gallery. The turbidity data for the river water collected from September 27, 1997 to March 06, 1998 ranged between 5 NTU and 120 NTU, which are converted to be 2 mg/L and 215 mg/L TSS by using the mathematical relationship (TSS measured in PPM = 0.0002 x NTU^3 – 0.018 x NTU^2 + 1.096 x NTU – 2.8415). In the same way, the turbidity data from water passing through the infiltration gallery were between 3 NTU and 34 NTU. This correlates to TSS values between 0.3 mg/L and 21 mg/L.

5.1 GENERAL

In order to properly size any water intake, treatment, or supply alternatives, the average daily demand and peak daily demand for all users must be identified. The following descriptions of current water demand and projected future demand are based on the *Comprehensive Water System Plan (CH2M Hill, 1995)*, the *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1997), and discussions with the City of Port Angeles, Daishowa, the WDFW rearing channel, and the Lower Elwha Klallam Tribe hatchery. Current and future water use for each of the affected entities is described in the following sections. A water demand summary is presented at the end of this section.

5.2 WATER SYSTEM DEMANDS

5.2.1 Port Angeles Municipal Demand

The City currently holds a groundwater right permit for 32.3 mgd (50 cfs) for municipal purposes. The current average daily demand for potable water is approximately 3.4 mgd (5.3 cfs) and the maximum daily demand is approximately 10.0 mgd (15.4 cfs). The yield of the current Ranney collector in 1994 was estimated at approximately 10.7 mgd (16.6 cfs).

The size of the municipal water treatment facility is based on the twenty-year projected maximum daily demand provided by the City. The twenty-year peak daily demand was determined from the *Comprehensive Water System Plan* (CH2M Hill, 1995). In the plan and the further confirmation with the City in 2000, the twenty-year peak day water demand projection was extrapolated from the numbers presented with the assumption that there have been no changes in factors effecting growth and demand. The twenty-year projected peak daily demand is 10.6 mgd (16.4 cfs). The present day yield of the current Ranney collector of 10.7 mgd (16.6 cfs) is sufficient to meet the twenty-year projected peak daily demand. Peak hour demands are met by the City's distribution system storage according to the Comprehensive Water Plan (CH2M Hill, 1995) as long as there is a daily maximum day water production of 10.6 mgd.

5.2.2 Industrial Demand

The City currently has a water right to divert 64.5 mgd (100 cfs) of river flow for industrial use. The industrial water right is used to supply water to Daishowa. According to Daishowa staff, the mill currently has an average daily demand of approximately 9 mgd (13.9 cfs) and peak daily demand of 14.0 mgd (21.7 cfs). Daishowa has a contract with the City to provide up to 20 mgd (31 cfs).

A large portion of the City's industrial water right was previously used to supply water for the Rayonier pulp mill that no longer exists. The City would like to maintain the water right and diversion capacity for this water in order to attract new industry to the area. This industrial reserve capacity totals 44.5 mgd (69 cfs).

5.2.3 WDFW Fish Rearing Channel Demand

The City currently has a water right to divert 32.3 mgd (50 cfs) for fishery use. The WDFW fish rearing channel currently has a contract with the City for 32.3 mgd (50 cfs) from the City's industrial diversion. Based on correspondence with WDFW, the rearing channel requires varying amounts of flow dependent on month of the year as shown in Table 5.1. The long-term water needs of the channel vary between 0.65 mgd (1 cfs) and 23.3 mgd (36.1 cfs). During dam removal and the delta erosion period WDFW has indicated that the peak daily demand for water will be 14.2 mgd (22 cfs). As stated earlier, the wells at the WDFW facility are currently rated for 3.6 mgd (5.6 cfs) but only actually provide 1.7 mgd (2.7 cfs).

(Note: WDFW needs to review and revise water demands in Table 5.1 based on 22 cfs short term demand)

Table 5.1

WDFW MONTHLY WATER DEMANDS

Month	Short-Term Water Demand During Dam Removal in mgd (cfs)	Long-Term Water Demand in mgd (cfs)
January	0.6 (1.0)	0.6 (1.0)
February	4.6 (7.1)	6.3 (9.7)
March	6.6 (10.2)	9.1 (14.1)
April	9.4 (14.6)	13.3 (20.6)
May	14.2 (22.0)	20.1 (31.1)
June	14.2 (22.0)	23.3 (36.1)
July	3.2 (5.0)	3.2 (5.0)
August	21.3 (33.0)*	21.3 (33.0)
September	21.3 (33.0)*	21.3 (33.0)
October	21.3 (33.0)*	21.3 (33.0)
November	0.6 (1.0)	0.6 (1.0)
December	0.6 (1.0)	0.6 (1.0)
Yearly Average	9.8 (15.2)	11.8 (18.2)

*WDFW has indicated that 18.1mgd (28) cfs of the water demand is for dilution of formalin and may be water directly from the river without the need for treatment.

5.2.4 Lower Elwha Klallam Tribe Fish Hatchery Demand

According to the *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1997), the current water demand for the hatchery is approximately 7.4 mgd (11.5 cfs). Based on discussions with the Lower Elwha Klallam Tribe, the hatchery will be expanding and will require varying amounts of water depending on the month of the year. The ultimate projected water demand will vary between 1.8 mgd (2.9 cfs) and 18.6 mgd (28.8 cfs). As a result of additional planning for the hatchery as cited in the *Water Quantity and Quality Parameters, Lower Elwha Klallam Tribal Fish Hatchery* report (Montgomery Watson Harza –MWH-October 2001) water requirements during the short term dam removal and delta erosion period will be a maximum of 12.4 mgd (19.2 cfs) as presented in Table 5.2

Table 5.2

TRIBAL HATCHERY SHORT TERM WATER DEMAND

Month	Water Demand in mgd (cfs)
January	5.7 (8.8)
February	9.4 (14.5)
March	9.8 (15.1)
April	11.7 (18.0)
May	12.4 (19.2)
June	1.7. (2.7)
July	2.8 (4.4)
August	3.1 (4.7)
September	3.3 (5.1)
October	3.6 (5.6)
November	4.2 (6.5)
December	8.3 (12.8)
Yearly Average	6.9 (10.6)

Source: *Water Quantity and Quality Parameters, Lower Elwha Klallam Tribal Fish Hatchery,* MWH, October 2001

A summary of the water demands for the entities evaluated in this report is presented in Table 5.3.

TABLE 5.3

WATER DEMAND SUMMARY

	Current Water Right	Current Average Daily Demand	Current Maximum Daily Demand	Mitigation Period Maximum Daily Demand	Current Contract Amount with City
Port Angeles Municipal	32.2 mgd (50 cfs)	3.4 mgd (5.3 cfs)	10.0 mgd[1] (15.4 cfs)	10.6 mgd (16.4 cfs)	--
Port Angeles Industrial	96.8 mgd (150 cfs)	--	--	--	--
Daishowa	--	9 mgd (13.9 cfs)	14 mgd (21.7 cfs)	14 mgd (21.7 cfs)	20 mgd (31 cfs)
WDFW	--	11.4 mgd (17.6 cfs)	21.3 mgd (33 cfs)[2]	14.2 mgd (22 cfs)[2]	32.3 mgd (50 cfs)
Reserve	--	--	--	--	44.5 mgd (69 cfs)
Tribal Fish Hatchery	Undefined	Varies by month	7.4 mgd (11.4 cfs)[2]	12.4 mgd (19.2 cfs)[2]	--
Totals	**129 mgd (200 cfs)[3]**	**23.8 mgd (36.8 cfs) [3]**	**52.7 mgd (81.5 cfs)**	**51.2 mgd (79.3 cfs)**	**96.8 mgd (150 cfs) [3]**

Notes:
1 Will use the storage water within its water distribution system to supply the current maximum daily demand.
2 Based on Maximum Monthly Demand
3 Does not include Tribal water

The Tribe has identified that their maximum long-term hatchery water supply need will be up to 18.6 mgd (28.8 cfs). With the addition of the Tribe's long-term hatchery water supply to the City's municipal and industrial water right amounts (129 mgd/200 cfs), the total long-term water supply need is 148 mgd (228.8 cfs).

6.1 MUNICIPAL INTAKE OPTIONS

The City of Port Angeles currently uses a Ranney collector to obtain water for municipal use. The construction and capacity of the existing Ranney collector is described in Section 4.1. It is anticipated that the current Ranney collector will continue to be used to supply water for a newly constructed municipal treatment plant. Even though the existing Ranney collector does an excellent job removing suspended solids from the Elwha River as indicated in Section 2.2, it is recommended that two improvements be considered. The existing two 600 Hp pumps (3700 gpm @ 1,530 ft) may need to be adjusted to boost the water to the new treatment plant depending on the site of the plant. A second improvement is that an air scour Ranney backwash system is needed during the first 5-years after the dam's removal due to the potential that sediment released during the dam demolition may affect Ranney capacity unless the Ranney is not used during high turbidity periods.

There is some concern over the declining yield observed in the City's existing Ranney collector as described in Section 4.1.2. The decrease in yield is assumed to be the result of migration of the river channel away from the collector. Continued migration or excessive aggradation of the river bed may potentially impact collector yield. The release of sediments associated with dam removal could accelerate this migration, although it could just as easily remedy the situation by causing the river to move back towards the Ranney collector. The *EIS* (ONP, 1996) proposed the construction of a new Ranney collector on the west side of the river offset possible river migration.

The municipal treatment plant constructed for the City will be capable of treating surface water under the requirements of the SWTR. The City currently has an industrial surface water intake capable of obtaining approximately twice as much as their permitted water right. A simple interconnection between the existing surface water intake or any other industrial intake proposed in this report will easily supplement any decrease in yield from the City's existing collector. An interconnection between the surface water intake, a recommended industrial and fisheries mitigation supply alternative and the existing Ranney collector is proposed as part of the mitigation measures for the City's municipal system. If needed, the National Park Service will assist the City in the development of a Habitat Conservation Plan for operation of the City's municipal and industrial water supply facilities, including provision for relocation of river flow to the east bank of the river in the vicinity of the Ranney well. The construction of an additional collector is unnecessary with these provisions.

6.2 GENERAL TREATMENT OPTIONS

As discussed in Section 3.2, the source water for the Ranney collector well currently used by the City of Port Angeles was classified as GWI in April 2000 by WDOH. Consequently, the proposed water treatment plant should be designed to meet the requirements of the SWTR. Several options to provide the required level of additional treatment have been reviewed and evaluated in consideration of the anticipated impacts of dam removal and the recent GWI classification. These options are based on the treated water production capacity of 10.6 mgd (16.4 cfs). To account for water production losses due to residuals disposal and backwashing the process capacity for each option has been sized for a nominal value of 11 mgd. The options include:

- Conventional Treatment
- Direct Filtration
- Ultra Filtration (UF) Membranes
- High Rate Treatment
- Diatomaceous Earth Filtration
- Slow Sand Filters

All of the treatment processes described below assume the existing Ranney collector will continue to be used as the municipal water intake and act as a pre-treatment process. The water would be pumped from the Ranney collector to a separate treatment facility. All of the treatment processes presented would be followed by a disinfection process. Disinfection process options are described in Section 6.4.

A comparison of the suggested raw water quality requirements for different treatment methods is presented in Table 6.1. This table is for comparative purposes only. Actual maximum values will depend on design and operation specifics.

Table 6.1

SUGGESTED MAXIMUM LIMITS ON RAW WATER QUALITY FOR ALTERNATIVE MUNICIPAL TREATMENT PROCESSES

Water Quality Parameter	Conventional Treatment[1]	Direct Filtration[1]	UF Membranes[1]	Actiflo[2]	Super Pulsator[2]	Diatomaceous Earth[2]	Slow Sand Filters[1]
Turbidity (NTU)	1,000	20	100	4,000	5,000	20	10
Color	1,000	20	15	500	250	5	25
Alkalinity (mg/L)	500	200	150	350	150	20	INA
Hardness (mg/L)	700	150	150	600	200	300	INA
Iron (mg/L)	2	0.5	0.5	10	>1	0.3	1
Manganese (mg/L)	0.5	0.1	0.1	1	>1	0.05	1
TOC (mg/L)	7	2.5	2	40	25+	INA	INA
Taste and Odor	10	4.5	3	INA	INA	INA	INA
Algae (ASU/mL)	10,000	1,000	1,000	30,000	10,000	No Upper Limit/Reduces Cycle	INA
Giardia (100 L)	20	3	100	2×10^8	INA	INA	INA
Cryptosporidium (100 L)	10	1	100	2×10^6	INA	INA	INA
Coliform (#/mL)	1,000,000	1,000	10,000	5,000,000	INA	INA	INA

1. Source : "Integrated Design and Operation of Water Treatment Facilities", Second Edition, Susumu Kawamura, Chapter 2 - Preliminary Studies, page 40, Table 2.4.5-1 Suggested Raw Water Quality for Practical Treatment Processes.
2. Information provided by manufacturer.

INA – information not available.

Each of the treatment options was evaluated based on capital costs, O&M requirements and associated costs, treatment capabilities, specific treatment requirements based on the Ranney collector water quality data, the anticipated effects of dam removal, and land use requirements.

6.2.1 Conventional Treatment

Conventional treatment generally refers to treatment processes consisting of coagulation, flocculation, sedimentation, and filtration. Figure 6.1 depicts a schematic of a typical conventional treatment process train. The design criteria for major unit process are listed below:

Unit Operation	Design Criteria	Range	Typical
1. Flash Mixing	Effective velocity Gradient G (0.5^1) x Mixing Time, T (gs)	300-1,200	1,000
2. Flocculation	Detention Time (t (min)	15-30	30
3. High-rate Settling (Sedimentation)	Surface Load (gpm/ft^2)	2.0-3.5	2.0
4. Filtration	Filtration Rate (gpm/ft ^{2})	1-6	3.0-3.5
	Backwash Rate (gpm/ft^2)	15-23	15
5. Disinfection	Chlorine Dosage (mg/L)	1 – 5	2

A primary oxidant may be used to control bacteria content, alga growth, taste, and odors. Iron and aluminum salts, such as ferric chloride or aluminum sulfate (alum), are commonly used to aid in coagulation of suspended solids to facilitate their removal by settling and filtration. Polymers may also be used in conjunction with or in place of metal salts. Both iron and aluminum salts consume a water's natural alkalinity and depress the pH of the water. Lime, soda ash, or caustic soda is typically added to supplement alkalinity, optimize the coagulation process, raise the pH, and reduce corrosiveness. The addition of coagulants is primarily used to remove suspended solids, but can also be used to remove TOC and color, or precipitate metals. The use of chemical coagulants to optimize the removal of TOC through flocculation and sedimentation is called enhanced coagulation.

Conventional treatment is commonly used for both surface water and groundwater sources, depending on the specific characteristics of the source water. The process can be easily adapted to a wide variety of source waters and can handle varying water qualities that may occur on a seasonal basis. After filtration, a final disinfectant (typically chlorine or chloramines) is added to reduce microbiological content to levels required by applicable health standards. Enhancements to the sedimentation process such as tube or plate settlers can be used to increase loading rates and enhance the efficiency of the process, resulting in smaller structure footprints and reduced structural costs. Filter backwash water is commonly recovered to conserve water and reduce the waste stream from the process that would require disposal.

Depending on the source, taste and odor problems may be treated using oxidants, powdered activated carbon, or other techniques. If dissolved iron and manganese become a problem, potassium permanganate, aeration, ozone, peroxide or chlorine are potential chemical treatment options.

With high quality source waters, the need for flocculation and sedimentation may not be required. The plant could operate as a direct filtration process that is discussed in the next section. The water quality on the Elwha River is expected to be excellent following dam removal and ecosystem recovery except during high run off periods, storms or similar events, as with current conditions. The operation of a conventional treatment plant as a direct filtration plant by by-passing the coagulation and sedimentation processes would reduce operation and maintenance costs during times when the source water quality is good. During high influent turbidity periods, the complete conventional treatment process would be used, particularly if a surface water intake was used to supplement the supply from the Ranney collector. The complete treatment process also provides multiple barriers to prevent the passage of cysts, viruses and similar contaminates to enhance the quality of the potable water from a public health viewpoint.

Treatment residuals are created in the conventional treatment process within the settling basin and during filter backwash and consist of chemical flocculent solids, sediment and similar residuals. The disposal of treatment residuals is discussed later in Section 6.6.

The estimated capital and annual operation and maintenance costs for a 10.6 mgd conventional water treatment plant are presented in Tables 6.2 and 6.3 respectively. For estimating purposes it was assumed that purchasing of liquid sodium hypochlorite (12.5% concentration) would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. All municipal treatment cost estimating details are presented in Appendix E.

Advantages

- Effective for treating water sources with highly variable quality.
- Coagulation process can be optimized to remove suspended solids and turbidity, or optimized to remove TOC or color through enhanced coagulation.
- Tolerant to shock loads of high turbidity with manual or automatic controls to adjust chemical additives.
- Technology is widely used and accepted by regulatory authorities.
- Dissolved iron and manganese can be removed through chemical oxidation and settling process.
- Taste and odor problems can be corrected.
- Can be used as direct filtration plant with a consistent high quality source water.

Disadvantages

- Conventional treatment plants require large land area.
- Treatment residuals require dewatering and disposal
- Requires operator proficiency in water chemistry.
- Higher operation and maintenance complexity compared to membranes

Table 6.2

CONVENTIONAL WATER TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$3,616,000
Operations and Maintenance Facilities	$784,000
Flocculation/Sedimentation Complex	$2,569,000
Filter Complex	$1,649,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Wash Water Recovery Basin	$729,000
Chlorine Building	$132,000
Decant Pump Station	$151,000
Sedimentation Ponds	$269,000
Subtotal	$11,211,000
Contingency (40%)	$4,484,000
Subtotal	$15,695,000
Engineering, Survey, and Construction Management (20%)	$3,139,000
Project Total	**$18,834,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar.
[2] Costs are based on the first quarter of year 2001 prices.

Table 6.3

CONVENTIONAL WATER TREATMENT PLANT ESTIMATED ANNUAL O&M COST

	Total Annual Treatment Costs
Labor	$476,000
Operation	$245,000
Maintenance	$60,000
Professional Services	$45,000
Other	$55,000
Subtotal	$881,000
10% Contingency	$89,000
Total	**$970,000**

Note:
[1] Costs are based on the first quarter of year 2001 prices.

6.2.2 Direct Filtration

Raw water with turbidity, color, taste, and odor that are low or unobjectionable may be treated by direct filtration. Figure 6.2 depicts a schematic of a typical direct filtration process train.

This treatment process is very similar to conventional treatment but sedimentation and in some cases flocculation may be eliminated if the source water is of high quality. A chemical coagulant or filter aid may be required to improve the performance of the filters. Due to the elimination of the sedimentation basins, both the capital and operation and maintenance costs are considerably lower when compared to conventional treatment.

Direct filtration is traditionally used for consistently high quality water sources. The high anticipated fluctuations in water quality within the Elwha River during dam removal and the unknown performance of the Ranney collector as a prescreen, may make this treatment process less flexible and less reliable than other options.

Treatment residuals are created in the direct filtration process only during filter backwash, which results in substantially less residuals disposal than conventional treatment. The disposal of treatment residuals is discussed in Section 6.6.

Estimated capital costs and annual operation and maintenance costs for a 10.6 mgd direct filtration plant are presented in Tables 6.4 and 6.5 respectively. For estimating purposes it was assumed that purchasing of liquid sodium hypochlorite (12.5% concentration) would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. All municipal treatment cost estimating details are presented in Appendix E.

Table 6.4

DIRECT FILTRATION WATER TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$2,774,000
Operations and Maintenance Facilities	$784,000
Flocculation Complex	$669,000
Filter Complex	$1,624,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Wash Water Recovery Basin	$729,000
Chlorine Building	$132,000
Decant Pump Station	$151,000
Sedimentation Ponds	$269,000
Subtotal	$8,444,000
Contingency (40%)	$3,378,000
Subtotal	$11,822,000
Engineering, Survey, and Construction Management (20%)	$2,364,000
Project Total	**$14,186,000**

Notes:
1 Costs do not include purchase of land, easements, and similar.
2 Costs are based on the first quarter of year 2001 prices.

Table 6.5

DIRECT FILTRATION WATER TREATMENT PLANT ESTIMATED ANNUAL O&M COST

	Total Annual Treatment Costs
Labor	$476,000
Operation	$245,000
Maintenance	$60,000
Professional Services	$45,000
Other	$55,000
Subtotal	$881,000
10% Contingency	$88,000
Total	**$969,000**

Notes:
[1] Costs are based on first quarter of year 2001 prices.

Advantages

- Lower capital costs
- Less operation and maintenance cost and personnel time
- Less residuals disposal compared to conventional treatment
- Requires smaller land area than conventional treatment

Disadvantages

- Requires source water with consistently high quality
- Not suitable for high solids loading or highly varying water quality
- Requires higher level of operator attention to account for lower reliability
- May require known water quality for acceptance by WDOH

6.2.3 Membranes

Membranes represent a physical process for treating drinking water rather than a chemical process. Membrane processes are generally categorized according to driving force, membrane type and configuration, and removal capabilities. Those generally classified as pressure-driven processes include:

- Reverse Osmosis (RO)
- Nanofiltration (NF)
- Ultrafiltration (UF)

- Microfiltration (MF)

In these processes, pressurized feed water enters vessels containing membranes that are permeable to water molecules but preclude substances greater than a specified size. MF and UF processes separate substances from feed water through a sieving action. Separation depends on the membrane pore size and interaction with entrained material on the membrane surface. NF and RO processes separate substances through a thin, dense, semi-permeable membrane barrier as well as by sieving action. The required membrane feed pressure generally increases as removal capability increases.

Membrane processes classified as voltage-driven include:

- Electrodialysis (ED)
- Electrodialysis Reversal (EDR)

These processes utilize alternating anion and cation transfer ion exchange membranes in flat sheet form placed between positive and negative electrodes. The application of voltage across the electrodes results in positively charged ions moving towards the negative electrode and negatively charged ions moving towards the positive electrode. This effect causes alternating compartments to become demineralized and the other compartments to become concentrated with ions. EDR is a variation of the ED process where electrodes are reversed on a set frequency to “electrically flush” the membranes to control scaling and fouling.

The electrodialysis process is very costly and rarely used in municipal water treatment applications. The process is generally utilized for source waters high in salinity and would be applicable if a desalination plant was required as an alternative water source. Desalination is not considered a feasible alternative for either municipal or industrial treatment based on expense and complexity.

Based on a preliminary review of the current and expected source water quality and overall water treatment objectives, the membrane process using microfiltration was selected for further evaluation. Figure 6.3 depicts a schematic of a typical membrane treatment process.

Membranes typically process between 85-90% of the influent water. The remaining water is rejected as waste and can be further recovered with additional membranes or must be disposed. Typically this waste does not have any treatment chemicals present. Additional treatment residuals are created when the membranes are cleaned. This backwash water is typically acidic or basic and is usually treated prior to disposal. The amount of backwash water generated is significantly less than the quantity of reject water generated. The disposal of treatment residuals is discussed in Section 6.6.

Estimated capital costs and annual operation and maintenance costs for an 10.6 mgd membrane plant are presented in Tables 6.6 and 6.7 respectively. For estimating purposes it was assumed that purchasing of liquid sodium hypochlorite (12.5% concentration) gas would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. All municipal treatment cost estimating details are presented in Appendix E.

Table 6.6

MEMBRANE TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$4,199,000
Operations and Maintenance Facilities	$784,000
Membrane Complex	$6,519,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Wash Water Recovery Basin	$729,000
Chlorine Building	$132,000
Subtotal	$13,675,000
Contingency (40%)	$5,470,000
Subtotal	$19,145,000
Engineering, Survey, and Construction Management (20%)	$3,825,000
Project Total	**$22,970,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar.
[2] Costs are based on the first quarter of year 2001 prices.

Table 6.7

MEMBRANE TREATMENT PLANT ESTIMATED ANNUAL O&M COST

	Total Annual Treatment Costs
Labor	$476,000
Operation	$203,000
Maintenance	$60,000
Professional Services	$45,000
Other	$55,000
Subtotal	$839,000
10% Contingency	$84,000
Total	**$923,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar.

Advantages

- Effective for treating sources of highly variable quality.
- Limited chemical handling or optimization of chemical dosing.
- Provides very effective removal of suspended solids, turbidity, and Giardia and Cryptosporidium-sized particles.

- Requires smaller footprint than most other forms of water treatment plants.
- Typically requires less manpower to operate.
- Reject water can be chemical free and potentially discharged to local water bodies.

Disadvantages

- Not effective for the removal of dissolved constituents in the water such as TOC, iron, and manganese without preliminary treatment.
- Bacteria, chlorine residual, and polymers can foul or damage membranes.
- Membrane backwash water may require further treatment prior to disposal.
- For low quality source waters, pretreatment requirements can be similar to those required for conventional treatment.

6.2.4 High Rate Treatment (Proprietary)

Two proprietary high rate flocculation/clarification systems were evaluated for municipal treatment alternatives. Systems evaluated were as follows:

- Microsand ballasted coagulation clarification (ACTIFLO by US Filter)
- Pulsed blanket clarifier (Super Pulsator by Ondeo Degremont, Inc.)

6.2.4.1 ACTIFLO

The Actiflo process is a compact clarification system using microsand-enhanced flocculation and clarification. Figure 6.4 depicts a schematic of the Actiflo treatment process. The manufacturer's literature on the Actiflo process is included in Appendix F.

A coagulant such as alum is added to the untreated water in a separate coagulation tank. The coagulated water then enters a second tank called an injection tank where microsand (60-120 µm) and polymer are added. The microsand provides a large contact area and acts as ballast therefore accelerating the settling of floc. The destabilized suspended solids bind to the microsand through polymer bridges. In the third tank, the particles agglomerate together and grow into high density floc known as microsand ballasted floc that settle quickly to the bottom of the lamella tube settling tank. A filtration process is required following the Actiflo treatment. The filters would be the same size and design as used in the conventional treatment alternative.

The sludge/microsand mixture collected at the bottom of the settling tank is pumped to hydrocyclones where the sludge is separated from the microsand. The recovered microsand is then recycled to the injection tank whereas the separated sludge is continuously discharged to the solids handling process. The Actiflo process has been shown to utilize less coagulation chemicals than traditional conventional treatment plants and therefore typically produces fewer residuals that would require disposal. The disposal of treatment residuals is discussed in Section 6.6.

Like traditional conventional treatment plants, an Actiflo plant can be run as a direct filtration process if the water quality from the Ranney collector remains consistently high in quality. The Actiflo process has the flexibility of by-passing the coagulation, flocculation, and sedimentation basin if water quality permits, and thus decrease the operation and maintenance costs.

Estimated capital costs and annual operation and maintenance costs for an 10.6 mgd Actiflo plant are presented in Tables 6.8 and 6.9 respectively. For estimating purposes it was assumed that liquid sodium hypochlorite (12.5% concentration) would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. All municipal treatment cost estimating details are presented in Appendix E.

Table 6.8

ACTIFLO WATER TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$3,183,000
Operations and Maintenance Facilities	$784,000
High Rate Clarification	$2,051,000
Filter Complex	$1,624,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Wash Water Recovery Basin	$729,000
Chlorine Building	$132,000
Decant Pump Station	$151,000
Sedimentation Ponds	$269,000
Subtotal	$10,235,000
Contingency (40%)	$4,094,000
Subtotal	$14,329,000
Engineering, Survey, and Construction Management (20%)	$2,866,000
Project Total	**$17,195,000**

Notes:
1 Costs do not include purchase of land, easements, and similar.
2 Costs are based on the first quarter of year 2001 prices.

Table 6.9

ACTIFLO WATER TREATMENT PLANT ESTIMATED ANNUAL O&M COST

	Total Annual Treatment Costs
Labor	$476,000
Operation	$235,000
Maintenance	$60,000
Professional Services	$45,000
Other	$55,000
Subtotal	$871,000
10% Contingency	$87,000
Total	**$958,000**

Notes:
1 Costs do not include purchase of land, easements, and similar.

Advantages

- Effective at treating source waters of highly variable quality including low turbidity water
- Lower capital cost than traditional conventional treatment
- Power costs are comparable to traditional conventional treatment
- Less chemical coagulants typically required, that translates to lower chemical costs
- Less treatment residuals typically generated
- Smaller facility footprint compared to traditional conventional treatment
- Maybe shutdown and restarted quickly
- Provides same flexibility as traditional conventional treatment for removal of dissolved constituents and treatment of high turbidity spikes
- Can be operated as a direct filter plant to reduce O&M costs if source water quality permits

Disadvantages

- Treatment residuals require dewatering and disposal
- Requires operator proficiency in water chemistry
- Higher operation and maintenance complexity compared to membranes

6.2.4.2 SUPER PULSATOR

The Super Pulsator is a high-rate clarifier that combines clarification and flocculation in the same treatment unit. Figure 6.5 depicts a schematic of the Super Pulsator treatment process.

A coagulant is added to the untreated water prior to rapid mixing. The coagulated water is then directed into a sealed vacuum chamber that controls flow into the clarifier's distribution duct. From the distribution duct, the water flows to distribution laterals that are evenly spaced over the clarifier floor. Vacuum pumps in the vacuum chamber cause the water level to rise. A timer-actuated vent valve vents the vacuum chamber to atmosphere. As the water level falls in the vacuum chamber, a pulse of water uniformly expands the entire surface of the sludge blanket, which is comprised of previously formed solids. The clarified effluent is collected in evenly spaced laterals that span the clarifier surface and connect to the effluent collection channel. Sludge concentrators, which also act as internal weirs, control the height of the sludge blanket and collect sludge. The concentrators are periodically emptied via sludge collection piping. The disposal of treatment residuals is discussed in Section 6.6.

Estimated capital costs and annual operation and maintenance costs for an 10.6 mgd Super Pulsator plant are presented in Tables 6.10 and 6.11 respectively. For estimating purposes it was assumed that the purchasing of liquid sodium chloride (12.5% concentration) would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. Municipal treatment estimating details are presented in Appendix E.

Table 6.10

SUPER PULSATOR TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$2,928,000
Operations and Maintenance Facilities	$784,000
Rapid Mixing Complex	$166,000
High Rate Clarification	$1,531,000
Filter Complex	$1,624,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Wash Water Recovery Basin	$729,000
Chlorine Building	$132,000
Decant Pump Station	$151,000
Sedimentation Ponds	$269,000
Subtotal	$9,626,000
Contingency (40%)	$3,850,000
Subtotal	$13,476,000
Engineering, Survey and Construction Management (20%)	$2,695,000
Project Total	**$16,171,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar.
[2] Costs are based on the first quarter of year 2001 prices.

Table 6.11

SUPER PULSATOR TREATMENT PLANT
ESTIMATED ANNUAL O&M COST

	Total Annual Treatment Costs
Labor	$476,000
Operation	$245,000
Maintenance	$60,000
Professional Services	$45,000
Other	$55,000
Subtotal	$881,000
10% Contingency	$89,000
Total	**$970,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar.

Advantages

- No under water moving parts
- Each process train requires only one basin
- Requires 1 hp per mgd of water treated
- Sludge blanket allows some fluctuation in influent turbidity
- PAC will be retained for a longer detention time in sludge blanket and hence is more efficiently used
- Uniform distribution of flocculation energy (reduced short circuiting)
- Sludge blanket cannot be lost due to operator error or malfunction of sludge blowdown system
- Smaller footprint compared to conventional treatment plant
- Ability to operate without polymer at lower hydraulic loading, rate

Disadvantages

- Treatment residuals require dewatering and disposal
- Requires operator proficiency in water chemistry
- Higher operation and maintenance complexity compared to membranes
- Treatment can be adversely affected by sudden change in water temperature
- Operation must be continuous to maintain the sludge blanket and treatment
- Difficult to maintain treatment for initially low turbidity water

6.2.5 Diatomaceous Earth Filters

Diatomaceous earth (DE) filters, the most common type of precoat filters, have been used effectively for the treatment of drinking water since 1942. In that year the U.S. Army adopted the process as a standard method of treatment largely due to its effectiveness in removing cysts. The precoat operation draws its name from the process of coating filter leaves with approximately 1/8" of material at the initiation of each filter operating cycle. Although diatomaceous earth, mined from the fossilized remains of microscopic plants called diatoms, is the most common precoat material used, other precoat materials such as ground perlite performs well in other applications. Figure 6.6 depicts a schematic of a DE treatment process.

In the DE filtration process, untreated water is passed through a uniform layer of the filter media that has been deposited (precoated) on a septum, a permeable material that supports the filter media. As the untreated water passes through the filter media (diatomaceous earth) and the septum, most of the suspended particles are removed and remain at the surface of the filter media layer. As the filter process continues, additional filter media, called body feed, is metered into the influent to maintain the permeability of the filter media as the process continues and the thickness of the media and accumulated filtered material (cake) increases. At a point that the

cake reaches a thickness where continued filtration is impractical due to the increasing pressure to push water through the media, the cake is removed and disposed. A new layer of precoat is reapplied on the septum and the filtration process starts over. The primary sources of DE are located in California.

Treatment residuals consist of the diatomaceous earth filter media and filtered solids. The disposal of treatment residuals is discussed in Section 6.6.

There are generally two basic groups of DE filter systems. Those that force water through the filter under pressure are enclosed vessels. Filters operated under vacuum may utilize open vessels.

Estimated capital costs and annual operation and maintenance costs for an 10.6 mgd typical pressure driven DE filtration system are presented in Tables 6.12 and 6.13 respectively. For estimating purposes it was assumed that purchasing of liquid sodium hypochlorite (12.5% concentration) would be used for disinfection, and sedimentation ponds would be used for residuals handling. Both disinfection options and residuals handling options are discussed in subsequent sections. Municipal treatment cost estimating details are presented in Appendix E.

Table 6.12

DIATOMACEOUS EARTH WATER TREATMENT PLANT ESTIMATED CAPITAL COST

General Items for WTP	$3,090,000
Operations and Maintenance Facilities	$784,000
Diatomaceous Earth Complex	$3,742,000
Clearwell and Effluent Pumping Facilities	$1,312,000
Chlorine Building	$132,000
Subtotal	$9,060,000
Contingency (40%)	$3,624,000
Subtotal	$12,684,000
Engineering, Survey, and Construction Management (20%)	$2,537,000
Project Total	**$15,221,000**

Notes:

1 Costs do not include purchase of land, easements, and similar
2 Cost are based on the first quarter of year 2001 prices

Table 6.13

DIATOMACEOUS EARTH WATER TREATMENT PLANT ESTIMATED ANNUAL O&M COSTS

	Total Annual Treatment Costs
Labor	$420,000
Operation	$281,000
Maintenance	$35,000
Professional Services	$45,000
Other	$55,000
Subtotal	$836,000
10% Contingency	$84,000
Total	**$920,000**

Notes:
[1] Costs do not include purchase of land, easements, and similar

Advantages

- Treatment costs may be considerably less than conventional treatment since coagulation, sedimentation, and granular media filtration are not required.
- No chemical handling or optimization of chemical dosing.
- The waste filter media is easily dewatered, and in some cases can be reclaimed for other uses such as soil conditioning or landfill cover.
- Effective in Giardia and similar small particle removal.

Disadvantages

- Generally more effective in high quality surface waters with turbidities less than 10 NTU. The process is not suitable for algae, color, taste, dissolved organics or soluble iron and manganese problems without conventional rapid mix, flocculation, and sedimentation facilities preceding the filters.
- Desired results require proper operation with respect to the application and replenishing of the filter cake.
- Pressurized process requiring added pumping costs.
- Filters are subject to shut down then recoating with DE after any power disruption or substantial pressure fluctuation.
- Filter is enclosed and process and status of DE coating is not visible to the operator.

6.2.6 Slow Sand Filters

Slow sand filters are sand filters operated at very low filtration rates without the use of a coagulant. In a typical slow sand filter, most of the solids are removed in a thin layer on top of the filter bed. This layer, composed of dirt and living and dead micro- and macro-organisms from the untreated water (the schmutzdecke), becomes the dominant filter medium in the process. Slow sand filters have cycle lengths varying from 1 to 6 months and are periodically cleaned as head loss through the filter rises. Cleaning is by draining and physically removing the schmutzdecke and up to 2 inches of sand. After a number of cleanings the sand is replenished.

The filtration rate of slow sand filters is 50 to 100 times slower than that of ordinary rapid sand and high-rate filters. Consequently, land requirements are significant. The filter area required for a 10.6 mgd plant would be approximately up to 3.4, acres this does not include the support buildings, disinfectant process, or treatment residuals disposal. The filters would also need to covered. Slow sand filters for treatment capabilities of 10.6 mgdl are not common because of the land requirements. Slow sand filters are not considered a feasible alternative for the City of Port Angeles municipal supply.

6.3 RECOMMENDED TREATMENT OPTION

Table 6.14 is a summary of the capital cost, annual operation and maintenance costs, advantages, and disadvantages of each of the municipal treatment technologies discussed in Section 6.2. Slow sand filters have been excluded from this summary because it was determined that this technology was unfeasible based on the required filter area.

Table 6.14 also includes the present worth value of the capital and operating cost of each treatment option.

Table 6.14

SUMMARY OF MUNICIPAL TREATMENT ALTERNATIVES

Treatment Process	Capital Cost[1]	O&M Cost[1]	20 Year Present Worth[2]	Advantages	Disadvantages
Conventional Treatment	$18,834,000	$970,000	$29,960,000	• Effective for treating water sources with highly variable quality. • Coagulation process can be optimized to remove suspended solids and turbidity, or optimized to remove TOC or color through enhanced coagulation. • Tolerant to shock loads of high turbidity with manual or automatic controls to adjust chemical additives. • Technology is widely used and accepted by regulatory authorities. • Dissolved iron and manganese can be removed through chemical oxidation and flocculation and settling process. • Taste and odor problems can be removed.	• Conventional treatment plants require large land area. • Treatment residuals require dewatering and disposal • Requires operator proficiency in water chemistry. • Higher operation and maintenance complexity compared to membranes

Treatment Process	Capital Cost[1]	O&M Cost[1]	20 Year Present Worth[2]	Advantages	Disadvantages
				• Can be used as direct filtration plant with a consistent high quality source water.	
Direct Filtration	$14,186,000	$969,000	$25,300,000	• Lower capital costs • Less operation and maintenance cost and personnel time • Less residuals disposal compared to conventional treatment • Requires smaller land area than conventional treatment	• Requires source water with consistently high quality • Not suitable for high solids loading or highly varying water quality • Requires higher level of operator attention to account for lower reliability • May require known water quality for acceptance by WDOH
Membranes without Pretreatment	$22,970,000	$923,000	$33,557,000	• Effective for treating sources of highly variable quality. • Limited chemical handling or optimization of chemical dosing. • Provides very effective removal of suspended solids, turbidity, and Giardia and Cryptosporidium-sized particles. • Requires smaller footprint than most other forms of water treatment plants. • Typically requires less manpower to operate. • Reject water can be chemical free and potentially discharged to local water bodies.	• Not effective for the removal of dissolved constituents in the water such as TOC, iron, and manganese without preliminary treatment. • Bacteria, chlorine residual, and polymers can foul or damage membranes. • Membrane backwash water may require further treatment prior to disposal. • For low quality source waters, pretreatment requirements can be similar to those required for conventional treatment.
Membranes with Pretreatment	$27,289,000	$978,000	$38,507,000		
Actiflo	$17,195,000	$958,000	$28,183,000	• Effective at treating source waters of highly variable quality including low turbidity water • Lower capital cost than traditional conventional treatment • Power costs are comparable to traditional conventional treatment • Less chemical coagulants typically required, that translates to lower chemical costs • Less treatment residuals typically generated • Smaller facility footprint compared to traditional conventional treatment • Maybe shutdown and restarted quickly • Provides same flexibility as traditional conventional treatment for removal of dissolved constituents and	• Treatment residuals require dewatering and disposal • Requires operator proficiency in water chemistry • Higher operation and maintenance complexity compared to membranes

Treatment Process	Capital Cost[1]	O&M Cost[1]	20 Year Present Worth[2]	Advantages	Disadvantages
				treatment of high turbidity spikes • Can be operated as a direct filter plant to reduce O&M costs if source water quality permits	
Super Pulsator	$16,171,000	$970,000	$27,297,000	• No under water moving parts • Each process train require only one basin • Requires 1 hp per mgd of watch treated • Sludge blanket allows some fluctuation in influent turbidity • PAC will be retained for a longer detention time in sludge blanket and hence is more efficiently used • Uniform distribution of flocculation energy (reduced short circuiting) • Smaller footprint compared to conventional treatment plant • Ability to operate without polymer at lower hydraulic loading, rate	• Treatment residuals require dewatering and disposal • Requires operator proficiency in water chemistry • Higher operation and maintenance complexity compared to membranes • Treatment can be adversely affected by sudden changes in water temperature • Operation must be continuous to maintain the sludge blanket and treatment • Difficult to maintain treatment for initially low turbidity water
Diatomaceous Earth	$15,221,000	$920,000	$25,773,000	• Treatment costs may be considerably less than conventional treatment since coagulation, sedimentation, and granular media filtration are not be required. • No chemical handling or optimization of chemical dosing • The waste filter medium is easily dewatered, and in some cases can be reclaimed for other uses such as soil conditioning or landfill cover. • Effective in Giardia and similar small particle removal.	• Generally more effective in high quality surface waters with turbidities less than 10 NTU. The process is not suitable for algae, color, taste, dissolved organics or soluble iron and manganese problems without conventional rapid mix, flocculation and sedimentation facilities preceding the filters. • Desired results require proper operation with respect to the application and replenishing of the filter cake. • Pressurized process requires added pumping costs. • Filters are subject to shutdown then recoating after any electric power disruption or substantial fluctuation of pressure.
Diatomaceous Earth with Preliminary Treatment	$19,536,000	$975,000	$30,719,000		

Notes:

[1] Treatment process costs assume that:

[a.] purchasing liquid sodium hypochlorite (12.5%), however, other disinfection alternative costs such as sodium hypochlorite, on-site generator, using chlorine gas, and UV disinfection are provided in Section 6.5, Recommended Disinfection Option;

[b.] residual disposal through sedimentation pond and landfill, however, the cost for using gravity thickener plus belt filter and landfill is provided in Section 6.7 recommended Treatment Residual Disposal.

[c] costs are based on the first quarter of year 2001 prices and do not include purchase of land, easements, and similar.

[2] Present worth is based on annual compounding discount rate of 6% for a 20-year period.

[3] The costs for both membrane and diatomaceous earth in this table are based on no pretreatment facilities being used. Pretreatment includes a rapid mix unit, and flocculation/sedimentation prior to the filter unit. To consider these pretreatment costs, use the following table:

Treatment Process	Subtotal	Construction Contingency 40%	Engineering Etc. 20%	Total	20-year Present Worth
Membrane in the table above	$13,675,000	$5,470,000	$3,829,000	$22,970,000	$33,557,000
Membrane with pretreatment facilities	$16,243,700	$6,497,500	$4,548,000	$27,289,200	$38,507,000
Diatomaceous earth in the table above	$9,060,000	$3,624,000	$2,537,000	$15,221,000	$25,776,000
Diatomaceous earth with pre-treatment facilities	$11,629,000	$4,651,000	$3,256,000	$19,536,000	$30,719,000

The recommended process for municipal treatment is a coagulation-sedimentation-filtration process to treat water from the City's existing Ranney collector or surface water. There are three such treatment processes described above, the "traditional" conventional treatment plant, the Actiflo ballasted flocculation treatment plant, and the high rate clarifier Super Pulsator process. All of these processes use coagulation chemistry followed by filtration to treat water, which allows for the greatest flexibility and reliability for treating a source water of unknown or highly variable quality. The major difference is the way each of these processes get suspended particles to coagulate and then settle. Either the conventional treatment or one of the high rate conventional treatment processes would be a technically appropriate treatment process to mitigate against the adverse impacts of dam removal, and these processes would meet all of the requirements of the SWTR, but at this time URS recommends the Actiflo process over all of the other treatment process investigated for the following reasons:

- Lower capital cost than traditional conventional treatment
- Lower chemical cost
- Smaller facility footprint
- Ease of operation
- Ability to treat both high and low turbidity water
- Higher treatment performance in side by side comparison
- Similar power cost

To confirm the recommendation of Actiflo over the other high rate process Super Pulsator, URS visited and made telephone contacts of existing operations of these two high rate treatment processes and potential pilot testing to determine the following:

- Cost of operation
- Ease of operation
- Stability of the process
- Chemical usage
- Performance at high and low TSS levels
- Sludge characteristics and volume
- Loss of sand for Actiflo
- Capability to remove iron, manganese, color, taste and odor

The Super Pulsator plant in Green River Wyoming and the Actiflo plant in Golden, Colorado were visited by URS, Reclamation, and City personnel. URS contacted two other Super Pulsator plants in North Carolina. The visits and telephone contacts indicated that the Actiflo process is better suited to the Elwha River application due to its ability to treat low turbidity water without extensive operator attention and because it can be turned off and restarted with the process operating in a stable mode in typically less than 30 minutes. The ease of operation for Actiflo was apparent and loss of sand was not indicated as significant at the Golden facility.

Pilot testing of the Actiflo process will be used to satisfy the DOH requirement for testing treatment processes and confirm its suitability for the Elwha River application.

The conceptual level cost estimate of the two alternatives showed that a 10.6 mgd Actiflo plant is approximately $1.6 million less expensive to construct than a conventional treatment plant. Much of the cost savings are due to the smaller size of the treatment basins, facility structure, and land requirements.

Significant information has been developed on the performance of the Actiflo process compared to conventional treatment plants. This information has been supplied by the manufacturer and reviewed by URS. The comparison information is presented in Appendix F.

According to the information, the combination of efficient mixing and microsand ballasted flocculation make Actiflo very effective in treating low temperature, low turbidity raw water that is often difficult to coagulate using traditional methods. Actiflo is often capable of producing <1 NTU clarified water from a wide range of raw water turbidities. The ability to effectively remove turbidity prior to the filters from difficult to treat influent results in lower filter loading and increased filter run-times.

The advantages of microsand enhanced flocculation provide for consistently high quality clarified water under a variety of treatment conditions including significant unexpected turbidity spike events or seasonally changing water conditions. The Actiflo process has also been shown to effectively treat extremely high, sudden turbidity spikes. The use of microsand in the process results in a relatively constant suspended solids concentration in the system, thus, extremely high or sudden suspended solids concentrations are effectively dampened by the already high suspended solids concentration normally maintained within the process. The overall result is stable treatment performance at influent water turbidities in excess of 1,000 NTU.

Studies were performed to compare the amount of coagulation chemicals required for the Actiflo process compared to conventional treatment for a variety of source waters in the United States. The studies showed a 30-50% reduction in the amount of alum required to effectively clarify the untreated water. This can translate into a significant operation and maintenance cost savings.

Like conventional treatment, the chemical dosing in an Actiflo plant can be optimized to remove iron, manganese and/or TOC. In fact, studies comparing Actiflo to conventional treatment for the removal of TOC for various water sources in the United States and Canada show that Actiflo was capable of reducing TOC by 17-78% while conventional treatment could only obtain 8-49% reductions (see Appendix F).

A study was conducted on a 10 mgd treatment plant in Golden, Colorado to determine the power requirement of the Actiflo process compared to the conventional plant. The conventional plant had a 5 horsepower (hp) flash mixer, and a 1 hp flocculation motor. The Actiflo plant, treating the same amount of water, required a 3 hp coagulation motor, a 3 hp injection tank motor, a 5 hp maturation tank motor, a 1 hp scraper, and 2-5 hp sand slurry pumps. The comparison was conducted for nine months and found that the power costs for the two processes were relatively identical despite the increased hp required for Actiflo. The reason for this was that the amount of backwash water for the Actiflo plant is less than that for the conventional plant. All the filter backwash water was pumped using a 75 hp motor. The filter turbidity loading of the Actiflo process was considerably less than for conventional treatment, because of the effectiveness of Actiflo to reduce turbidity levels to below 1 NTU through clarification before filtration. The less turbidity that goes onto the filter means the less backwashing and cleaning required. These longer filter runs resulted in reduced power consumption.

The Actiflo process would be followed by filtration through a multi or dual-media filter. The filter media consists of at least anthracite, sand and gravel layers. In order to clean the filter, an air-water backwashing system would be developed to fluidize the media bed and flush out filtered particles for disposal. The disposal of the treatment residuals is discussed in Section 6.6.

It was assumed that the chemicals used for the coagulation and flocculation would be caustic soda, alum and polymer. Provisions for alkalinity adjustments have not been provided for in the preliminary design or costing. The need to use alkalinity adjustments in the treatment process can be determined through laboratory bench scale testing during the design phase and added to the final design.

The existing Ranney collector has a capacity of 10.7 mgd (16.6 cfs) which meets the projected 20-year demand as described in Section 4. The Actiflo plant will treat water from this existing collector. A cross connection to the industrial intake facilities will be provided to supplement a potential reduction in yield from the existing Ranney collector. The Actiflo process will meet all the treatment requirements of the SWTR and be capable of treating Elwha River surface water if required. The cross connection from the proposed industrial intake and the municipal system are shown on the industrial intake figures.

6.4 DISINFECTION OPTIONS

Disinfection technologies readily available and in common use each have their strengths and weaknesses in treating municipal drinking water. The following section describes some of the available disinfection technologies. Along with the brief profiles of the varying technologies, factors such as their relative effectiveness, formation of disinfection by-products (DBPs), operational complexity, safety risk and relative cost are summarized in Table 6.15.

6.4.1 Chlorine Gas (Bulk Liquid)

A chlorine gas disinfection system is currently in use by the City of Port Angeles. Chlorine gas is produced at chlor-alkali plants and shipped to water treatment plants as a liquid in pressurized bulk containers. For more than a century chlorine gas has been used successfully to disinfect drinking water. When added to water, chlorine forms hypochlorous acid, an active disinfectant.

The main capabilities of this disinfectant are:

- Destruction of a broad range of microorganisms, including bacteria, viruses and some protozoa.

- Controls many taste, color and odor problems in untreated water by oxidation of constituents that cause these problems.
- With proper dosages, remains as chlorine residual in water distribution systems to protect against growth of biofilm or microorganisms. This residual can serve as an indicator of water quality.

Chlorine gas is the most widely used form of disinfection used in the United States. Although chlorine gas has a broad range of capabilities at a cost-effective price, there are concerns associated with the hazards of transportation and storage of chlorine gas, the possible creation of harmful DBP's, and its weakness in inactivating Cryptosporidium.

Chlorine gas can also be generated on-site to eliminate the risk of transportation. Due to the high capital costs and operation and maintenance issues associated with having a small chemical plant on-site, chlorine gas generation for the City of Port Angeles was not considered feasible.

A comparison of chlorine gas compared to other disinfectants is presented in Table 6.15.

Table 6.15

COMPARISON OF DISINFECTION PROCESSES

Disinfection Process	Disinfection Effectiveness			By-Product Formation			Oxidation	Safety Risk	Complexity	Cost[2] $/gal
	Bact/ Virus	Cysts	Residual	Organic	Brominated	Inorganic				
Chlorine Gas	Very Good	Fair	Good	High	High	No	Good	High	Low	6
Hypochlorite	Very Good	Fair	Good	High	High (Bromate)	High	Good	Medium	Low	9
Chloramines	Fair	Very Poor	Excellent	Medium	No	No	Poor	Low	Medium	9
Ozone	Excellent	Excellent	No	Low	High (Bromate)	Medium	Very Good	High	High	130
Ultraviolet	Good	(Under Study)	No	No	No	No	--	Low	Low	60

1. Table from *Water Engineering & Management*, January 2001, Vol. 148, No. 1, pp. 13-16.
2. Relative cost comparison, cost dependant on installation size.

6.4.2 Hypochlorites

Both sodium hypochlorite, and calcium hypochlorite offer an excellent alternative approach to disinfection. The active ingredient in both compounds is the hypochlorite ion, which hydrolyzes to form hypochlorous acid.

Sodium hypochlorite (bulk liquid), often called liquid bleach, is considered to the second cheapest disinfectant after bulk liquid chlorine gas. Commercially available as a 12.5% solution, it offers most of the advantages of chlorine gas yet it does not have transportation or storage hazards to the extent present with chlorine gas.

Bulk sodium hypochlorite has two problems. First, it tends to decompose in storage depending on the storage temperature, its age, concentration, and contaminants it may contain. A much larger issue is the possible presence of bromates, this EPA-regulated DBP can come from bromide impurities that may be in the sodium chloride from which sodium hypochlorite is made.

On-site generation of sodium hypochlorite is possible but has a high initial capital cost and requires routine maintenance. On-site generation of sodium hypochlorite was not considered feasible for Port Angeles.

Calcium hypochlorite is normally delivered to water treatment plants in a powder or granular form and mixed with water for application. It is often supplied in bags, briquettes, or other solid forms that are used in erosion type feeders. In smaller quantities, it about twice as expensive as sodium hypochlorite. Nonetheless, it is often preferred, primarily in smaller water treatment plants, because it is more stable and produces far less inorganic DBPs. In smaller amounts, it also is easier to handle and store.

Calcium hypochlorite requires special storage care to avoid contact with organic materials. These two substances can generate enough heat and oxygen to start a fire. When mixed with water, calcium hypochlorite causes an exothermic reaction and hence may create a hazard. To prevent excessive heat, the dry chemical should always be added to the correct amount of water, rather than water added to the chemical.

A comparison of hypochlorites to other disinfectants is presented in Table 6.15.

6.4.3 Chloramines (Ammonia-Chlorine Process)

This process involves the addition of ammonia and chlorine compounds separately to a water treatment system. The two ingredients (usually, anhydrous ammonia and hypochlorous acid) react to form chloramines. The ingredients also can be ammonium salts and liquid hypochlorites. This treatment procedure also is called chloramination or the chloramine process.

Compared to chlorine gas, using chloramines as the primary disinfectant produces fewer DBPs and does not combine with organics in the water to form trihalomethane. The chloramine process may be used as secondary disinfectant to provide a longer lasting residual in the distribution system, if desired.

A comparison of chloramines to other disinfectants is presented in Table 6.15.

6.4.4 Chlorine Dioxide

Chlorine dioxide is usually produced on-site by mixing chlorine gas with sodium chlorite. It is recognized as an efficient oxidizer and a broad-spectrum, fast acting biocide. It is used primarily for pretreatment of surface waters that have odor and taste problems, or are high in manganese content. Chlorine dioxide cannot be transported as a compressed gas; it has to be generated on-site. Its use and generation requires skilled operators, further laboratory analyses, and additional chemical storage, which add to a higher operating costs, therefore, chlorine dioxide is not considered a feasible alternative for Port Angeles.

6.4.5 Ozone

Ozone is the most powerful disinfectant of those used in water treatment. However, ozone is highly unstable in water and does not provide a long term residual. A secondary disinfectant (chlorine) is often required for distribution protection. Ozone is a very strong oxidizing agent as well as a broad range biocide. It is very unstable and must be generated on-site. One method is to pass dry air or oxygen through a high-voltage electrical discharge. It is the most expensive of the chemical disinfectants. Ozone is excellent for the following:

- Inactivating all pathogenic organisms – bacteria, viruses, as well as the protozoa, Giardia and Cryptosporidium.
- Eliminating bad taste, odor, and color of water by oxidizing the offending organic and inorganic constituents.
- Converting iron and manganese to insoluble hydroxide sludge for easy removal.
- Reducing the formation of trihalomethanes.

A major drawback of using ozone is it converts bromides in the water to undesirable bromates. High cost and operational complexity of its production are also significant limitations to its use.

A comparison of ozone to other disinfectants is presented in Table 6.15.

6.4.6 Ultraviolet Light (UV)

UV radiation is a good biocide, but like ozone provides no residual for distribution protection. The drinking water industry's migration toward UV has been fueled by the finding that UV light can inactivate *Cryptosporidium parvum* at cost-effective dose. There are three type of UV systems currently used in drinking water. They are low-pressure (LP) lamps, LP high-output (LPHO) and medium-pressure (MP) lamps. LP and LPHO lamps emit irradiation primarily at 254 nm, while MP lamps deliver continuos-wave UV light at higher intensities and across a range of wavelengths. The table below gives some technical data for the three UV systems.

Table 6.16
COMPARISON OF THREE UV LAMPS

Parameters	LP	LPHO	MP
Spectral distribution	Monochromatic	Monochromatic	Polychromatic
Temperature-F/C	95-113/35-45	122-176/50-80	752-1652/400-900
Power, W	45-100	100-400	1,000-25,000
Track Record	Extensive	Limited	Low
# of lamps required	High	Moderate	Low

UV lamps are surrounded by quartz sheaths, and the jacketed lamps are immersed in the flowing water. The flow is typically in a closed pipe and may be parallel or perpendicular to the lamp axes. It is important to ensure turbulent flow conditions within the UV disinfectant unit to allow all elements of the fluid to come sufficiently close to the lamp surfaces while minimizing the degree of transverse mixing (short-circuiting). Careful monitoring of microbial inactivation and lamp intensity is a requirement with UV disinfection.

The contact times for UV disinfection systems can be relatively short, generally under 1 minute. Therefore, the space required for UV disinfection units is relatively small. Because no residual is created, an additional final disinfection process would be required. The potential for UV reactions to produce organic by-products is minor because the intensities required for UV disinfection are less than those needed to cause photochemical effects. Operationally, employing an effective cleaning program to periodically remove biological and chemical fouling materials from the lamp jacket or Teflon tube surfaces is essential.

A comparison of UV to other disinfectants is presented in Table 6.15.

6.5 RECOMMENDED DISINFECTION OPTION

The recommended disinfection alternative for the City's municipal treatment plant is to use sodium hypochlorite as a first disinfectant and chloramines as a second disinfectant. Sodium hypochlorite is an effective disinfectant that has few safety and health risks during transportation and handling, and is the least expensive disinfection alternative to chlorine gas due to the facts that sodium hypochlorite is more safe to handle and does not cause serious public health and safety related risks associated with accidental leakage of the chlorine gas. Like chlorine gas, sodium hypochlorite will combine with naturally occurring organic matter in the water to form DBPs. The amount and nature of organic material in the water to be treated during dam removal and how those organics react with chlorinated compounds is unknown. To protect against the possible presence of organics and subsequent potential for DPB formation, a chloramine process will also be constructed within the treatment facility. Chloramines are typically not used as a primary disinfectant. Chloramines are used to provide a chlorine residual in the finished water, but do not form organic DBPs to same extent as hypochlorites. By using chloramines in conjunction with sodium hypochlorite, the City will be able to meet all of the disinfection requirements while providing the operational flexibility to minimize DBP formation if substantial organic material is present in the incoming raw water.

The estimated cost for disinfection given in this report was based on the purchase of liquid sodium hypochlorite. The costs for using a chlorine gas system, having an on-site sodium hypochlorite generator and UV disinfection with an on-site sodium hypochlorite are provided in Table 6.17.

Table 6.17
DISINFECTION OPTION COSTS

Disinfection Options	Capital Cost	O&M Cost	Total 20-Year Present Worth[1]
Chlorine Gas System	$316,000	$14,000	$477,000
Sodium Hypochlorite (liquid @ 12.5%)	$221,000	$18,000	$427,000
Sodium Hypochlorite with On-Site Generator	$238,000	$14,000	$399,000
UV Plus Sodium Hypochlorite with On-Site Generator	$627,000	$12,000	$765,000

Notes:
[1]For 20 years at 6% interest rate. Present worth is based on annual compounding discount rate of 6% for a 20-year period.
[2]Costs are based on the first quarter of year 2001 prices.
[3]Costs do not include purchase of land, easements, and similar.

6.6 TREATMENT RESIDUAL DISPOSAL OPTIONS

6.6.1 Estimated Residual Quantities.

As discussed earlier, water for the municipal treatment plant will come from the existing Ranney collector. The current average turbidities of water within the Ranney collector range from 0.04 to 1.0 NTU. Other water quality data includes:

- pH 6.5 – 8.5
- TOC 20 mg/L
- TSS parameters 7 mg/L

TSS concentrations within the Ranney collector have not been recorded regularly. The only TSS measurement conducted in 1973 measured 0 mg/L TSS. Based on a long-term TSS value of 69 mg/L presented in Table 2.4 and a 90% removal rate from the Ranney collector, TSS in water from the Ranney is assumed to average 7 mg/L. For a conservative estimate of residuals production from a conventional treatment plant the annual average water quality (dosage) was assumed. In addition, it was assumed that 10 mg/L of coagulation chemicals would be required to settle the suspended solids. Residuals from a conventional treatment process are created both in the sedimentation basin and during filter backwash cycles. The estimated residuals production from a conventional treatment process (assumes 0.44 lb. dry sludge/lb. alum [*AWWA Water Quality and Treatment*, 5th Edition, 1999, page 16.3]) would be approximately 150 lb./day inorganic aluminum solids plus the TSS in the water (dry weight) at the average flow rate of 4 mgd, and 390 lb./dry (dry weight) at the peak flow rate of 10.6 mgd. This residual would be a combination of solids from the river and coagulation chemicals.

Unlike all the other conventional filtration processes, the residuals from membranes would consist of much more quantity of water. A typical percent recovery for membranes is approximately 85-90%, which means that 10-15% of the untreated water that passes through the membrane is rejected as waste. For example, assuming 200 mg/L of TSS in the untreated water and 90% membrane recovery, the estimated residuals from membranes is 1.1 MGD with 2,000 mg/L (0.2%) of TSS at the peak flow of 10.6 mgdl. The rejected waste could be further treated with additional membranes to achieve an overall maximum recovery of up to 98%, but at an additional capital cost. The rejected membrane waste would contain inorganic solids from the river but does not contain any coagulation chemical residual, since no chemicals are typically used in the treatment process.

Additional residuals from a membrane process are also created during membrane backwash cycles, where solutions of acid and or base are used to clean the membrane units. It is difficult to determine the amount of backwash water that would be created without pilot testing of the anticipated water quality. The residual water created during these backwash events may be acidic or basic. Many times this residual water is treated to a neutral pH before being handled in the same manner as daily reject water.

6.6.2 Settling Pond/Landfill Disposal

A common practice for the handling of residuals from municipal treatment plants utilizes multiple holding basins that allow time for the solids in suspension to settle on the bottom. The water in these basins is decanted off the top and run back through the treatment plant. The solids at the bottom of the basins would need to be removed periodically. Solids are removed by first taking the active basin off-line and diverting residuals handling to one of the other basins. Next the water is decanted off the top of the basin exposing the residuals to be removed.

The solids at the bottom of the settling ponds generally will contain about 5% solids and presents some difficulties for transport and disposal. One option would be to try and dispose of the residuals while wet. The residuals would be transported as liquid to a landfill facility that can accept liquid wastes under PL-91-512 (Solid Waste Disposal Act). At the average treatment plant flow rate approximately 650 cubic yards of residuals at 5% solids would be collected each year. This represents an expensive option in regards to transportation and disposal costs, since there are no landfills that accept liquid waste in the immediate vicinity of Port Angeles.

A preferable option would be to dewater the residuals before disposal. Residuals can be thickened naturally by allowing in-place air drying or removal and placement on drying beds, depending on the climatic conditions. In Port Angeles the average precipitation of 26 in/yr exceeds the average lake evaporation of 22 in/yr. Typically a six month period of excess evaporation is needed to achieve adequate drying, therefore natural drying is probably not well suited for this climatic area. Another option would be to remove the residuals and mechanically dewater with rented filter press or belt press equipment. For ordinary landfill disposal the sludge would have to be chemically stabilized with use of polymers and mechanically dewatered to reach a 25% solids concentration to qualify as a "solid" for disposal purposes.

Based on the anticipated residual quantities presented above, approximately 130 cubic yards of dewatered residuals would be created each year. The frequency that this waste would need disposal would depend on the size of the settling basin and the on-site storage capacity. It is estimated that a basin 200 feet square with 3 to 1 side slopes and 5 feet deep would need to be cleaned once every 9 years.

According to the City of Port Angeles, the municipal solid waste landfill located within the city limits, is scheduled for closure by 2007. The City plans to truck all municipal waste to an out-of-town landfill after the closure that would represent an additional transportation cost to this option.

Using a settling basin and disposing of solids in a landfill is more difficult for municipal residuals created through membrane treatment. The solids in a membrane reject water are not likely to settle within the basin because they consist of mostly stable inorganic colloids. A coagulant could be added to the basins to destabilize the colloids and promote settling, but the operation and optimization of settling within the retention ponds would be operator intensive and add additional chemical costs to the treatment process.

6.6.3 Settling Pond/Reuse with Composted Wastewater Biosolids

Similar to the previous option the water treatment residuals would be placed into multiple holding basins and ultimately removed and dried to be combined with the composted wastewater biosolids from the City's wastewater treatment plant. The composting operation is located at the City landfill site. The combined water and wastewater residuals may be reused for agricultural purposes, soil amendment, fill, cover, and similar uses.

6.6.4 Ocean Discharge

Another residuals option that would not require dewatering, transportation and disposal would be to pump treatment residuals directly to the ocean. This option would require higher capital costs for the pipeline, ocean outfall, and pumping station than settling and landfilling. In addition, the continued pumping cost and environmental monitoring cost of residuals disposed in the ocean would be greater than the periodic cost of dewatering, transporting and disposal.

This option would be suitable for municipal water treatment residuals generated from either conventional treatment or membranes. In addition, this option would be highly suitable for disposal of residuals created during the treatment of the industrial and fisheries demand, because of the substantial volumes generated.

Discharge to the ocean would require a NPDES permit from the state under WAC 173-220. The permit would be acquired from the State of Washington and would require a public comment period and addition review by other government agencies such as, the Army Corps of Engineers, the United States Fish and Wildlife Service, the National Marine Fisheries Service, other state agencies and the Lower Elwha Klallam Tribe.

The Daishowa mill currently has an NPDES permit to dispose of treatment residuals with a shoreline outfall located directly at the edge of the treatment plant boundary. There is no pumping cost, or pipeline maintenance associated with their disposal.

6.6.5 Sanitary Sewer Disposal

An additional disposal option for municipal residuals would be discharge to the sanitary sewer system and treatment by the City wastewater plant. Municipal residuals from conventional or membrane treatment could be directly discharged to the sewer or be put through a gravity thickening process to decrease volumes discharged. Direct discharge to the sewer would produce about 88,000 gpd of 1% TSS residuals at an average treatment capacity (4.4 mgd) or 212,000 gpd of 1% TSS residuals under peak capacity (10.6 mgd). Additional hydraulic capacity in the sewer and wastewater treatment plant would be required to accommodate the additional flow. The residuals would appear as inorganic solids within the biosolids of the primary clarifier.

6.6.6 Discharge to Surface Water

The disposal of residuals from the conventional treatment of the municipal water supply to a nearby stream or river would not be a viable option because of the permitting challenges associated with discharging a chemical residual into a receiving water. Even disposal back into the Elwha River during dam removal is not viable because municipal residuals disposal will be a long-time operation and maintenance requirement required after the water quality in the Elwha River has been restored requiring the construction of residuals handling facilities.

Municipal residuals created from membrane treatment may potentially be permitted for disposal in the river because of the absence of chemical residue. The pH of membrane backwash water may need to be adjusted prior to disposal.

6.6.7 On-Site Residuals Dewatering

Another disposal option for municipal residuals would be discharge to the concrete equalization tank with the hydraulic detention of one day, then pump the sludge to the concrete gravity thickener with the hydraulic detention time of 2 days. After settling for 2 days, the sludge is sent to a belt press. The solids content form the equalization tank is between 0.5 and 1%, and the solids content after the gravity thickener is about 4%. The solids content after the belt press is around 25-30%, which meets the requirement of landfill disposal or if combined with composed wastewater biosolids for agricultural, fill, cover, or similar uses.

6.7 RECOMMENDED TREATMENT RESIDUAL DISPOSAL

The recommended alternative for disposal of treatment residuals will be to use settling ponds. Two settling basins will be required. Solids from the treatment plant clarifier and filter backwash water will be pumped to a lined detention pond that will allow the solids to settle. Decant water from the top of the ponds will be pumped back through the treatment plant. The settled solids will be periodically removed by changing operations to the other basin, draining off the remaining decant water, chemically stabilizing the solids, and mechanically dewatering the solids with rented equipment prior to disposal or reuse. As previously described, it is estimated that the ponds would require solids removal once every 5-10 years depending on the solids generated. Until 2007 the solid may be disposed of in the City's landfill. The City is currently exploring how to dispose the solid wastes after 2007, and hence the associated costs for the treatment residual disposal will change after the year 2007. As an estimate of the cost change, residual disposal cost will be increased by 30% after 2007. As a result of the landfill closure the combination of the water treatment residuals with biosolids compost is the most favorable option.

Though the settling ponds are recommended, the cost for the use of gravity thickening with a belt press was compared with the recommended option of residual disposal (see Appendix G), which is presented in Table 6.18.

Table 6.18
COST COMPARISON BETWEEN RECOMMENDED AND ON-SITE RESIDUAL DEWATERING OPTIONS

Treatment Residual Disposal	Subtotal	Construction (40%)	Engineering (20%)	Total Cost
Recommended Option	$1,148,000	$459,000	$321,000	$1,928,000
On-Site Residual Dewatering Option	$1,634,000	$654,000	$457,000	$2,745,000

Note:
[1] The gravity thickener with belt press option includes treatment residual equalization tank, gravity thickener, belt press, dewatered sludge holding tank, polymers, and associated pumping and piping systems.

As described in the next section, many of the proposed locations for the water treatment plant are near the Fairchild International Airport. The Federal Aviation Administration (FAA) has restrictions on open bodies of water near airports in order to reduce the potential for attracting birds. The regulations state that open bodies of water are prohibited within a 10,000-foot radius of the airport boundary. These regulations are open for interpretation by individual airports. For the Fairchild International Airport, the ocean is well within this 10,000 foot restricted area. Based on the location of the recommended treatment plant site, and discussions with airport staff, the treatment residuals settling ponds may require measures to deter birds from interfering with airport operations. Use of netting over the settling ponds similar to the netting used at the WDFW rearing channel is recommended.

6.8 POSSIBLE TREATMENT PLANT LOCATIONS

6.8.1 General Location Criteria

Criteria used in siting a treatment plant site include available land. The proposed treatment plant will require approximately 10 acres. The plant will typically have single story buildings and possibly some high bay buildings. Truck access is required for chemical delivery and general operation and maintenance. The site will have net protected (similar to WDWF rearing channel net coverage) water ponds for sludge handling as it is assumed that the open water will attract birds. Possible plant locations reviewed to date include properties owned by the City of Port Angeles, the Port of Port Angeles, Rayonier, and private individuals. Possible treatment plant locations are shown on Figure 6.7.

Site evaluation criteria used for siting a treatment plant include hydraulics, site factors, plant pipelines, and location of other utilities. These criteria are identified as follows.

6.8.1.1 Hydraulics

The existing system uses the pumps in the Ranney well to provide high pressure discharge pumps. Treatment plant locations will have to break pressure, treat the water and re-pressurize to get the treated water from the plant back into the system. Plant location should be sited to minimize additional water line construction to get water into different pressure zones.

6.8.1.2 Site Factors

Site location will need to consider the following items:

- Topography
- Initial plant construction with available room for expansion
- Operation and maintenance requirements
- Geology
- Environmental considerations, wetlands
- Compatibility of the proposed buildings with the adjacent land uses.
- Security
- Access
- Proximity of the plant with respect to the availability of existing water distribution facilities, electrical power, gas and communication utilities

- Zoning

6.8.1.3 Plant Pipelines

Plant siting needs to be close to the location of the existing pipeline from the Ranney well to distribution. The further the plant is away from the existing pipeline, additional expense will be needed to pipe water from the existing pipeline to the plant and back to the distribution system. It is also advantageous to have the sludge storage and evaporation ponds close to the plant to reduce the piping between sedimentation and filtration to the ponds.

6.8.1.4 Other Utilities

Treatment plants will require other utilities for operation including sanitary sewer, storm sewer, electric, gas, communication and telemetry. Having a treatment plant in a remote location will increase the overall capital cost of the facility because of the utility extensions required to provide service.

6.8.2 Plant Locations - Properties Owned by the City of Port Angeles

Properties currently owned by the City of Port Angeles identified for possible treatment plant locations include the following:

- City owned property along the Elwha River at the existing Ranney well and WDFW rearing channel
- South end of City landfill

The site of the current Ranney collector and WDFW fish rearing facility is not of sufficient size to support the approximate 10 acres required for construction of a municipal treatment plant. The site is bordered by the Elwha River on one side and a steep hill on the other. The WDFW fish rearing channel runs the entire length of the property. There is insufficient room for a filter facility or treatment residuals settling pond.

The City also owns a 40-acre parcel of land south of the WDFW facility. The majority of this parcel is on a steep hill and heavily wooded. The industrial intake and tunnel are located on this parcel. The amount of flat land available to construct a plant is very limited on this parcel and therefore not considered a feasible site.

Further south of the industrial intake, the City owns a small parcel on the west side of the river. This parcel is too small for a municipal treatment plant and would require intake and distribution piping to be constructed under the river.

The City landfill site is a possible location for the water treatment plant as shown on Figure 6.7. The current landfill is scheduled to be closed by 2007. The southern portion of the landfill site is located near the municipal distribution pipeline and consists of natural undisturbed soil. It is currently being used for material storage and as a transfer station. The wastewater treatment biosolids composting facility is located on the site and hence offers a potential water treatment residuals management option. Issues with this site include topography, environmental conditions and existing building structures. In addition to the composting operation being located at the landfill site, another advantage of this site is that the City is the current owner of the site and is planning redevelopment of the site to accommodate other City facilities compatible with water treatment facilities.

6.8.3 Plant Locations - Properties Owned by the Port of Port Angeles

Figure 6.7 shows the location of four possible treatment plant sites currently owned by the Port of Port Angeles. The sites are all in close proximity to the Fairchild International Airport. The Port of Port Angeles Site 1 is located on the east side of Lower Elwha Road, just south of the City's water line which comes directly from the Ranney collector. The site is relatively flat and clear of heavy vegetation. A drainage runs along the east side of this site and shows some evidence of erosion of the site and possible flooding. The site is in close proximity to the municipal distribution pipeline, has good vehicle access, and has utility access. Based on communication with the Port of Port Angeles and the Fairchild Airport Manager, this site has restrictions placed on it by the Federal Aviation Administration (FAA). The site was purchased with FAA funding and therefore cannot be sold. The Port of Port Angeles has indicated that it may be possible to develop a long-term lease on the property.

The Port of Port Angeles Site 2 is also on Lower Elwha Road, located on the west side and north of the existing water line coming from the Ranney collector. This site also has good access to existing utilities and does not have the same FAA site restrictions that the Site 1 has. The Site 2 is also in close proximity to the existing water line.

The Port of Port Angeles Site 3 is located on the west side of the north-south runway. This site has the same FAA restrictions as Site 1 and could not be purchased outright for the construction of a water treatment plant.

The Port of Port Angeles Site 4 is immediately adjacent to the Fairchild International Airport east-west runway. This site is being considered by the Port as a prime site for future development as an airport industrial park. The site is also not located near the existing water system transmission main and would thus require a costly pipeline extension to deliver untreated water to the site and a parallel treated water return pipeline. As a result of these two factors, this site will not be considered further.

Discussions with the Port of Port Angeles have suggested looking at another site which contains about 140 acres. This site is shown as Port of Port Angeles Site 5 on Figure 6.7. The site does not have any of the FAA restrictions on sales of the property. The disadvantage with the site is that it is not adjacent to the existing water line. Inlet and treated water pipeline to and from the plant will be required to connect to the existing pipeline. The length of pipeline required would be approximately 10,000 feet. Much of this site is heavily wooded with apparently poor surface water drainage. The eastern portion of the identified property is flat, unvegetated and does not appear to have drainage concerns. A visual inspection of this parcel revealed it is currently being used for agriculture. Just north of this property is a parcel used for flying remote controlled airplanes that would be another strong potential for a municipal treatment plant site.

All of the sites identified above are within a 10,000 foot radius of the airport boundaries, and therefore have restrictions prohibiting the construction of an uncovered settling pond for treatment residual disposal.

6.8.4 Plant Locations - Property Owned by Rayonier

Another proposed location for a municipal treatment plant is the site of the former Rayonier mill, which is located near the City of Port Angeles wastewater treatment plant. Hydraulically, the existing water distribution system for the City is set up to receive water from the west side of the City. The existing infrastructure has the larger diameter pipes at the west end. Locating the water treatment plant at the Rayonier site would involve upgrading the existing water distribution system to incorporate larger diameter pipes from the Rayonier site. The existing, but currently non-operational, Jones and Water Street Pump Station would have to be upgraded to transport all of the treated water into the City of Port Angeles distribution system. In addition, the 9th & Jones Street Pump Station would have to be upgraded.

The Rayonier site is currently under investigation by the EPA, Washington State Department of Ecology and the Lower Elwha Klallam Tribe for environmental contamination caused by mill activities. The remedial investigation work plan is scheduled to be complete by January 2002.

In addition, the Rayonier site is also currently under investigation for potential commercial development by both the City and the Tribe. The Lower Elwha Klallam Tribe has also identified portions of the site that have cultural and historical significance. For all these reasons, the former Rayonier site is not considered a viable option for a municipal water treatment plant site.

6.8.5 Plant Locations – Property Owned by Private Individual

There is the possibility of purchasing private property for the treatment plant site. The first consideration would be to determine if the property has the correct zoning for use as a treatment plant. The disadvantage includes the costs for acquiring the property and need to remove any residential structures on the site.

Figure 6.7 shows the locations of four possible treatment plant sites currently owned by private parties. The property location, area, and ownership are listed below:

Table 6.19
POTENTIAL PRIVATE PROPERTY FOR WATER PLANT LOCATIONS

Property ID	Location	Area, Acre	Owner
Private Location Site 1	West of Port of Port Angeles Site 2	16.52	Jaretta H. Pollow J.H. Dobrowsky
Private Location Site 2	Gravel Site West of Port of Port Angeles Site 3	4.57	Reggie L. Nason
Private Location Site 3	North of Gravel Site	63.29	Unknown
Private Location Site 4	Northwest of Gravel Site	31.15	Reggie L. Nason
Private Location Site 7	East of City landfill and south of private site 4	1.56	Unknown

All of the privately owned sites would require removal of existing trees, stumps and/or surface vegetation. Extensive removal of trees from Private Sites 3, 4, and 5 would be required. Site 2 is a quarry and would require extensive regrading or the use of imported material to restore the site. Private site 5 is not large enough for the entire treatment facilities but may be desirable if a portion of Private Site 4 were obtained for the plant location. All of the private sites would require the land to be purchased.

6.9 RECOMMENDED TREATMENT PLANT LOCATIONS

Based on the findings of a plant site review meeting on June 27, 2001 with ONP, Reclamation, the City, and the Tribe the top five sites were prioritized. The sites were selected and are listed from being most desirable to less desirable as follows.

1. City owned landfill site
2. Private Site 4 and 5
3. Port of Port Angeles Site 3
4. Port of Port Angeles Site 2
5. Port of Port Angeles Site 1
6. Port of Port Angeles Site 5

There are three recommended treatment plant locations at this time based on a cursory review of all the identified sites. The City owned landfill site, Private Site 4 and 5, and the Port of Port Angeles Site 3 are potential candidates for the development of a treatment plant. The Port of Port Angeles Site 3 is relatively flat recently logged and undeveloped and could be available for construction immediately. Private Sites 4 and 5 are immediately adjacent to the City owned landfill site and could be used as an extension for the development at the landfill site. Only 10 acres of the 32 plus acres of the site are proposed for water treatment usage. Use of the landfill site will require coordination with other proposed uses of the site that are currently being planned and include the composting facility, material storage, and transfer station. The advantage with this site is that the City is the current owner of the site, and the wastewater biosolids composting facility is onsite and would provide easy access for handling of water treatment residuals. The two private sites or the Port of Port Angeles Site 3 would have to be purchased or leased.

Table 6.20 is summary of the site evaluation criteria for each of the four sites considered.

For the purposes of this report, it is recommended that the City landfill site will be the treatment plant location. A siting study to further examine the environmental issues, geological and geotechnical issues, and property acquirement and cost issues will be required. Additionally an agreement with the City will be required for use of the land before the recommendation can be confirmed.

The site layout for an Actiflo plant, filter units, and treatment residuals disposal on the landfill site is shown on Figure 6.8. The entire treatment plant will be enclosed and include office space, laboratory space, and chemical storage.

At the time of this report, there are four residences currently upgradient of the proposed treatment plant between the existing Ranney collector and the proposed treatment plant sites. Based on discussions with the City of Port Angeles' utility engineer, these residences may be transferred over to the Dry Creek Water Association (if accepted by DCWA) as a result of the recent GWI classification of the current source and lack of sufficient chlorine contact time in the present system.

Table 6.20

MUNICIPAL WATER TREATMENT PLANT SITE EVALUATION CRITERIA

7.1 GENERAL

The water supply and treatment for the City of Port Angeles' industrial water supply and for the Tribe's hatchery during the dam removal and delta erosion period are presented and evaluated in this section. In combination with the industrial and fisheries water supply an alternative municipal water supply is also considered. Methods of supplying water from the Elwha River downstream from the dams during this period of time include the following:

- Surface water intake
- Infiltration gallery
- Vertical wells
- Radial collector wells

Alternative water supplies from other sources are also evaluated and include supplying water from the Upper Elwha River above Lake Mills, Morse Creek, Little River, and Indian Creek.

In order for surface water from the Elwha down stream of the dams to be viable for the industrial and hatchery water supply, treatment will be necessary to provide the appropriate water quality for use. The methods of treatment evaluated

- Chemical coagulation and sedimentation
- Membrane filters
- Disk filters

Alternatives combining the more favorable water intake and treatment options are evaluated and capital and operating costs are presented with consideration of the advantages and disadvantages of each industrial and fisheries alternative. Where applicable, items requiring additional information are identified.

The quantities of water required for the City and Tribe are presented in Table 7.1 and are the basis for the evaluations. The City's municipal demand is included to provide an alternate or supplemental water source to the City's existing radial collector well.

Table 7.1

WATER INTAKE AND TREATMENT CAPACITIES

Entity	Intake Capacity[1]	Treatment Capacity[2]
Port Angeles Municipal	32.2 mgd (50 cfs)	10.6 mgd (16.4 cfs)[3]
Port Angeles Industrial	96.8 mgd (150 cfs)	--
Daishowa	--	14 mgd (21.7 cfs)
WDFW	--	14.2 mgd (22 cfs)
Tribal Fish Hatchery	18.6 mgd (28.8 cfs)	12.4 mgd[1] (19.2 cfs)
TOTAL	148 mgd (228.8 cfs)	51.2 mgd (79.3 cfs)

1 Based on current water rights except no water right for Tribe has been established.

2 Based on current maximum monthly demand.

3 Accounting for water production lost due to backwash and residuals disposal the actual process capacity is approximately 11 mgd (17 cfs).

The City's remaining industrial water supply, that is 150 cfs less the current industrial demand (Daishowa-21.7 cfs and WDWF-22 cfs) in Table 7.1, is 68.6 mgd (106.3 cfs).

7.2 INDUSTRIAL AND FISHERIES INTAKE OPTIONS

There are a variety of alternatives for obtaining water from the Elwha River or the alluvium directly under and adjacent to the river to supply the industrial and fisheries water needs and supplement the municipal supply. These alternatives are:

- Surface Water Intake
- Infiltration Gallery
- Vertical Wells
- Radial Collector Wells

The following sections provide a brief description of each water intake option focusing on constructability, performance, operation, and maintenance issues.

7.2.1 Surface Water Intake

The City currently obtains water for use by Daishowa and the WDFW fish rearing facility through a surface water intake. At approximately river mile 3.48, a rock diversion dam backs up water into the industrial intake tunnel. The capacity of the intake tunnel is estimated at 193.5 mgd (300 cfs). The invert elevation of the tunnel inlet is 59.0 feet, and the elevation of the riverbed in this area is approximately 53 feet. Water from the inlet tunnel flows into the Industrial Diversion Channel then to the industrial pipeline to Daishowa, to the WDFW Rearing Channel, and to a bypass channel back to the river. Fish screens are provided at the inlet to the industrial pipeline that takes water from the industrial diversion channel but there are no provisions for the screening of fish at the current river intake.

Based on the sediment model developed by Reclamation, using surface water during extended periods of good water quality during dam removal may be possible for the industrial and fisheries supply with no treatment. In addition, the use of surface water for the long-term water supply after the water quality impacts of dam removal have passed is preferred from an economic standpoint. This is preferred because it is a gravity driven delivery system rather than pumped as required for groundwater sources, although the groundwater sources provide good quality water that may ultimately require less treatment.

During periods of poor water quality resulting from dam removal and delta erosion, a surface water intake without additional treatment facilities is not an acceptable alternative because of the large amount of suspended sediment that would be in the water, making it unsuitable for fishery or industrial use. During periods of good quality water, however, a surface water intake will be suitable for use by all of the current industrial and fisheries water users.

Use of the current surface water intake is possible but the invert of the intake is higher than the current riverbed elevation and resulting water surface. The existing rock diversion structure just downstream of the intake raises the water level to allow water to enter the intake. The existing rock diversion requires frequent maintenance and is not accepted as suitable for the passage of migrating fish. A new surface water intake facility that has provisions for fish passage and passage of large quantities of sediment during dam removal is required.

New surface water intake facilities would need to be constructed to allow fish passage and allow heavy loads of sediment and riverbed materials to pass downstream. The intake facilities would also need to allow for continued recreational use of the river by boats and kayaks.

A conceptual design of a surface water intake facility is shown in Figures 7.1 and 7.2. The facility features two radial sand-out gates, one on each side of the river. It is anticipated the gate on the side adjacent to the intake would be operated continuously to pass sediment downstream and keep the sediment from depositing near the intake structure. The gate can also be used to pass debris downstream. The opposing sand-out gate can be used to de-sand the upstream pool for operation and maintenance. Radial gates can also be used to assist in reducing upstream flood height relating to the intake facilities.

The fish ladder or passage represents approximately 110 feet of the intake facility. The fish ladder is designed with approximately a 10:1 downstream slope. The ladder has multiple 1-foot drops. Each drop is controlled with a sheet pile wall with concrete cap. The top of the cutoff walls slope at a 3:1 slope from the walls to the control elevation. The walls were designed this way to create eddies adjacent to the walls. Eddies will allow a resting place for migrating fish as they move up the ladder. The design of the eddies will allow for the release of sediments that may accumulate during high turbidity periods.

The reported average daily flow for the Elwha River is 1,500 cfs. Given this flow, the depth of flow through the fish ladder is approximately 2.5 feet on average. There are low flows of 300 to 500 cfs during the summer months. The depth of flow through the fish ladder for these low flows could reach 1-foot. If necessary the ladder would be designed with a smaller pilot channel for these low flow periods to provide a greater depth and facilitate fish migration.

Prior to the surface water being used for fishery or industrial purposes it is anticipated that fish screening will have to be provided. This may be provided at the new inlet or prior to treatment.

The *Sediment Analysis and Modeling of the River Erosion Alternative Report* (BOR, October 1996) predicts the river bed will aggrade 6 feet at the current industrial intake with no diversion structure in place. If surface water is utilized only to supply water in the long-term after the water quality impacts of dam removal have passed, a surface water diversion structure may not be required; however, the length of time for the riverbed to aggrade 6 feet has not been determined.

Figure 7.1 (11x17) Intake facility

Figure 7.2 (11x17) Intake facility details

7.2.2 Infiltration Gallery

7.2.2.1 In-Stream Infiltration Gallery

One type of infiltration gallery consists of a series of horizontal drains located in the riverbed and placed within a constructed filter pack approximately 5 feet deep. An infiltration gallery's yield is directly related to surface water recharge. In order to ensure adequate yield from an infiltration gallery, it must be located on a relatively stable stretch of river that is not likely to migrate away from the intake screens of the gallery. The *Water Quality and Mitigation Measures Report* (BOR, March 1997) recommended an infiltration gallery be constructed within the river channel from 1,000 to 2,400 feet upstream from the existing industrial surface water diversion at river mile 3.48. This gallery was sized to meet the needs of Daishowa and Rayonier with a safety factor of 2. The proposed location as depicted on Figures 7.3 and 7.4 represents a relatively stable stretch where river migration would be minimized. Due to the anticipated need to treat the water from the infiltration gallery, a layout of conventional chemical treatment and clarification facilities is shown on Figure 7.4.

An in-stream infiltration gallery system would consist of intake screens, filter pack, filter pack armor, conveyance system, and connection to the existing industrial intake tunnel through a vertical riser structure. The conveyance system includes an intake piping system feeding a pump station by gravity, pump station, discharge piping system, backwash system and vertical riser structure. A cofferdam would be required for all in-stream construction. The conveyance system would connect the intake screens with the pump station and then run under the river bed to connect with a vertical riser structure leading to the existing industrial intake. Piping would vary in size from 24 to 72-inch. The backwash system would provide a means of clearing intake screens and maintaining the filter pack performance by reversing the flow of water through a portion of the intake system. The pump station would require approximately four high volume, low lift conveyance pumps and one backup. An additional pump and piping system would be required for the backwash system. A combined air and water backwash system may also be considered.

The vertical riser structure was initially proposed in the *Water Quality and Mitigation Measures Report* (BOR, March 1997). The pump station would fill the vertical riser structure that would push water through the existing industrial diversion tunnel under pressure. The use of the vertical riser would eliminate the ability to utilize the tunnel as a surface water intake in the future when water quality should return to background quality. An additional option would be to pipe the flow from the pump station around the existing industrial intake tunnel directly to the head of the industrial diversion channel.

The performance of an in-stream infiltration gallery is highly dependent on the design of the filter pack and screens. A high yield filter pack will produce low quality water, while a filter pack producing high quality water will produce lower yield and potentially clog. The quality of water from an infiltration gallery is directly correlated to the water quality within the river and the type of filter pack used. For the purpose of this report, an assumption was made that 25% of the fine suspended solids within the Elwha River would pass through an infiltration gallery and require further treatment. This is a relatively conservative assumption based on professional

Figure 7.3 (11x17)

Alt. 1 Site Layout

Figure 7.4 (11x17)
Alt. 1 Layout.
This is the layout of the facility.

experience and the 1998 study conducted by Gathard Engineering and Consulting on the performance of the Tribal infiltration gallery. This study suggested approximately 85-92% of the river water turbidity and up to 90% of the TSS was removed through the gallery (Gathard Engineering and Consulting, 1998).

Another consideration for the design of an in-stream infiltration gallery is the potential affect of aggradation of lake delta sediment over the top of the infiltration gallery. Several feet of aggradation are predicted and may result in the reduction of yield from the gallery.

A limited bench scale test conducted by URS indicated that an infiltration gallery could be very susceptible to clogging with river sediment. Sediment from Lake Aldwell was used for the test. The 18-inch deep, 0.5 mm sand media used for the test clogged rapidly due to surface blinding. Water passing through the media prior to blinding still had fine sediment visually evident in the filtrate. The bench scale unit required frequent backwashing. Both water and air were used for backwashing with air having the best result, but over time the yield of the test unit decreased even after each backwash. The importance of this bench scale test is that it demonstrated that an infiltration gallery in the riverbed would likely have the following characteristics:

- Frequent backwashing will be necessary.
- Loss of yield from the gallery over time is likely.
- Fine sediment found in the Elwha sediment will likely pass through the bed, thus requiring further treatment to meet user water quality requirements.

Additional major operational and maintenance issues associated with an infiltration gallery include cost and maintenance of the pump station and careful monitoring of gallery performance in order to optimize backwash cycles and gallery yield.

Rather than having an in-stream infiltration gallery with an engineered filter pack, it may be possible to construct a gallery along the river bank, which would take advantage of the filtering ability of the natural soils. This concept is described in the next option.

7.2.2.2 On-Land Infiltration Gallery

An option for long-term fisheries water supply for the Tribe would use an on-land infiltration gallery. This concept applies a single horizontal drain located parallel and adjacent to the river channel and is placed within a high permeability filter pack to ensure a hydraulic connection to the river, as well as the alluvial aquifer. The proposed location for such a gallery is north of the WDFW Rearing Channel. One benefit of this gallery is water supplied for the Tribal hatchery could be delivered by gravity without the need for pumping.

The on-land infiltration gallery would consist of several primary components including the following: an intake screen, filter pack, conveyance piping, and site improvements. The installation of the intake screen will require excavation to approximately 8-feet below the water table. The required depth of excavation, approximately 12 to 15-feet below ground surface, and the required length will result in a large quantity of excavated material and subsequent filter pack and backfill cover placement. The filter pack for an on-land gallery will be a relatively coarse-grained material, medium-sized gravel. The intake screen could be arranged with one or more connections to a central vault, which would be connected to a pipeline to the Tribal hatchery. Due to the required screen length, approximately 1,300-feet for a flow of 12.4 mgd (19.2 cfs), the intake will be constructed with several clean-outs to allow sediments to be removed from the screen. This intake would also require a pipeline capable of conveying the desired flow, this would be a 30-inch pipe for a flow of 12.4 mgd (19.2 cfs). This pipeline could be constructed using one of three methods:

- Sloped with open cut excavation, depths from 5-feet to 25-feet.
- Sloped with open cut excavation combined with micro-tunneling, for depths over 15-feet.
- Siphon with minimum depth open-cut excavation.

The pipeline will convey water from the vault near the intake to the cistern at the Tribal hatchery. This will require approximately 5,000-feet of pipeline with an estimated 20 to 30-feet of downward elevation change. Site improvements required for this intake will include a service road along the intake alignment to provide access to clean-outs and along the conveyance pipeline for maintenance. Pumping of the water is possible but not considered due to an increase in operating cost and the availability of gravity flow.

The design of an on-land infiltration gallery is strongly influenced by existing conditions, more so than an in-stream gallery due to the dependence on the native soil ability to deliver water to the gallery. The capacity for this type of intake is influenced by several variables:

- Distance from river
- Hydraulic conductivity of native soil and potential for having the conductivity reduced by the high quantities of fine sediment in the water during dam removal and delta erosion. Water and fine sediment will be drawn through the native soil to the on-land infiltration gallery and has the potential for the fine sediment to reduce the native soil permeability and thus reduce the water yield from the gallery.
- Depth of excavation
- Draw down at the intake screen
- Riverbed aggradation
- Migration of the river away from the on-land gallery

These variables provide an indication of some risks associated with an on-land infiltration gallery, these include variability of the native soils, potential for reduced hydraulic conductivity due to fine sediments in the water, and river migration. Variation in the native soils and the potential for clogging with fine sediments that will be in the river water during dam removal can effect the capacity of the on-land gallery. If some areas are less permeable than that estimated for sizing the facility, the facility may be unable to provide the required capacity. River migration can also have a dramatic effect on intake capacity and the stability of the intake. If the river migrates away from the intake screen on one side of the existing river channel, the capacity of the intake is reduced. If the river should migrate toward the intake, the screens could potentially be exposed and damaged, that would eliminate any filtering. The riverbed aggradation, which is expected following dam removal, will increase the migration potential of the river in the area of the proposed gallery.

The water quality will be very similar to the river's water quality and potentially better than from an in-stream infiltration gallery due to passing through native soils. It is anticipated that the water quality from the on-land infiltration gallery will vary more than the water from a vertical well or a radial collector well because the wells are deeper and thus have more capability to remove turbidity. This is especially true for water temperature and colloid and macrophage concentrations. The typical filter pack used for an on-land gallery will provide essentially no filtration of the water coming from the river, instead the native materials between the filter pack and the river will provide the filtration of the river water. The effectiveness of the filtration of the water due to the native soils may vary and will depend on the soils at the location of the gallery.

On-land infiltration galleries are not recommended and will not be considered further considering the high potential for river migration, varying native soil conditions, and potential reduction in capacity over time.

7.2.3 Vertical Wells

The quality of water from a surface water diversion or an infiltration gallery is directly related to the quality of water within the river and would require additional treatment to remove suspended solids prior to supplying the industrial and fisheries demand. Groundwater is an alternative source of water that could provide a higher quality supply of water and is less dependent on water quality within the river. Based the geology of the area and the initial hydrogeologic research presented in the *Water Quality and Mitigation Measures Report* (BOR, March 1997) and the *Lower Elwha River Groundwater Resource Evaluation* (URS, September 2001), the only source of groundwater in the region with the capacity to supply a sufficient quantity of water would be the alluvial aquifer that is recharged by the Elwha River.

The alluvial aquifer in this area consists of permeable layers and channels. These layers and channels run horizontal with the flow of water and produce the highest yield of water. A vertical well drilled through the alluvium would have a limited surface area exposed to these high yield layers. Even with multiple screened intervals, the amount of surface area of the well located within a permeable layer would be limited by the thickness of the permeable layer.

Historical and modeled performance of vertical wells in the alluvium of the Elwha River are available to allow an estimate of the number of wells required to provide water for fishery and industrial demands. Based on the performance of wells at the WDFW fish rearing facility and Tribe, a typical single vertical point well could produce between 0.4 mgd (0.6 cfs) and 1.5 mgd (2.3 cfs). In the *Lower Elwha River Groundwater Resource Evaluation*, production rates for vertical extraction wells are estimated to be approximately 1.3 mgd (2 cfs). This is within the range of the existing vertical wells at the hatchery and rearing channel. The depth of the wells is recommended to be approximately 85 to 95 feet depending on the local conditions at each well.

The current total maximum monthly water demand for Daishowa, the WDFW rearing channel, the Tribal hatchery, and allowance for the City municipal water supply is 51.2 mgd (79.3 cfs). Based on the findings of the groundwater resource evaluation, the well field would consist of 40 to 50 or more wells located on 300 to 400-foot centers. Wells may be located on the west side of the river in the Middle Basin above the existing City Ranney well and on both sides of the river in the Lower Basin between the existing City Ranney and the Tribal hatchery.

In addition to the wells, other facilities including pumps, manifold piping, access roads, and an electrical conveyance and control system would be required.

Potential concerns with vertical wells are the following:

- A large number of wells are required and will have to be monitored for performance.
- Pumping is required that will have operating cost including electric power and pump and well maintenance.
- At a 300 to 400-foot spacing, there is not enough river length in the middle and lower basins for the number of wells required.
- The well manifold piping, electric power and control, and access road will be required on both sides of the river and will be through various properties and require appropriate easements for construction and operation.
- There is potential for reduction of capacity due to the plugging of the interstitial spaces in the alluvium with fine sediment thus reducing the permeability of the aquifer due to the withdraw of water by the vertical wells. Due to this concern it is recommended that vertical wells along the river not be used during the high turbidity periods expected from dam removal and delta erosion. Another source of water supply will be required during the period when the vertical wells are not in use. This is further described in the radial collector well water supply section that follows.

As a result of there not being enough river length to place wells and the recommendation to not use the wells during periods of high turbidity in the river and thus require another supply of water, the use of vertical wells is not recommended and will not be considered further.

7.2.4 Radial Collector Wells

An alternative method to obtain groundwater would be to use radial collector wells. An example of a collector well is the City's current Ranney well. The name Ranney refers to a specific company that designs and installs these types of wells. A collector well consists of a vertical caisson installed through the depth of the aquifer. Screened lateral pipes extend from the caisson at various depths into the most permeable sections of the aquifer and transfer water into a vertical caisson where it can be pumped to the transmission main.

The City's current Ranney collector draws water from a depth of 60 feet below the river channel. The current Ranney collector produces approximately 10.7 mgd (16.5 cfs), down from an initial production of 15.7 mgd (24.3 cfs). The reduced yield has been attributed to river migration. The existing collector well has been shown to remove over 99% of the river water turbidity during high turbidity events although the percent removal efficiency is lower when the river water turbidity is low.

For conceptual level design and cost estimating, it is assumed that each new collector well installed along the Elwha would produce approximately 12 mgd (18.6 cfs). The actual capacity of each collector along with the exact location of each collector will need to be determined through hydrogeologic testing. Hydrogeologic investigation will also be required to determine the maximum yield of the alluvium along with the actual impacts of pumping on salt water intrusion and the effect on production from surrounding existing wells although the groundwater resource evaluation does not identify this to be likely.

Figure 7.5 and 7.6 are a site layout and plan of a potential radial collector well system. A total of six new collector wells and the existing City Ranney well are indicated on the figures with interconnecting piping. Four new wells with a potential capacity of 48 mgd (74.4 cfs) plus the existing Ranney are shown to serve the City industrial and municipal needs. Two wells are shown to provide the Tribal hatchery with 24 mgd (37.2 cfs) of water. Both of these quantities are greater than required for the current water requirements as presented in Table 7.1. In the City's case if Daishowa were to exercise its entire contract water requirement of 21 mgd (31 cfs) or another water using industry were to locate in the City's service area, there would be approximately 19.8 mgd (30.6 cfs) of water available for these purposes. In the Tribe's case one well at an anticipated 12 mgd (18.6 cfs) is not enough to supply the Tribe's anticipated water need, thus two wells are provided. An interconnecting pipeline between the Tribe and City wells is indicated to improve the reliability of both systems.

In accordance with the recommendations of the groundwater resources evaluation report it is recommended that radial collectors be located at least 1,500 feet apart to prevent hydraulic draw down from influencing adjacent collector wells. Based on the length of river available for placement of the collector wells approximately eight new collector wells may be provided for a total of 96 mgd (148 cfs) of water supply capability. This amount of water is adequate to supply the current City and Tribe water needs but limits the amount available for an increase in the industrial supply if a new major industry was to be added to the City system and assumes no reduction in capacity due to river migration.

There are three major concerns over the utilization of radial collector wells for supplying the industrial and fisheries demand.

- First is the concern that fine colloidal particles released with the lake sediments will pass through the river alluvium and enter the collector well, and therefore require further treatment of the water.
- The second concern is that fine particles could clog the interstitial spaces of the river bottom for at least up to the first several feet into the riverbed. This would impact permeability and reduce the ability of the surface flow of the river being able to recharge the aquifer and therefore reduce the yield of the collector wells.
- The third concern is that river migration will reduce the yield of the collectors or flood the area around collectors making access difficult.

The potential that fine particles may clog the interstitial spaces within the alluvium and reduce the permeability of the aquifer and therefore the yield of the radial collector leads to a recommendation that the radial collector wells not be used during high turbidity periods expected from dam removal and delta erosion.

Using limited bench scale testing conducted by URS, an attempt was made to simulate the effect that varying levels of sediment in the river water would have on the river bed in areas where water was being withdrawn from the groundwater. From the limited bench scale demonstration it can be safely said that as the solids concentration of the river water increases, the amount of water coming directly from the river in the immediate vicinity of the collector well will be reduced as the riverbed becomes clogged. As a result the water collected will increasingly have

Figure 7.5 Site Layout Radial Collector Wells

Figure 7.6 Radial Collector Wells and Interconnect to Municipal Supply to come from the alluvial groundwater as the riverbed in the immediate vicinity of the well may become blinded. If radial collectors are located along the entire length of the river, the blinding effect on the riverbed and resulting loss of aquifer recharge will limit the amount of water that can be withdrawn from the aquifer due to lack of recharge.

To reduce the potential blinding effect, shutting down the radial collectors at high solids periods in the river that will occur as the lower portions of the dams are removed and during heavy delta erosion periods, would improve the life and production rate for the radial collector wells. Subject to further testing, at this time shutoff of the radial collector wells at 10,000 mg/l TSS is assumed. Fine solids are likely to be in the water withdrawn from the radial collectors during periods of high river water turbidity as is currently experienced at the City's Ranney well. As a result of this recommended shut down of the wells, another source of water supply would be required during high turbidity periods

With 99% removal and 10,000 mg/l TSS in the river, the water collected by the radial collectors could approach 100 mg/l. This may be in the proximity of 130 NTU. This level of turbidity can be reduced to acceptable levels by the conventional water treatment technologies used by Daishowa or as is proposed for the new Port Angeles municipal treatment plant. The water directly from a radial collector, however, would not be suitable for the low turbidities in the 1 to 20 NTU range required for the hatchery and the rearing channel. As a result, supplemental treatment of approximately 26.6 mgd (41.2 cfs) of WDFW (14.2 mgd/22.0 cfs) and Tribal hatchery (12.4 mgd/19.2 cfs) water would be required. The treatment process would have to consider the effect of treatment chemicals on the fish.

As a result of:

- there not being enough river length to place radial collector wells for the entire (current and remaining) industrial and hatchery water supply,
- the need for treatment of the fishery water during high turbidity periods,
- the recommendation to not use the wells during periods of high turbidity in the river due to riverbed blinding, and
- the need for an alternate supply of water during high turbidity periods;

the use of radial collector wells is not recommended and will not be considered further.

7.3 TREATMENT OPTIONS

There are a number of ways to treat water to meet the industrial and fisheries demand. It is currently proposed that treatment would only be required for water obtained through a surface water intake or infiltration gallery. The treatment objectives for the fisheries users is to provide them with water not to exceed 20 NTU (estimated to be 15 mg/l TSS) on a continuous basis based on the recommendations of the Montgomery Watson Harza (MWH) work completed for the Tribe in October 2001. According to MWH, 80 NTU (estimated to be 50 mg/l TSS) can be provided for three to five day surges and 100 NTU (estimated to be 80 mg/l TSS) for surges less than 24 hours in length. Aluminum sulfate is a coagulating chemical that may be used in treatment. MWH has indicated that the National Ambient Water Quality Criteria (NAWQC) developed by the EPA recommends an acute exposure level of <0.75mg/L and a chronic exposure level of <0.087 mg/L for aluminum for the Tribe's hatchery operation. MWH also recommends pH to be in the range of 6.5 to 7.5 with changes no greater than 0.5 during any 24 hour period. Similar water quality for the WDFW Rearing Channel is assumed to be acceptable.

The water quality for Daishowa can have a higher turbidity due to the conventional treatment process available at the plant to further treat the water. The level of treatment is significantly less than the level of treatment required for the municipal demand as concern for TOC, TTHM, Giardia cysts, and human health viruses are not an issue.

The treatment processes considered for this industrial and fisheries demand include the following:

- Chemical coagulation and sedimentation
- Membrane filters
- Disk filters

The following sections briefly describe each of these processes and how they could potentially be used for this project. It is assumed that any treatment process for industrial and fisheries demand would be in operation for approximately five years. The *Sediment Analysis and Modeling of the River Erosion Alternative Report* (BOR, October 1996) showed that the river water quality should return to background levels within three to five years and use of surface water would be possible. The continued use of the treatment facilities after three to five years would be at the discretion of the City or Tribe.

7.3.1 Chemical Coagulation and Sedimentation

This process consists of adding coagulation chemicals to the water and allowing the suspended solids to coagulate and settle. This process is essentially the same as the conventional treatment alternative for municipal treatment except that filtration and disinfection would not be required. The use of chemical coagulation was the selected mitigation alternative in the *EIS* (ONP, 1996).

The treatment facility would be located at the site of the existing WDFW fish rearing channel. Traditional water treatment coagulation chemicals would be used with this alternative such as alum and polymer. The *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1996) conducted jar tests to determine coagulant doses. The report determined that a dose of 10 mg/L alum and 13 mg/L polymer could reduce 30,000 mg/L suspended solids down to a turbidity of 5 NTU.

Further bench scale water treatment testing was conducted at the URS Seattle lab to determine if solids could be removed from synthetic river water.

The bench scale test indicated that without the use of coagulating chemicals and a 30 minute settling time, the following clarified water turbidities were achieved.

TSS	Initial Turbidity (NTU)	30 Minute Turbidity (NTU)
3.5% (35,000 mg/L)	> 1,000	310
0.42% (4,200 mg/L)	> 1,000	120
0.045% (450 mg/L)	120	28

With the addition of coagulating chemicals to the preceding samples, clarification of the water was achieved to the range of 9 to 21 NTU using 7 to 130 mg/L dosages of alum depending on the initial water quality.

A major concern with this alternative is the production of treatment residuals. The residuals would consist of settled river solids and coagulant chemicals. The quantities of residuals produced and the associated disposal options are provided in Section 7.5.

The impact on the fish of coagulant residuals in the water delivered to the WDFW fish rearing channel and Tribal hatchery at an aluminum concentration of <0.087 and <0.75 mg/L for chronic and acute levels respectively, is the range for acceptable aluminum concentrations for fisheries use (MWH 2001). High doses (300 mg/L) of the polymer polyacrylamide (PAM) has been tested and shown to have adverse effects on fish while dosages at 10 mg/L have not had an effect on Mytilus galloprovincialis embryos (Bill Gardner, Memorandum, November 16, 2000 based on Northwestern Aquatic Sciences testing). Additional research by MWH (October 2001) on use of cationic and anionic PAM indicates that neutral and anionic charged polymers may be of concern for aquatic organisms however with the addition of sediments the toxicity is greatly reduced. The use of dry anionic PAMs have shown no toxicity to fish according to Sojka and Lentz (1997) as cited in the MWH work. The residue of polymer in the treated water is not anticipated to be even to the 10 mg/L range as polymer dosages in full scale treatment facilities are typically less than 10.

7.3.2 Membrane Filters

The use of membrane technology to physically separate suspended solids from the source water was also considered for the industrial and fisheries demand. The membrane treatment process can either push water across the membrane with pressure or draw water through the membrane with a vacuum. This process and associated equipment is the same as described under the municipal treatment process descriptions. A membrane treatment system could provide an excellent quality of water to all users and would not require chemical additions and associated concerns with treatment residual disposal.

Membrane manufacturers were contacted to discuss the feasibility of using membranes during high turbidity events. Based on a description of the potential water quality, the membrane manufacturers did not feel that the treatment of raw water using membranes would be feasible due to the potential for clogging and fouling during high turbidity events. As a result, some form of pretreatment to reduce solids would be required to use membranes. The equipment cost of membranes is substantially more expensive than a coagulation and sedimentation treatment facility sized to meet the industrial and fisheries demand. Based on the need for pretreatment combined with the substantial membrane capital costs, membranes are not considered a viable option for treating the industrial and fisheries water.

7.3.3 Disk Filters

Another treatment option similar to membranes is the use of rotating disk filters. Disk filters consist of a series of rotating disks with fabric filters that provide a physical barrier to remove suspended solids. The water passes through the filters by hydraulic head rather than induced pressure or vacuum. As the disk rotates, a scrapper removes caked solids from the upper portion of the disk, preparing the surface for filtration before it is resubmerged. The solids have no chemical added and could potentially be put back into the river.

Like traditional membranes, disk filter manufacturers were contacted to determine the feasibility of this process for the Elwha Project. The manufacturers responded by indicating that the water source had not been sufficiently described in the sediment transport model to provide confidence in applying their equipment. The manufacturers recommended pilot testing before providing even conceptual level estimates on cost or sizing. The biggest variable for this process is filter loading rate. Based on the sediment fractionation analysis, the potential clay fraction of the suspended sediments could reduce the surface loading rates to below 0.1 gpm per square foot. Based on this loading rate, the potential equipment costs alone could exceed $60,000,000. Based on this conservative design estimate, disk filters were not considered a viable option for treating the industrial and fisheries demand.

7.3.4 Storage Basins

Another treatment alternative is to provide settling basins with storage volume for industrial and fisheries users. The basins could be filled with surface water or water from an infiltration gallery during periods of low turbidity in the Elwha. When the river becomes excessively turbid, water could be supplied to users by the storage reservoir. Based on the sediment model and water demand, it was assumed that a 5-day storage volume for maximum monthly demand would be a conservative conceptual estimate for sizing a basin. The basins could also be used continuously, providing a 5-day detention time for settling solids before distribution to users.

The major concerns with settling basins are the amount of land they would require and the water quality within the basins. A reservoir with 5-day detention time for the maximum monthly demand of 51.2 mgd (79.3 cfs) is approximately 45-acres of land, assuming the basin could be constructed with a 20-foot depth (18 feet water depth). In addition, 5-days of detention time would increase the temperature of the water and potentially promote alga blooms. Water temperature is an important water quality parameter for the fisheries. Also, a portion of the suspended sediment in the Elwha River water during dam removal could be colloidal. These colloidal particles are similarly charged on a molecular level and very stable within the water column, which means, the particles will not settle out without a chemical coagulant or other water treatment process. Water from a settling basin would need further treatment before distribution to individual users. Any treatment would have to consider the effect of treatment chemicals on the fish.

Figures 7.7 and 7.8 present a proposed location for a single basin and locations for multiple storage basins to serve each major user.

Due to the need for additional treatment and potential temperature affects the storage reservoir alternative will not be considered further.

7.4 ALTERNATIVE TEMPORARY WATER SOURCES

During the early stages of the planning process, several potential temporary sources of water were identified for supply of water for fishery and industrial use during the dam removal period. It was proposed that these sources could be utilized during high turbidity events in the Elwha River in place of an extensive water treatment system. The potential temporary water sources include the following:

- Upper Elwha River above Lake Mills
- Morse Creek
- Little River and Indian Creek

Figure 7.9 presents the locations of the potential alternative water sources.

The following criteria were used to evaluate these sources:

- *Water Quantity* - adequacy of in-stream flow to permit diversion of a sufficient quantity of water
- *Water Quality* - acceptable quality of diverted water or need for additional treatment
- *Water Rights* - existing water rights that are assumed to be in effect for the duration of this project or the feasibility of a temporary water right being issued for the necessary quantity of water

Evaluation of the prospective water sources included field verification of the constructability of all proposed diversion structures and conveyance systems. In addition, water quantity and quality information has been collected, where available, from the Washington State Department of Ecology (DOE) and the USGS for the prospective locations.

Based on the descriptions presented below, none of the alternative temporary water sources appear to be a feasible alternative for supplying water to any of the water users during dam removal.

7.4.1 Upper Elwha River

The Upper Elwha River refers to a potential surface water diversion just upstream of Lake Mills within the Olympic National Park Boundaries and within a National Wilderness Area. A pump station, and over 68,000 feet of 48-inch pipeline with its potential adverse impacts to Park resources would be required to transfer water from above Lake Mills to the inlet of the current industrial channel. The environmental effects of the new pipeline through the Park and wilderness would have to be considered and would be subject to review.

The drainage basin above this proposed intake is approximately 198 square miles.

Figure 7.7 - Alternate 3 Single Storage Reservoir

Figure 7.8 - Alternative 3 Multiple Storage Reservoirs

Figure 7.9 – Alternative Water Sources

Water Quantity

USGS flow records collected from a temporary gauging station located above Lake Mills from 1994 through 1998 indicated a minimum in-stream flow of 132.9 mgd (206 cfs). Without consideration of in-stream flow needs, the Elwha River supplies sufficient quantities of water to meet current demands from the location of the existing intake. It was assumed that a temporary intake located above Lake Mills would also supply sufficient quantities to meet the proposed demands.

Water Quality

The Upper Elwha is located upstream of all disturbances that would be caused by dam removal; therefore, water quality should not be impacted by dam removal activities. Water quality should be generally comparable to current conditions, although short periods of increased suspended solids and turbidity would be expected during high flow events without the dampening effects of Lake Mills and Lake Aldwell.

Water Rights

There is no current water right on the Upper Elwha River that is located within the Olympic National Park. A federal reserved water right exists for flow in the Elwha River and its tributaries within the Olympic National Park. The U.S. Supreme Court has ruled that when the federal government reserved land it also, often through an implied intent, reserved enough water necessary for the purpose of the land reserved. Because this right is based on federal law, state law does not apply, except that state courts do have, under limited conditions, jurisdiction to determine the water requirements for the federal purpose. According to the NPS, the process for determining the extent and quantity of the federal reserved water rights for Olympic National Park has not occurred nor is there anticipation that it will occur in the near future.

The use of Park water by outside entities is governed by federal law and NPS policy. The requirements of Public Law 91-383, as amended by Public Law 94-458, have been incorporated into NPS policy in Director's Order #35A, Sale or Lease of Park Services, Resources, or Water in Support of Activities Outside the Boundaries of National Park Areas. This policy allows the use of Park water for public accommodations or services when there are no reasonable alternative water sources, and when water use would not jeopardize or unduly interfere with Park resources.

It is the opinion of the NPS that the water right on the Elwha River within the Park boundary would need to be established along with a study of the impact of this alternative on Park resources. The process could potentially be extremely lengthy and may represent an unfeasible delay in the dam removal process.

In addition, the proposed pump station and transmission pipeline to collect water from above Lake Mills would be located in a National Wilderness Area. Wilderness area policy discourages construction of mechanical facilities within its boundaries if other alternatives are available. Construction of a pump station, transmission pipeline, and electric power supply for this alternative would need to have the approval of the Superintendent, this may also significantly impact the dam removal process. Considering the various requirements necessary for obtaining water from the Upper Elwha, this alternate source of water supply is not recommended.

7.4.2 Morse Creek

Morse Creek is located east of Port Angeles and flows directly into the Strait of Juan de Fuca. The drainage area totals approximately 47 square miles. Until approximately 1977, Morse Creek was the primary source of domestic water for the City of Port Angeles.

Port Angeles City Light has had a hydroelectric plant on Morse Creek since 1987. According to discussions with City engineering staff, hydropower has not been produced from the plant for nearly 10 years, but the City is currently in negotiations with a private contractor to take over operation of the facility. The hydropower facilities include a single turbine generator unit housed in a masonry block structure located approximately 3 miles downstream of the City's Morse Creek Diversion Structure. The power station is capable of an average annual generation of approximately 3 million kilowatt hours and was designed for a maximum capacity of 12.3 mgd (19 cfs).

The City's use of Morse Creek water for the hydroelectric project is based on a change-of-use permit granted by DOE, allowing non-consumptive and emergency municipal drinking water supply use of Morse Creek during the life of the hydropower project.

In addition, the Clallam County Public Utility District (PUD) No. 1 diverts 0.97 mgd (1.5 cfs) of flow from Morse Creek to its water treatment plant located on the east bank of the creek.

Water Quantity

Flow data for Morse Creek is based on the gauging station at the existing Morse Creek intake structure. Flow data was kept for 11 years between 1966 and 1977. The mean annual flow in Morse Creek as measured at the Morse Creek diversion facilities is 77.4 mgd (120 cfs). The minimum flow recorded was 5.1 mgd (7.9 cfs) according to the USGS flow records. The flow data was used to determine river flow exceedance probabilities on a monthly basis and is shown in Table 4-6 of the *Comprehensive Water System Plan* (CH2M Hill, 1995).

The City's water right on Morse Creek, as described below, is for 12.9 mgd (20 cfs). The hydropower facility, including the 24-inch penstock, was designed for a maximum capacity of 12.3 mgd (19 cfs). The hydroelectric project is operated in accordance with in-stream flow requirements set by DOE in order to protect the fish habitat. These requirements allow for river flow-based operation of the project under conditions stated as follows:

- December through March and May through July – Hydropower operation allowed at all stream flows
- April – Hydropower operation allowed only at stream flows above 85 cfs
- August – Hydropower operation allowed only at stream flows above 90 cfs
- September – No hydropower operation permitted regardless of river flow
- October – Hydropower operation allowed only at stream flows above 100 cfs
- November – Hydropower operation allowed only at stream flows above 65 cfs

Based on a review of the river flows at the various exceedance probabilities demonstrates that the hydropower station operation and/or the potential to obtain significant flow for an alternative industrial supply is severely constrained by the maintenance of river flows to protect fish habitat.

Water Quality

The water quality of Morse Creek is analyzed by the Clallam County PUD because Morse Creek is one of its primary water supplies. Water quality data is presented in Table 4-5 of the *Comprehensive Water System Plan* (CH2M Hill, 1995). The data reflect three seasonally distinct sampling periods, and are indicative of a relatively high quality water source that is comparable to the Elwha River.

Water Rights

The City of Port Angeles currently holds two Morse Creek water rights. The minimum stream flow water right for 15 cfs, certificate number 874, was granted in 1935, certifying the City's use since 1920. The second right for 5 cfs, certificate number 2345, was granted in 1945, certifying the City's use since 1922. A change-of-use permit was granted for each of these water rights in 1985 to allow the City's water rights, totaling 20 cfs, to be used for a new hydroelectric power generation station. The change-of-use was from municipal use to non-consumptive use for hydroelectric power generation and for emergency municipal water system supply use during the life of the hydroelectric project. The City's intention was that the hydropower diversion was a temporary use of the water from Morse Creek, and the City expected to maintain its municipal water system right.

In 1993, superseding water rights were granted for these Morse Creek water rights that could be interpreted to indicate that the City might not have the flexibility to exercise its original municipal use rights upon discontinuation of hydropower generation. The designated use for these Morse Creek water rights is currently under review by the City and DOE.

Other water rights in Morse Creek include seven small rights for domestic general, irrigation, and fish use, totaling 3.52 cfs, 0.25 acre-feet/year and 290 acre-feet/year. The Clallam County PUD's surface water right to Morse Creek is for a peak flow of 0.97 mgd (1.5 cfs) and annual withdrawal of 379 acre-feet/year (0.34 mgd as an annual average flow). The PUD also has a groundwater right to Morse Creek for a peak withdrawal of 1.94 mgd (3 cfs), but it was not in use as of 1995.

As described in the previous section, the Morse Creek water rights have seasonal restrictions to protect fish habitat. DOE has stated that the water right is not likely to be increased or to have the seasonal restrictions removed. In addition, DOE has stated that it does not support the concept of changing the municipal use water right to a temporary industrial use water right during dam removal.

Due to the limitations on water rights and the limited available capacity, this source has been eliminated from further consideration as a potential alternative source for the industrial/fishery supply during dam removal on the Elwha River.

7.4.3 Little River And Indian Creek

Little River and Indian Creek are tributaries to the Elwha River, Little River joins from the east and Indian Creek joins from the west. The surface diversions being considered would be located just upstream from the Elwha confluence point of each respective waterway, and it would be necessary to divert each water source to the location of the existing industrial diversion on the Elwha. Both creeks enter the Elwha River at the south end of Lake Aldwell. The drainage area for each creek is approximately 18 square miles.

Water Quantity

The small drainage areas indicate there would be a very limited amount of water available, although an accurate estimate cannot be quantified without some stream flow measurement or hydrologic evaluation. The water flows for each of these potential alternative water sources were estimated at 3.9 mgd (6 cfs) each based on the hydrologic characteristics of their basins.

DOE has stated that they do not support moving existing water rights upstream of the current location and that obtaining a water right for these tributaries would require hydrology studies and a potentially lengthy permitting process.

Due to the limited drainage areas and resulting limited stream flows, and the difficulty in obtaining permits for these tributaries, these sources have been eliminated from further consideration.

7.5 TREATMENT RESIDUALS DISPOSAL

7.5.1 Estimated Residual Quantities

One option for treating the industrial and fisheries demand as previously discussed would be to obtain water through an infiltration gallery or surface water intake and use chemical coagulants such as alum, to settle suspended solids in a treatment plant. The amount of residuals produced from a chemical treatment of the industrial and fisheries demand was estimated based on the daily sediment load predicted by hydraulic scenario 4 presented in the *Sediment Analysis and Modeling of the River Erosion Alternative Report* (BOR, October 1996). For a conservative estimate on residual production, it was assumed that 25% of the suspended solids in the river would pass through the infiltration gallery and enter the treatment facility. The quantity of material entering the treatment facility is a function of the suspended solids concentration passing through an infiltration gallery coupled with the demand for water being treated at that time. It was assumed that a coagulant dose of approximately 12 mg/L would be required to remove the suspended sediments. The maximum monthly flow rates for Daishowa, WDFW, and the Tribal hatchery were used in the analysis.

The analysis showed that the average quantity of treatment residuals produced is approximately 46 tons per day (dry weight) or approximately 11,000 cubic yards per day based on a 0.5% solids concentration. The treatment residuals are based on daily TSS in the river predicted by the sediment model reduced by 75% removal by an infiltration gallery. Then the additional 12 mg/l of coagulant at the predicted water demands of the industrial and fisheries users is added. These numbers represent an average per day quantity based on 1,219 days used in the hydraulic model to predict sediment transport during the anticipated dam removal period. The actual daily values will vary widely. On a peak turbidity day as much as 1,786 tons per day (424,000 cubic yards) of residuals could be produced and require disposal.

Because of the high volume of solids created treating the industrial and fisheries demand, the only viable options for disposal of these residuals would be to discharge them back into the Elwha River or pump them to the ocean. The City's sanitary sewer and wastewater treatment facilities could not handle the residual quantities generated. In addition, settling basins and landfill disposal are not a viable option because of the large sediment volume. Based on the average daily solids generation presented above, residuals dewatered to 25% solids would require the daily disposal of 220 cubic yards of solids, requiring roughly 22 truck loads a day. This number of truckloads is excessive and therefore is not a viable residuals disposal option.

7.5.2 River Discharge

The *Water Quality and Mitigation Measures Report* (BOR, March 1997) proposed that residuals created from the treatment of the industrial demand be disposed back into the Elwha River. Residuals disposal back to the source water body is not commonly permitted because of the potential impact to the receiving water. Since the Elwha River will be highly impacted from sediment as a result of dam removal and delta erosion, the release of treatment residuals back into the river is a logical and cost effective alternative method for disposal. The only difference between the residuals returned to the river compared to the solids within the river would be the addition of coagulation chemicals.

The *Water Quality and Mitigation Measures Report* (BOR, March 1997) proposed the use of the chemical coagulants alum along with PAM to flocculate the suspended solids to treat the industrial demand. Work was done and submitted to DOE on toxicity testing of a PAM coagulant (Magnafloc 905-N, manufactured by Cytec Industries Inc.) to determine the impact on embryos of the mussel, *Mytilus galloprovincialis,* from residual discharge. The results of the *Toxicity Test Report* (Northwestern Aquatic Sciences, January 1999) indicate PAM concentrations of 300 mg/L impaired larval survival and development, but no significant abnormal development was observed in the 10 mg/L PAM test.

The concentration of PAM in the water column and sediment of the Elwha River after release of the treatment residuals would be significantly less than 10 mg/L due to the dilution of flow within the river. In addition, the disposal of residuals back into the river would only be required during the period when the Elwha River was significantly impacted by dam removal or delta erosion. Once the river returns to background water quality conditions, treatment of industrial and fisheries water would not be required, and therefore residuals disposal would no longer be an issue.

Despite the test results and condition of the river during dam removal, DOE has expressed concern over the release of residuals back into the river. The proposed action would likely require a modification to existing water quality standards. As part of the NPDES and 401 permit review, DOE would have to consider whether such a modification was justified. Generally, DOE does not permit deleterious material removed from a water body to be returned to a water body. In addition, DOE had concern over the release of PAM back to the river and the possible toxicity, fate and transport in the aquatic system. DOE has stated that they are more familiar with the release of alum residuals back into receiving waters and that alum as a sole coagulant may be preferential. The effectiveness of alum alone to coagulate the suspended solids is not typical because the addition of polymers with alum is effective in reducing overall chemical costs. The use of polymers however is typically very small, often in the range of 1 to 5 mg/L.

7.5.3 Ocean Discharge

This option entails pumping the residuals as a slurry directly to the ocean as described under the alternative disposal options for municipal residuals. This option would require higher capital costs for the pipeline, ocean outfall, and pumping station than discharging to the Elwha River, as well as higher operation costs for pumping. As with discharging to the Elwha, this option would also require environmental monitoring.

Discharge to the ocean would require a NPDES permit from the state under WAC 173-220.

7.6 INDUSTRIAL AND FISHERIES WATER SYSTEM ALTERNATIVES

7.6.1 Alternative 1 – Infiltration Gallery Followed by Chemical Treatment

7.6.1.1 Description

This alternative is a refinement of the mitigation measure proposed in the 1996 EIS. The industrial and fishery intake will consist of an infiltration gallery. Water from the infiltration gallery will be further treated through a conventional treatment process consisting of chemical addition, flocculation and sedimentation. The location of the site and a site layout for this alternative are shown on Figures 7.3 and 7.4. The design criteria and conceptual drawings for the infiltration gallery are presented in Appendix H.

The surface water intake facility is sized to provide 148 mgd (229 cfs) and the treatment plant is sized to handle 51.2 mgd (79.3 cfs).

It is proposed that intake and treatment facilities be shared between the City and Tribe. It is also proposed that a new surface water intake facility be constructed to supply water to the existing industrial diversion channel for long-term supply of water without the need for pumping or treatment.

Water will be conveyed to the treatment plant via a 54-inch pipe connected to the vertical riser structure through the existing intake tunnel. In-line blenders will be utilized for chemical addition, and each blender will have a chemical diffuser back-up. Chemical addition is anticipated to include caustic soda, alum, and polymer. Chemicals and doses used for estimating required equipment and sludge quantities are based on jar testing performed for the *Water Quality Analysis and Mitigation Measures Report* (BOR, March 1997).

The conceptual design of a rectangular basin treatment facility is shown on Figure 7.4. The treatment facility consists of two trains, each train treating half of the design capacity. Each train consists of a three chamber flocculation process with tapered mixing energy using variable speed flocculating mixers. Water then flows to a sedimentation basin with tube settlers to increase the loading rate and decrease the required detention time necessary for settling. After settling, a portion of the flow is pumped back to the head of the WDFW fish rearing channel, and the remaining flow is put into the current industrial pipeline. Water for the Tribal hatchery is split from the industrial pipeline as shown on Figure 7.3. Design criteria for the industrial/fishery treatment facility are presented in Appendix I.

Treatment residuals will be collected from each train through two longitudinal and one lateral chain and scrapper collection system. Residuals will be moved into a sump and pumped for ocean or river discharge. Anticipated residual quantities and disposal are discussed further in Section 7.5. For the purpose of developing a conservative cost estimate, it was assumed that treatment residuals would be pumped along a new pipeline to the ocean for discharge.

This alternative does not include treatment for the City's remaining industrial capacity of 68.7 mgd (106.3 cfs). In order to obtain and treat the 68.7 mgd of capacity, another treatment facility would need to be constructed. The treatment facility could be located in the area just south of the proposed facility due to space constraints. A cost for this increased intake and treatment quantity is included in the following section.

The industrial and fishery treatment facility will need to be constructed prior to the construction of the infiltration gallery. During construction of the treatment facility, water will continue to flow from the existing surface water diversion into the industrial channel. The construction of the infiltration gallery will have an adverse affect on water quality. Water for industrial and fishery use will need to be treated during infiltration gallery construction. It is anticipated that a temporary pumping facility will be required to by-pass the surface water diversion tunnel until the treatment plant is connected to the infiltration gallery.

7.6.1.2 Costs

Capital cost and operation and maintenance cost for this alternative is presented in Table 7.2. A detailed cost breakdown for Alternative 1 is presented in Appendix J.

Table 7.2

CAPITAL COST AND OPERATION AND MAINTENANCE COST FOR ALTERNATIVE 1 - INFILTRATION GALLERY FOLLOWED BY CHEMICAL TREATMENT

	Capital Cost without Reserve Industrial Treatment Capacity 148 mgd (229 cfs) Intake 51.2 mgd (79.3 cfs) Treatment	Annual O&M Cost without Reserve Industrial Treatment Capacity 148 mgd (229 cfs) Intake 51.2 mgd (79.3 cfs) Treatment	Capital Cost with Remaining Industrial Treatment Capacity 119.9 mgd (185.6 cfs)[1]	Annual O&M Cost with Remaining Industrial Treatment Capacity 119.9 mgd (185.6 cfs)[1]
Alternative 1 – Infiltration gallery and treatment facility	$47,477,000	$1,135,000	$84,300,000	$1,475,000

Based on first quarter year 2001 prices

[1] 51.2 mgd + 68.7 mgd remaining industrial treatment capacity = 119.9 mgd

Long-term operation and maintenance costs will be significantly less than the information above. As stated earlier, it is proposed that a surface water diversion be used to supply water via gravity to all users after dam removal and related erosion of the delta material has subsided. There would be no pumping cost or treatment cost for long-term operation. Long-term O&M costs will involve only operation of the infiltration gallery, not the chemical treatment facilities.

7.6.1.3 Advantages/Disadvantages

Advantages

- Conventional treatment is a proven and accepted technology.
- Single point of water intake for all industrial and fishery use.

Disadvantages

- Conventional treatment process is land intensive.
- Some chemical coagulant residual may remain in the treated water that is used by fisheries.
- Feasibility of conventional treatment for industrial/fishery use depends on the ability to discharge treatment residuals for 3 to 5 years to the ocean or back into the Elwha River during the dam removal and delta erosion period.
- Possible clogging or limited yield of the infiltration gallery, even with back-flushing capability. May require periodic renovation of filter pack to regain capacity.
- Conventional treatment processes require high operator proficiency.
- Infiltration gallery and treatment will be abandoned after dam removal and delta erosion period is over. No salvage value of the capital cost is anticipated.
- Shared use of the infiltration gallery and treatment facility will require policy negotiations between the City and Tribe.
- Legal consideration for Tribe associated with off-reservation supply of water.

7.6.1.4 Environmental Issues

The largest environmental issue associated with this alternative is the disposal of treatment residuals from the conventional treatment process. Treatment residuals include river sediments and coagulation chemicals.

Treatment residual disposal for the industrial/fishery treatment facility is the critical issue on the feasibility of this alternative. The quantity of treatment residuals produced during dam removal are based on the anticipated water quality criteria discussed in Section 7.5. The costs associated with this alternative were developed for pumping the residuals along a separate pipeline to the ocean for off-shore discharge. A substantial cost saving could be achieved with a DOE permit to discharge residuals back into the Elwha River.

7.6.1.5 Required Permitting/Legal Considerations

- Permits related to construction of infiltration gallery within the river.
- Right-of-way for access to infiltration gallery site.
- Discharging treatment residuals into river or ocean only during dam demolition and subsequent delta sediment erosion phase.
- Obtain Tribal right for use of water from the infiltration gallery location, if required.

7.6.1.6 Additional Data Needs

- Jar tests to determine coagulant doses and potential need for alkalinity adjustment.
- Soil boring to determine groundwater level and soil suitability for treatment plant at the proposed site.
- Model infiltration gallery in lab to determine feasibility and design criteria.
- Anticipated potential impact of coagulant residual on fish hatchery and rearing facilities.
- Impact of residuals on receiving water (ocean or river).

7.6.2 Alternative 2 - New Surface Water Intake Facility Followed by Chemical Treatment

7.6.2.1 Description

This alternative is similar to Alternative 1 but instead of an infiltration gallery a new surface water intake facility is provided and chemical treatment and a high solids clarification process is provided to provide water not to exceed 20 NTU. The chemical treatment and clarification process will be designed for a higher solids loading than the process anticipated for Alternative 1. This is required because the infiltration gallery for Alternative 1 has the capability to remove more solids than is anticipated for a surface water intake. The location of the facilities and layout are presented in Figure 7.10.

The surface water intake facilities and clarification facilities will be a combined facility to provide water for both the City and Tribe. The industrial and fishery water supply intake facilities are sized to provide 148 mgd (229 cfs) and the treatment plant is sized to provide 51.2 mgd (79.3cfs). Three circular clarifiers are proposed with related chemical storage and feed, rapid mixing, sludge pumping and flow splitting. The clarifier design will be similar to units used for mining operations that are suited for high loadings of solids. Up to four additional clarifiers can be added to provide the remaining City industrial water supply of 68.6 mgd (106.3 cfs) with each clarifier having a capacity of 17 mgd. Space for one additional clarifier is available at the site shown on Figure 7.10 but additional units would have to be located elsewhere. The clarifiers are shown as circular units on Figure 7.10 but may also be rectangular sedimentation basins as indicated for Alternative 1.

Figure 7-10 – New Surface Water Intake Followed by Chemical Treatment

The treated water leaving the clarifiers will flow to a flow splitting structure for distribution to the entities using the water. These entities would include at least the following:

- City Industrial
  Daishowa
  WDFW Rearing Channel
- Tribe

7.6.2.2 Costs

Capital and operation and maintenance costs for this alternative are presented in Table 7.3. A detailed cost breakdown for Alternative 2 is presented in Appendix K.

Table 7.3

CAPITAL COST AND OPERATION AND MAINTENANCE COSTS FOR ALTERNATIVE 2 - NEW SURFACE WATER INTAKE FACILITY FOLLOWED BY CHEMICAL TREATMENT

	Capital Cost without Reserve Industrial Treatment Capacity 148 mgd (229 cfs) Intake 51.2 mgd (79.3 cfs) Treatment	Annual O&M Cost without Reserve Industrial Treatment Capacity 148 mgd (229 cfs) Intake 51.2 mgd (79.3 cfs) Treatment	Capital Cost with Remaining Industrial Treatment Capacity 119.9 mgd (185.6 cfs)[1]	Annual O&M Cost with Remaining Industrial Treatment Capacity 119.9 mgd (185.6 cfs)[1]
Alternative 2 – Surface water intake facility followed by chemical treatment	$42,800,000	$1,000,000	$72,200,000	$1,400,000

Based on first quarter year 2001 prices

[1] 51.2 mgd + 68.7 mgd remaining industrial treatment capacity = 119.9 mgd

If an additional clarifier were provided with a capacity of 17 mgd (26.3 cfs) to provide addition water for industrial development in Port Angeles either by Daishowa or others, the capital cost for the additional clarifier and related pumping, piping and related facilities would be $7,300,000.

Long-term operation and maintenance costs will be significantly less than that presented in Table 3. It is proposed that the new surface water diversion and intake will remain in use but the need for treatment will not be necessary after the effects of the dam removal and delta erosion period has subsided. Long-term O&M costs will involve only operation of the diversion and intake, not the clarification facilities.

7.6.2.3 Advantages/Disadvantages

Advantages

- The existing surface water intake has served the industries and City successfully for many years and the new intake is expected to provide similar service.
- Fish protection and allowance for migration will be provided.
- Chemical treatment and clarification are conventional processes that are proven.
- A single water intake in a stable portion of the river will be provided for a reliable water supply that is less subject to the effects of river migration.
- A single treatment facility for all of the water users will be more efficient and less costly to operate than multiple facilities.

Disadvantages

- Conventional treatment is land intensive.
- Some chemical residuals may remain in the treated water that is used by the fisheries and plant operations will have to monitor these residuals to assure water quality requirements are satisfied.
- Feasibility of conventional treatment for industrial/fishery use depends on the ability to discharge treatment residuals for 3 to 5 years to the ocean or back into the Elwha River during the dam removal and delta erosion period.
- The treatment process will require high operator proficiency.
- After the dam removal and delta erosion period is over the river will return to background water quality and it is anticipated that the treatment facilities will no longer be required. The salvage value of the treatment facilities will be limited unless the Tribe and City continue to use the treatment facilities at their discretion. The intake will remain in use.
- Shared use of the surface water intake facilities and treatment facilities will require policy negotiations between the City and Tribe.
- Legal consideration for the Tribe will be needed associated with off-reservation supply of water.

7.6.2.4 Environmental Issues

The major environmental issue associated with this alternative is the disposal of treatment residuals from the treatment process. Treatment residuals include river sediments and coagulation chemicals.

Treatment residual disposal for the industrial and fishery treatment facility will be similar to that proposed for Alternative 1. Disposal of the residuals will be to the river or the ocean as presented in Section 7.5. The capital and operating costs for disposal of the residuals was based on piping them to the river for discharge. This discharge to the river would be only during the dam demolition and erosion phase of the restoration when the demolition activities and erosion would already degrade water quality.

This alternative varies from the *Water Quality Analysis and Mitigation Measures* (Reclamation, March 1997) recommendations in that a new surface water intake is proposed instead of an infiltration gallery that was proposed to be used during and after the dam removal and delta erosion period.

7.6.2.5 Required Permitting/Legal Considerations

- Permits related to construction of a new surface water intake facility in the river.
- Discharging treatment residuals into the river or ocean during the dam removal and delta erosion period.
- Tribal legal consideration is required for use of water from an off reservation location, if required.

7.6.2.6 Additional Data Needs

- Jar tests to determine coagulant doses and the potential need for alkalinity adjustment.
- Soil borings to determine groundwater level and soil suitability for treatment plant at the proposed site.
- Anticipated impact of residuals on the receiving water (ocean or river).

7.6.3 Alternative 3 – Reduced Treatment Capacity Concept

This section does not present a new alternative, but rather, presents a way of reducing the amount of water that would need to be treated to meet the industrial and fisheries demand. *The Sediment Analysis and Modeling of the River Erosion Alternative* (BOR, October 1996) modeled the impact of dam removal on the water quality of the Elwha River. Water quality will be impacted by the process of dam removal combined with rainfall and subsequent runoff within the watershed. The majority of the sediments that will impact water quality are trapped behind the Glines Canyon dam. The proposed dam removal process consists of cutting a series of blocks out of the dam to allow the reservoir to drain. The reservoir will first be lowered 80 feet to the level of the penstock intake and the exposed concrete dam removed. Then a series of blocks will be cut out of the dam to lower the reservoir in a stepped process. This process allows for the controlled release of water and sediments. By releasing the water in a series of controlled events, the majority of the trapped sediments, that are located within the delta, will be gradually re-deposited along the length of the reservoir and ultimately down the length of the Elwha River. During the removal of each block, a release of highly turbid water will result as settled sediments upstream of the dams are resuspended in the water and released over the dam.

During the actual dam removal process, the timing of the release of suspended sediments and impact to the river will be a controlled process. Two factors influence the timing of the block removal and subsequent sediment release. The first factor will be rainfall and runoff inflow to the reservoir. During periods of high inflow to the reservoir, no blocks are scheduled due to concerns over safety and water quality. These periods include winter high flows (typically during November and December through January) and spring high flows (typically from April or

May through July). There are no block removals scheduled during these periods and the sediment model reflects this.

The second factor that influences the timing of block removal is the seasonal return of the salmon for spawning. During specific times of the year when salmon are typically returning to the Elwha to spawn, dam demolition will stop so that water quality is not adversely impacted. These periods are known as “fish windows”. It is anticipated that these breaks in the dam removal process will allow an opportunity for the fish to begin the migration upriver, where they can be captured and allowed to spawn within the protected environment of a hatchery. The initial sediment model developed in the 1996 report included these fish windows. Since 1996 the fish windows have been modified and additional model runs were developed to reflect the changes. These additional model runs were used for this reduced capacity concept analysis.

As stated earlier, the release of suspended sediments from the reservoirs during the dam removal process is directly related to the amount of water running into the reservoir from the watershed and the timing of dam notching. To examine the effects of inflow on the model, the model was run under four separate hydraulic scenarios. Each scenario was based on actual hydrograph data of the watershed during four different past time periods. Hydrologic scenario 1 includes the lowest peak discharge for any three consecutive water years of record. Scenario 4 includes the highest peak discharge for any three consecutive years of record. Scenarios 2 and 3 were arbitrarily chosen to represent the range between extremes of scenarios 1 and 4.

The sediment model was run for each of these hydraulic scenarios and showed that water quality returned to near background conditions within four years for each scenario. Figures 7.11 through 7.14 show the predicted TSS concentrations within the river during each of the hydraulic scenarios. These figures also show the maximum monthly water demand for all of the industrial and fisheries consumers. The maximum monthly water demand is shown in Table 7.4.

Table 7.4

INDUSTRIAL AND FISHERIES MAXIMUM MONTHLY WATER DEMAND

Month	WDFW Fish Rearing Facility (cfs)	Tribal Hatchery (cfs)	Daishowa (cfs)	Total Demand (cfs)
January	1	12.6	21.7	35.3
February	9.7	19.1	21.7	50.5
March	4.1	21.4	21.7	47.2
April	20.6	26.1	21.7	68.4
May	31.1	28.8	21.7	81.6
June	36.1	2.9	21.7	60.7
July	5	3.5	21.7	30.2
August	33	4.3	21.7	59.0
September	33	5.3	21.7	60.0
October	33	6.5	21.7	61.2
November	1	8.3	21.7	31.0
December	1	10.5	21.7	33.2

Figure 7.11 Hydraulic Scenario 1

Figure 7.12 Hydraulic Scenario 2

Figure 7.13 Hydraulic Scenario 3

Figure 7.14 Hydraulic Scenario 4

Figures 7.11 through 7.14 show that the periods of highest water demand do not correspond to the periods of poorest water quality. An analysis was performed to determine if a combination of surface water and treated water could be used to supply the total industrial and fisheries maximum monthly demand and what the impact of the surface water component would be. The goal of this analysis was to reduce the amount of treatment facilities required to supply clean water for all users. This reduction in treatment capacity would be a direct reduction in project cost. This reduction in treatment facility capacity applies to each of the four alternatives presented in Section 5.

An analysis was performed that looked at each day during the 3 year 4 month modeling period for each hydraulic scenario. It was predicted from the model that immediate water quality impacts of dam removal would subside 1,218 days after the first day of dam removal activities. The water quality for each day of the model run was compared against the maximum industrial and fisheries demand for that day. Every day the TSS concentration was greater than 20 mg/l, and the maximum industrial and fisheries demand for that day was greater than a range of selected design capacities, that day was counted as having surface water quality greater than background conditions. A value of 20 mg/L TSS was selected as the background condition because it represents a current average TSS concentration the Elwha River.

For example, as indicated on Figure 7.15 a design capacity of the new water supply facilities of 83.5 cfs has zero days when Elwha River surface water TSS would be greater than background, because a facility or groundwater system designed for 83.5 cfs will meet the maximum monthly demand of all users and surface water directly from the River would not be required. As shown on Figure 7.15, if the design capacity of a facility or groundwater system was sized for 62 cfs, the model predicts that between 5 and 10% of the days during a 1,218 day period, surface water plus 62 cfs of groundwater (or new water supply) could be used to meet the maximum industrial and fisheries demand, but that surface water would have a TSS concentration greater than background.

This represents a risk to the industrial and fisheries consumers. Up to 122 days during the Elwha restoration project, Daishowa, the WDFW facility and the Tribal hatchery would experience TSS greater than background. To further investigate an example of the risk to consumers, hydraulic scenario 3 was examined further. Hydraulic scenario 3 was selected because it represents the scenario with the greatest number of days surface water TSS would be greater than background. The analysis was conducted assuming that a treatment facility or groundwater system was sized to handle 62 cfs. The analysis resulted in the following risk scenarios:

- It appears that the only months of concern for this scenario are April and May of years one and two, respectively. The worst case risk scenario based on total length of consecutive days presented above would be 15 consecutive days in the end of April year one with an average TSS to consumers of 399 mg/L, followed by 25 consecutive days in May of year one with an average TSS concentration to consumers of 216 mg/L. This is a total of 40 days of water quality greater than the background. This represents an unacceptable risk based on the length of time of each of these events and the quality of water available for use.
- During the month of April, 68 cfs of water is required to meet the industrial and fisheries demand, only 62 cfs of treated water is available, leaving 6 cfs of surface water required to meet demand. In April of year one, there are 15 consecutive days when the average surface water TSS concentration is 4,519 mg/L. When 6 cfs of 4,519 mg/L TSS water is combined

Figure 7.15 Percentage of Days TSS in Surface Water Greater than 20 mg/L with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 399 mg/L on average.

- During the month of May, 82 cfs of water is required to meet the industrial and fisheries demand, only 62 cfs of treated water is available, leaving 20 cfs of surface water required to meet demand. In May of year one, there are 25 consecutive days when the average surface water TSS concentration is 888 mg/L. When 20 cfs of 888 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 216 mg/L on average.
- In April of year two, there are 30 consecutive days when the average surface water TSS concentration is 1,963 mg/L. When 6 cfs of 1,963 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 173 mg/L on average.
- In May of year two, there are 2 consecutive days when the average surface water TSS concentration is 895 mg/L. When 20 cfs of 895 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 218 mg/L on average.
- In May of year two, there are 5 consecutive days when the average surface water TSS concentration is 2,382 mg/L. When 20 cfs of 2,382 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 581 mg/L on average.
- In April of year three, there are 9 consecutive days when the average surface water TSS concentration is 36 mg/L. When 6 cfs of 36 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 3 mg/L on average.
- In May of year three, there are 14 consecutive days when the average surface water TSS concentration is 36 mg/L. When 20 cfs of 36 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 9 mg/L on average.
- In May of year three, there are 8 consecutive days when the average surface water TSS concentration is 68 mg/L. When 20 cfs of 68 mg/L TSS water combine with 62 cfs of clean water, the resulting TSS of the water delivered to the industrial and fisheries consumers is 17 mg/L on average.

It does not appear that reducing the design capacity of a treatment facility or groundwater system below the maximum monthly demand of 51.2 mgd (79.3 cfs) will be feasible because of the limitations it places on the operations of the hatchery and rearing channel and the risk of a limited number of days when there is not acceptable water quality. In addition, this reduced capacity concept relies heavily on the sediment transport model and does not provide the reliability required to ensure the water quality needs of the various users are met.

7.7 ALTERNATIVES EVALUATION

Table 7.5 is a summary of the capital cost, annual operation and maintenance costs, advantages and disadvantages of each of the industrial and fisheries alternatives discussed in Section 7.

Alternative 3 – The Reduced Capacity Concept, was not included in this summary as it was determined not to be feasible. The capital and operation and maintenance costs are presented for providing treatment for both maximum demand and for the remaining industrial water supply (remaining industrial water supply and treatment costs in parenthesis). A summary of the water quantities during the mitigation period used to develop the costs is presented below.

Port Angeles Municipal	10.6 mgd (16.4 cfs)
Port Angeles Industrial - WDFW Fish Rearing Facility	14.2 mgd (22 cfs)
Port Angeles Industrial - Daishowa	14 mgd (21.7 cfs)
Tribal Fish Hatchery	12.4 mgd (19.2 cfs)
Total Water Demand During Dam Removal and Delta Erosion Period	**51.2 mgd (79.3 cfs)**
City's Remaining Industrial Capacity	**68.6 mgd (106.3 cfs)**

Table 7.5

INDUSTRIAL AND FISHERIES ALTERNATIVES EVALUATION

Alternative	Capital Cost	O&M Cost	20-Year Present Worth	Advantages	Disadvantages
Alternative 1 – Infiltration Gallery with Chemical Treatment Facility	$47,477,000 ($84,300,000)	$1,135,000 ($1,475,000)	$60,495,000 ($101,218,000)	• Conventional treatment is a proven and accepted technology. • Single point of water intake for all industrial and fishery use.	• Conventional treatment process is land intensive. • Some chemical coagulant residual may remain in the treated water that is used by fisheries. • Feasibility of conventional treatment for industrial/fishery use depends on the ability to discharge treatment residuals for 3 to 5 years to the ocean or back into the Elwha River during dam removal. • Possible clogging or limited yield of the infiltration gallery, even with back-flushing capability. May require periodic renovation of filter pack to regain capacity. • Conventional treatment process requires high operator proficiency. • Infiltration gallery will be abandon after dam removal and delta erosion period is over. No salvage value of the capital cost is anticipated. • Shared use of the infiltration gallery and treatment facility will require policy negotiations between the City and Tribe. • Legal consideration for Tribe associated with off-reservation supply of water.

Alternative	Capital Cost	O&M Cost	20-Year Present Worth	Advantages	Disadvantages
Alternative 2 – New Surface Water Intake Facility Followed by Chemical Treatment	$42,800,000 ($72,200,000)	$1,000,000 ($1,400,000)	$54,270,000 ($88,260,000)	• The existing surface water intake has served the industries and City successfully for many years and the new intake is expected to provide similar service. • Fish protection and allowance for migration will be provided. • Chemical treatment and clarification are conventional processes that are proven. • A single water intake in a stable portion of the river will be provided for a reliable water supply that is less subject to the effects of river migration. • A single treatment facility for all of the water users will be more efficient and less costly to operate than multiple facilities.	• Conventional treatment is land intensive. • Some chemical residuals may remain in the treated water that is used by the fisheries and plant operations will have to monitor these residuals to assure water quality requirements are satisfied. • Feasibility of conventional treatment for industrial/fishery use depends on the ability to discharge treatment residuals for 3 to 5 years to the ocean or back into the Elwha River during the dam removal and delta erosion period. • The treatment process will required high operator proficiency. • After the dam removal and delta erosion period is over the river will return to background water quality and it is anticipated that the treatment facilities will no longer be required. The salvage value of the treatment facilities will be limited unless the Tribe and City continue to use the treatment facilities at their discretion. • Shared use of the surface water intake facilities and treatment facilities will require policy negotiations between the City and Tribe. • Legal consideration for the Tribe will be needed associated with off-reservation supply of water.

Notes:

[1] Costs for providing water quantities based on maximum demand, then for demand based on the providing the remaining City industrial water supply are in parenthesis.

[2] Present worth costs are for capital and operating costs over 20 years at an annual compounding interest rate of 6%.

[3] All costs are based on year 2001 prices.

7.8 RECOMMENDED INDUSTRIAL AND FISHERIES WATER SYSTEM

Based on an evaluation of performance, capital cost, operation and maintenance issues and cost, and environmental concerns, it is recommended that Alternative 2 – New Surface Water Intake Facility Followed by Chemical Treatment be provided to meet the industrial and fisheries water supply requirements during the dam removal and delta erosion period of 3 to 5 years.

The use of surface water to meet long-term water needs was recommended for each of the alternatives developed using the surface water intake facility provided with Alternative 2. The new surface water intake facility would allow sediment to pass downstream, while allowing fish migration.

It is recommended that the surface water intake and chemical treatment facilities be a shared facility between the City and the Tribe at the site of the City's existing intake. There is a high potential for river migration downstream of the existing industrial intake. The site of the current industrial intake provides the most reliable location for gravity driven water withdrawal.

The concept of a shared surface water intake facility has many political and operational challenges including ownership, liability, and maintenance. In addition, the proposed surface water intake would require the Tribe to obtain their hatchery water off reservation. For these reasons, a separate Tribal, gravity driven, infiltration gallery located on the Halberg property was investigated. A preliminary investigation of the separate Tribal infiltration gallery indicated that a river bank infiltration gallery contains unacceptable risks for a reliable supply compared to a surface water intake located at the site of the City's existing industrial diversion. Risks associated with a river bank on-land infiltration gallery located on the Halberg property include unknown, uncontrolled filter performance from natural soils and inability to assure performance with highly probable river migration at the proposed location.

The proposed schedule for the implementation of planning, engineering and construction of Elwha water quality mitigation is presented in Figure 8-1. The schedule indicates the industrial and fisheries mitigation to be complete by June 2005 and the municipal water treatment mitigation to be complete by mid July 2005.

An independent review of the planning process was conducted at the 10%, 50%, and 75% completion levels of this report. A Value Study Team (VST) consisting of representatives from the National Park Service, Bureau of Reclamation, URS Corporation, the City of Port Angeles Engineering Department, and CH2M Hill conducted the review for the first two review sessions. The final review session also included Gathard Engineering Consulting as engineering representative from the Lower Elwha Klallam Tribe.

The goal of the VST was to follow a Job Plan that provides a reliable, structured approach to achieve the most appropriate and highest value solution for the project. Initially, the team examines the component features of the project to define the critical functions, governing criteria, and associated costs. Using creativity (brainstorming) techniques, the team suggests alternative ideas and solutions to perform those functions, consistent with the identified criteria, at a lower cost or with an increase in long-term value. The ideas are evaluated, analyzed, and prioritized, and the best ideas are developed to a level suitable for comparison, decision making and adoption.

The results of the VST process included a series of proposals for the design team to consider during the development of this report and ultimate conclusions on the recommended alternatives. A brief summary of each of the proposals developed by the VST is presented in Appendix L, along with the design team's response to those proposals. The VST also brainstormed numerous other ideas that were not developed as in depth as the proposals. A deposition of those ideas and the response from the design team is also included in Appendix L.

Appendix L

Value Study Team Proposals, Deposition Of Ideas, And Design Team Response

The first VST report is dated October 10, 2000 and evaluated the project at the 10% completion stage. The following is a brief summary of the nine proposals developed by the VST and the Design Team's response to how those proposal affected the development of this report. A complete description of the value planning process, VST members, and detailed proposals including potential cost savings can be found in the Value Planning Report (BOR, October 2000).

Proposal	Design Team Response
1. Construct raw water storage for industrial users at the Rayonier site	Raw water storage was considered and developed as an industrial and fisheries mitigation alternative.
2. Construct treated water storage for Port Angeles Municipal Use	Storage for the municipal demand was not considered because all of the recommended municipal treatment alternatives would supply and treat the maximum daily demand.
3. Industrial water treatment by ceramic microfiltration	The physical separation of suspended solids from water was found to be infeasible due to unknown water quality and inability to pilot test. Conservative conceptual cost estimates made this process more expensive than other alternatives developed.
4. Pilot test infiltration gallery	A bench scale test of infiltration gallery performance is currently being conducted by URS
5. Reduce proposed infiltration gallery capacity	This proposal refers to the size of the infiltration gallery proposed in the 1996 EIS. This report has used a reduced capacity for conceptual level design.
6. Use Ranney collectors to supply industrial and fisheries demand	This proposal was considered and developed as an industrial and fisheries mitigation alternative.
7. Obtain water for municipal supply from industrial and fisheries infiltration gallery	A cross connection between the industrial and fisheries intake to the municipal supply is proposed within this report
8. Use Ranney collector for municipal and Daishowa demand, use infiltration gallery for fisheries and reserve industrial capacity demand	The design team will evaluate this proposal after the Hydrogeological recommendations are available.
9. Bench top testing for disk filtration	The manufacturers of disk filtration units have indicated that their process is not appropriate for consideration without the actual water quality available for pilot testing. The actual water quality is unavailable until the dams are removed.

The following is a list of the VST deposition of ideas and corresponding Design Team response. These ideas were not developed in detail by the VST.

Floating Fabric Storage (5-10 day).	Not developed in favor of other storage ideas.
Pumped Storage (underground).	Considered by study team to be infeasible.
Line and use Rayonier Mill tank for storage.	See Proposal 1 above
Fill up converted oil tanker for storage.	Not developed in favor of other storage ideas.
Barge mounted desalting plant in the strait for water supply.	Desalination is the most expensive form of treatment.
Pay for Daishowa Mill reduced operations or temporary closures.	Lost customer base and market share from plant closures is unacceptable to Daishowa
Build enhanced water treatment system with state of the art controls, operating with minimum labor.	Proposed water treatment system design will attempt to minimize operating costs without excess capital cost.
Use Morse Creek as a permanent municipal water supply.	Not developed; insufficient creek flows.
Use groundwater wells as emergency or temporary supply when turbidity is too high.	The use of groundwater was considered and developed as an industrial and fisheries mitigation measure.
Use new and existing Ranney collectors as permanent source for all water use.	See Proposal 6 above.
Drill an exploratory well to test for groundwater yield.	A hydrogeologic study of available groundwater in the area is currently being conducted by URS
Remove Glines Dam first to partially control sediment impact and mitigation needs.	Sequential dam removal was considered as part of the EIS process and rejected.
Build reservoir and treatment facility at the Rayonier site.	Rayonier site is unavailable due to environmental investigation and potential future commercial development.
Combine Tribal and State Fisheries.	This idea was not acceptable to the WDFW and Tribal fisheries.
Increase treated storage capacity.	See Proposal 2 above
Increase treated and raw water storage (5-10 days).	5-day storage was considered and developed as an industrial and fisheries mitigation alternative.
Construct infiltration gallery on Elwha River above the dams (Lake Mills) to supply all water needs during dam deconstruction.	This proposal was considered and developed as an industrial and fisheries mitigation alternative. Intake above Lake Mills was determined to be infeasible.

Cut municipal demand by conservation and economic incentives.	Conservation as a mitigation measure against dam removal was unacceptable to the City.
Promote water efficient fixtures.	Conservation as a mitigation measure against dam removal was unacceptable to the City.
Ration water in Port Angeles during high turbidity periods.	Conservation as a mitigation measure against dam removal was unacceptable to the City.
Switch to ozone disinfection during deconstruction to eliminate concerns with trihalomethane (THM).	Ozone was considered as a disinfection alternative.
Upgrade Daishowa water treatment plant to optimize process and water use.	Daishowa has indicated that additional solids into their plant will impact their NPDES permit and claim that it will not meet the requirements of the Elwha Act. Upgrades to Daishowa are considered as a risk reduction rather than primary mitigation measure.
Combine domestic/industrial infiltration galleries.	See Proposals. 5, 7, and 8 above
Diatomaceous earth treatment for potable; and combination of reuse and storage for non-potable water during dam removal.	Diatomaceous earth was considered and developed as a municipal mitigation alternative.
Relocate State Fishery (Rearing Channel).	This is unacceptable to the WDFW based on a recent listing of Puget Sound chinook Salmon under the Endangered Species Act
Reduce municipal demand by implementing wastewater reuse; install a reclaimed water system.	A non-potable system to reduce municipal demand is not acceptable to the City and would not reduce cost of the project.
Recycle water through fisheries during high turbidity periods.	Water recycling through the WDFW fish rearing channel or Tribal hatchery is not acceptable to either entity.
Use solids from coagulation/flocculation for beach rebuilding or landfill cover.	Transportation of solids from industrial and fisheries treatment is infeasible based on the projected quantities.
Use industrial disk filter pretreatment.	See Proposal 9 above.
Postpone construction/use of New Ranney Well and restore and maintain yield of existing Ranney by periodic jetting of laterals and flushing of materials or by adding new laterals.	Cross connection of industrial intake to municipal intake will eliminate the need to modify existing Ranney.
Use centrifugal sand separators.	Quantity of water required for industrial and fisheries treatment make this an unfeasible option based on cost

Use sluicing system to separate coarse sediments.	Sluice system would not remove a significant fraction of the coarse sediments compared to an infiltration gallery or settling basin.
Develop a true groundwater system for permanent municipal supply to avoid surface water treatment rule.	There is not a sufficient source of groundwater in the immediate area that would not be impacted by surface water
Promote 75-percent industrial water reuse.	Conservation as a mitigation measure against dam removal was unacceptable to Daishowa.
Avoid withdrawing any water from Elwha during high turbidity.	Alternative sources were considered and developed as an industrial and fisheries mitigation alternative.
Use porous ceramic system for removal of solids.	See Proposal 3 above.
Use proceeds from electric generation sales to fund mitigation.	Refer to the BOR and ONP.
Use Ultraviolet disinfection to reduce disinfection by-products (DBP).	Ultraviolet disinfection was considered and developed as a municipal mitigation alternative
Use pilot systems.	See Proposal 4 above.
Use benchtop tests, for example test membranes with core samples.	See Proposal 9 above.
Analyze and test competing technologies.	Testing competing technologies not appropriate without actual water quality.
Define available acreage at existing facilities for comparison to new facility footprints.	Acreage and siting issues have been considered.
Bring product/equipment representatives to site for consultation.	Equipment manufactures have been consulted in the development of this report.
Identify model municipalities' performance (i.e., the Ranney collector at Kelso on the Cowlitz River, near Mount Saint Helens.	No municipality has had to mitigate against the impacts of dam removal. The performance of other systems to similar events (i.e. Mount St. Helens, mudslides) are highly dependant on the specifics of the installation (i.e. geology, hydrogeology)
Gather more information on infiltration gallery performance and Ranney collector performance.	Bench scale testing currently being conducted by URS
Use Ultraviolet disinfection with no residuals. Monitor residuals with non-heterotropic system.	Ultraviolet disinfection was considered and developed as a municipal mitigation alternative
Look at the risk/benefit of the total systems.	Risk/benefit has been considered during the alternatives evaluation process.

Make new facilities modular to allow increases/decreases during dam removal.	Modular features will be incorporated into design.
Sell concrete to recycler to fund mitigation.	Refer to BOR and ONP

The second VST report is dated November 14, 2000 and evaluated the project at the 50% completion stage. The following is a brief summary of the eight proposals developed by the VST and the Design Team's response to how those proposal affected the development of this report. A complete description of the value planning process, VST members, and detailed proposals including potential cost savings can be found in the Value Planning Report (BOR, November 2000).

Proposal	Design Team Response
1. Look at cycles in water use verses predicted water quality to try and reduce required treatment facilities. Utilize water reuse.	This concept was considered and developed as an industrial and fisheries mitigation alternative.
2. Reuse water from WDFW facility to supply Daishowa.	Water passing through WDFW channel used as attraction flow to capture new fish. If alternatives to attraction flow are developed by fisheries group, this proposal may be possible.
3. Use vertical wells in place of Ranney collectors	This proposal was considered and developed as an industrial and fisheries mitigation alternative.
4. Replace conventional treatment plant with membrane filtration.	This proposal was considered and developed as a municipal mitigation alternative.
5. Determine actual capacity of proposed Ranney collectors rather than using 10 mgd assumption.	A hydrogeologic investigation is currently be conducted by URS which will address this proposal
6. Install first new Ranney collector to determine capacity and impact on existing.	A hydrogeologic investigation is currently be conducted by URS which will address this proposal
7. Sequential installation of Ranney collectors to determine performance.	A hydrogeologic investigation is currently be conducted by URS which will address this proposal
8. Salvage and reuse Rayonier water supply pipeline.	The cost of this proposal was actually more than using new construction. In addition the City may need to retain that pipeline to supply new industry at the former Rayonier site.

The following is a list of the VST deposition of ideas and corresponding Design Team response. These ideas were not developed in detail by the VST.

Go with all Ranney wells, and a raw water intake off the surface, and blend both sources.	See Proposal 1 above.

Use State Hatchery water (less attraction flow, if needed) for input to the papermill.	See Proposal 2 above.
Refine the cost of Ranney Wells (current costs, other providers).	Conceptual level costs of Ranney collector based on manufacturer information and engineers estimate.
Return State Fishery water (less attraction flow, if needed) to industrial supply line and reduce Alternative 1 infiltration gallery capacity and treatment effort by 30 cfs or reduce Alternate 2 by one Ranney well.	See Proposal 2 above.
Review Ranney well capacity. Is 10 mgd enough, why not more?	See Proposal 5 above.
Install Ranney wells sequentially. Start with No. 3 and see if it influences No. 4. Then adjust spacing (optimize capacity) as necessary.	See Proposals 6 and 7 above.
Use Ranney wells sized for current demand for industrial needs. Design for 48 cfs and use fisheries outflow as backup or add a Ranney well for backup, and maintain surface water diversion.	See Proposal 1 above.
Use aquifer water storage and recovery to meet peak demands, downsize facilities for industrial and municipal supply.	Groundwater injection and aquifer storage is not considered practical because of the limited storage capacity of the aquifer.
For the municipal water treatment system, go with rapid mix flocculation and multimedia filtration (eliminate clarification).	Direct filtration was considered as a municipal treatment alternative.
Salvage the existing waterline between the Daishowa and Rayonier Mills to use for the Tribal hatchery.	See Proposal 8 above.
Make industrial water treatment in Alternative. 1 modular to facilitate reuse somewhere else after the 7± years of use	Alternative 1 is not the preferred alternative. If water treatment is required, the design will be as modular as possible.
Delete the infiltration gallery in Alternative 3 and use the reservoir for supply until the peak passes.	Alternative 3 is not the preferred alternative based on water quality issues.
In Alternative 1 build one set of settling basins and split the flow to industrial and city users.	Settling basins would not remove colloidal particles.
Evaluate Daishowa discharge water for reuse for fisheries or municipal needs.	Daishowa is downstream of all other users, reuse would require treatment and pumping.

Develop unit cost for alternatives based on life cycle costs (include O&M).	A present worth analysis was performed on all alternatives considered.
Use a well point system for industrial source instead of Ranney wells.	The operational complexity and unknown water quality associated with well points make this proposal less favorable than Ranney collectors. More information on the feasibility of well points will be obtained from the hydrogeologic investigation currently being conducted by URS
Test sediment spreading in the river. Place sediment or other media in the river and measure attenuation and duration for the sediment/media event to pass the intakes.	Sediment pilot testing in the river was not considered due to adverse impacts to river ecology and proposed project schedule.
Look at flows and durations from Mount St. Helens, including secondary damage to better estimate impact of sediment release.	Sediment transport model was developed by BOR
Aggressively pursue short term conservation efforts.	Conservation as a mitigation measure against dam removal was unacceptable to the City.
Stage the construction of Ranney wells in Alternative 2 over several years (build closer to need).	See Proposal 7 above.
Reuse tribal fisheries discharge for Daishowa intake.	Water reuse from Tribe facility may be required if insufficient groundwater is available to meet demand.
Drill six more State hatchery wells, install pumps as needed or in just two or three wells.	See Proposal 3 above.
Internal reuse of Daishowa mill, National Park Service (NPS) could pay for reuse facilities to minimize water needs.	Internal water reuse may be required if insufficient groundwater is available to meet demand.
Provide a 44.5 mgd infiltration gallery intake that is not used until worst sediment periods are over.	This is to supply Port Angeles reserve capacity in the event that it is required. This will be considered once the hydrogeological study findings are known.
Increase the capacity of the existing Ranney well.	Will not significantly reduce the amount of facilities or cost of providing water to meet total demand
Design a Ranney well with a higher capacity.	Ranney collectors will be designed to yield maximum capacity.
In Alternative 4, decrease the pipe diameter and augment flow with fishery water.	Alternative 4 is not the preferred alternative based on water rights issues.

Augment Daishowa mill with backwash water from the water treatment plant.	Filter backwash water is high in solids that would need to be removed and disposed by Daishowa.
Build storage at Morse Creek or the Little River.	Storage is not the preferred alternative based on water quality issues.
Show the cost share potential from the Tribe, City, County, State, and others. The study and design teams do not need it, but the decision makers, NPS and Congress need it.	A cost sharing proposal will be developed in coordination with ONP.
Develop a well field to supply backup source for city and go to a membrane filtration for treatment.	Well fields and membranes have been considered and developed as part of the mitigation alternatives.
Obtain the geologic report for Alternative 2 for less than $750,000.	A hydrogeologic study is currently being conducted by URS
Use Alternative 2 with membrane filter plant.	See Proposal 4 above.
Review cost models for value mis-matches.	Costs have been reviewed for consistency.
Better determine duration and concentrations of sediment flows.	See Proposal 1 above.
In Alternatives 3 and 4, remove the infiltration gallery and let the reservoir provide storage during shutdown; blend with low TSS water.	Alternatives 3 and 4 are not the preferred alternatives based on water quality and water rights issues.
Define flexible alternative methods of operation including mixing, switching, blending, storing, and conservation.	See Proposal 1 above.

The third VST report is dated February 16, 2001 and evaluated the project at the 75% completion stage. The following is a brief summary of the nine proposals developed by the VST and the Design Team's response to how those proposals affected the development of this report. A complete description of the value planning process, VST members, and detailed proposals including potential cost savings can be found in the report.

Proposal	Design Team Response
1. Enhance communication and transfer of critical project information and proposals among the different project planning teams.	The water quality mitigation design team has worked to keep all project planning teams informed on project developments.
2. Modify dam removal schedules so that periods of worst water quality occur during lowest monthly use.	The dam removal process/schedule is coordinated by BOR. The dam removal team will be reviewing the proposed schedule in the near future.
3. Release sediments during dam removal with a low level ungated outlet.	Sediment release scenarios were examined during the EIS. The natural erosion alternative was chosen.

4. Release sediments during dam removal with a low level gated outlet.	Sediment release scenarios were examined during the EIS. The natural erosion alternative was chosen.
5. Replace Ranney wells with well points.	The operational complexity and unknown water quality associated with well points make this proposal less favorable than Ranney collectors. More information on the feasibility of well points will be obtained from the hydrogeologic investigation currently being conducted by URS
6. Use presedimentation basin in place of infiltration gallery.	This idea would be considered if and infiltration gallery turns out to be infeasible based on bench scale testing and hydrogeologic investigation
7. Replace chemical treatment with slow sand filtration.	Filter surface area required for this alternative make it unfeasible for treating significant quantities of water.
8. Construct a more temporary industrial and fisheries treatment facility than proposed by design team.	If a chemical treatment facility is required for the industrial and fisheries demand, modifications to the proposed design will be considered.
9. Replace Ranney collectors with horizontal wells.	A hydrogeologic investigation is currently being conducted by URS, which will determine the feasibility of horizontal wells.

The following is a list of the VST deposition of ideas and corresponding Design Team response. These ideas were not developed in detail by the VST.

Time the removal of the lowest 30 feet of the dams for January – March, when water demand is low.	See Proposal 2 above
Remove the Dams without using the diamond wire saw.	The dam removal process is currently under review by the BOR dam removal team.
Build only one new Ranney well, across the river from the existing well, and well fields. (Ranney wells close to the Reservation may aggravate salt water intrusion problems.)	A hydrogeologic investigation is currently being conducted by URS that will determine potential locations and capacities of groundwater collectors, including the affect of salt water intrusion.
Use the 10 acres of wetlands adjacent to the State Hatchery for a pre-sedimentation basin.	See Proposal 6 above
Deliver 1,000 parts per million water to the Daishowa Mill for short (high concentration) periods and help the mill with solids disposal.	The ability of Daishowa water treatment plant to handle increased solids loading is considered as part of risk reduction, rather than as a primary mitigation measure.

Revise the hydrogeologic study to include salt water intrusion and well field analyses.	Already being done in hydrogeologic study.
Lower the dams in smaller steps (lower and more frequent sediment discharges).	The dam removal process is currently under review by the BOR dam removal team.
Revise the fish windows.	The fish windows are defined by fish migration.
Remove the sediments from behind the dams more rapidly.	The dredge and slurry technique for removal of lake sediments was examined in the 1996 EIS. It was not the recommended alternative.
Blend surface with fish hatchery water.	Reuse of fish hatchery water is being considered for risk reduction rather than as a primary mitigation measure.
Blend surface and well water.	This concept was considered and developed as an industrial and fisheries mitigation alternative.
Use Army Reserve/National Guard water purification units (up to 100,000 gallons per day per unit).	The use of temporary water purification units was not considered due to the length of time water treatment will be required.
Use an unlined dike reservoir for a storage reservoir	Water storage is not a preferred alternative based on limited land availability and water quality issues (temperature, dissolved oxygen, algae, etc.)
Increase interaction between Dam Removal, Flood Control, and Water Mitigation Teams.	See Proposal 1 above.
Imitate the 100-year flood event (in January).	Accelerating sediment transport during periods of low water usage to increase water quality reliability will only be necessary if untreated surface water is utilized for supply. The use of untreated surface water will be determined after the results of the hydrogeologic investigation.
Confine sediment effects to January-February.	This conflicts with the dam schedule and would require a more notching than can be fit into this short time period.
Blow up the dams (or lowest portion) and accelerate the sediment passage.	See Proposals 3 and 4 above.
Avoid the need for supplemental Environmental Impact Statements.	The need for a supplemental EIS will be determined by the National Park Service after consideration of the proposed and recommended alternative.
Place a filter cloth at the tunnel entrance.	Determined by the study team to be less feasible due to product limitations and cost.

Construct plywood or other temporary treatment facility with a 5-year design life and 3-month active use period.	See Proposal 8 above.
Use Jersey barriers and plastic for basin walls.	This idea would be appropriate if a temporary industrial and fisheries treatment system is selected based on limited available groundwater.
Use Ecology blocks for basin walls.	This idea would be appropriate if a temporary industrial and fisheries treatment system is selected based on limited available groundwater.
Use a slow sand filter instead of treatment for industrial (and fisheries) water.	See Proposal 7 above.
Use horizontal wells under the river.	See Proposal 9 above.
Use well points instead of infiltration gallery.	See Proposal 5 above.
Add laterals to the existing Ranney Well.	The capacity of the existing Ranney collector can easily be supplemented by an interconnection to the industrial intake.

Appendix A

Port Angeles Ranney Collector Water Quality

Appendix B

Daishowa Turbidity Data

Appendix C

Rayonier TSS Data

Appendix D

National Primary Drinking Water Regulations

Appendix E

Municipal Treatment Cost Estimating Details

Appendix F

Manufacturer Information On Actiflo Process

Appendix G

Cost Estimate For On-Site Residual Dewatering Option

Appendix H

Design Criteria for Industrial and Fisheries Infiltration Gallery

Appendix I

Design Criteria For Industrial and Fisheries Treatment Facility

Appendix J

Alternative 1 – Infiltration Gallery and Treatment Cost Estimating Details

Appendix K

Alternative 2 – New Surface Water Intake Facility Followed by Chemical Treatment Cost Estimating Details

Appendix K
Alternative 2 – New Surface Water Intake Facility Followed by Chemical Treatment Cost Estimating Details