GENERAL SEWER PLAN

CITY OF
NORTH BONNEVILLE,
WASHINGTON

Updated
by

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CERTIFICATE OF ENGINEER

GENERAL SEWER PLAN

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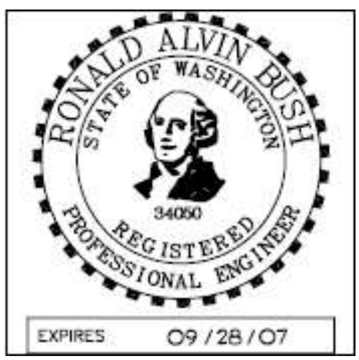

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1.0 INTRODUCTION

1.1 Purpose

This report has been prepared to assist the City of North Bonneville (City) in planning for the continued operation and maintenance of their existing wastewater treatment and disposal system. It reviews the existing service area, describes the existing facilities and evaluates their ability to meet needs of the present community, and proposes improvements to serve the community as it is expected to develop over the design period through the year 2017 and then modified in April of 2006 which brought the design period to 2026. This report has also been prepared to meet the standards of the State of Washington Department of Ecology (Ecology) established in WAC 173-240-050. Parametrix Inc. prepared and submitted the General Sewer Plan to the State of Washington Department of Ecology (DOE) in 1997. That Plan was approved by DOE. The modifications in 2006 were performed by Ronald A. Bush. The City of North Bonneville contracted with Ronald A. Bush to make modifications to the Plan associated with the inclusion of the property located on the north side of the City that was not included in the original version of the Plan.

The City is located in Skamania County, a county with a population of less than 50,000 persons. Skamania County has adopted a policy of excluding planning under the State of Washington Growth Management Act (GMA) and the requirements of WAC 365-195.

The City of North Bonneville owns, operates, and maintains the system of sewers, pumping stations, and wastewater treatment facilities serving the area under the provisions of NPDES Permit WA-002338-8, enclosed in the Appendix A. There are no notifications, stipulations, or final orders against the City for non-compliance with the permit. The system has been in operation for approximately 28 years. No previous engineering studies or general sewer plans have been prepared for the City treatment plant. The system is beginning to require additional maintenance, as in the replacement of pumps, and it was recognized that an engineering study was needed.

The decision to modify the Plan to include parcels of property lying along the north side of the City was made due to zoning changes of this property from timber conservancy to residential type zoning and the owner of the property informing the City of the intent to develop the parcels.

1.2 Study Area

The City is approximately five miles from the “Bridge of the Gods” west on State Route 14 in the scenic Columbia River gorge. A vicinity map is shown in Figure 1-1.

The City is located on relatively level alluvial deposits and fill. The principal soil groups in the area are the Arents, Bonneville stony sandy loam, and Pilchuck very fine sandy loam as described and mapped by the “Soil Survey of Skamania County Area, Washington (USDA, 1990). These soils can all be generally described as deep, “somewhat excessively well-drained” soils of alluvial origin. The Arents soil differs from the others in that it is formed from recent construction fill instead of from naturally deposited soils. All have high permeability, but are subject to brief flooding and high water tables, which limit their use for on-site disposal systems. The soils are suitable for residential and commercial construction provided their limitations (flooding, caving of excavation walls, etc.) are considered. These limitations are not expected to be a limiting factor to area development.

Most of the city is bounded by the Columbia River on the south and Hamilton Creek, Greenleaf Lake, and Greenleaf Creek on the north. One small residential neighborhood is located north of Hamilton Creek. Although all of these water bodies are within the 100-year flood plain, the banks are steep and the flood zone does not extend into any residential or commercial areas, nor does it extend into the wastewater treatment plant property. The flood zone is indicated on the Sanitary Sewer System Map in Appendix C.

Climatological data was collected at Bonneville Dam and is summarized for the period of 1951-1973 in Table 1-1 (USDA, 1990).

Figure 1-1

TABLE 1-1.—TEMPERATURE AND PRECIPITATION
(Recorded in the period 1951-73 at Bonneville Dam, OR)

Month	Temperature					Precipitation				
	Average daily maximum	Average daily minimum	Average	2 years in 10 will have-- Maximum temperature higher than--	2 years in 10 will have-- Minimum temperature lower than--	Average	2 years in 10 will have-- Less than-	2 years in 10 will have-- More than-	Average number of days with 0.10 inch or more	Average snowfall
	°F	°F	°F	°F	°F	In	In	In		In
January	42.4	32.2	37.3	58	13	13.14	7.03	18.14	18	8.2
February	47.6	35.2	41.4	62	22	8.14	4.85	11.09	14	1.1
March	51.9	36.4	44.2	69	26	8.45	5.34	11.26	16	1.1
April	58.8	41.0	49.9	79	31	5.31	3.06	7.15	11	0.0
May	66.1	46.5	56.3	89	34 3.52	2.03	4.73	8	0.0	
June	71.3	52.1	61.7	93	40	2.52	1.16	3.61	6	0.0
July	78.3	56.1	67.2	99	46	0.70	---	1.21	2	0.0
August	78.1	55.7	66.9	99	46	1.47	0.17	2.45	3	0.0
September	73.0	52.4	62.7	94	41	3.11	1.27	4.60	6	0.0
October	62.1	46.1	54.1	80	36	6.83	3.21	9.77	12	0.0
November	50.7	39.2	45.0	65	26	10.85	5.92	14.87	15	0.4
December	44.1	34.2	39.2	61	17	13.14	9.03	16.90	18	2.3
Yearly:										
Average	60.4	52.2	52.2	---	---	---	---	---	---	---
Extreme	---	---	---	102	6	---	---	---	---	---
Total	---	---	---	---	---	77.18	66.40	87.57	129	13.2

1.3 Population

The 1997 population of North Bonneville is 539 persons. In April of 2005 the population of North Bonneville was 741. The City does not have a current comprehensive plan, thus there is no estimate of future populations. To estimate future loads on the wastewater treatment plant (WWTP), population projections were essential. Two approaches were used to estimate future populations for preparation of this General Sewer Plan:

1. From the past ten years of population records, the historical average growth rate (four percent) was used to project future populations.

2. Existing zoning and density information for residential, commercial, and industrial developments were used to determine future “buildout” (maximum density). City planning staff then estimated the rate at which various zones would be developed .

Both of these methods included new community developments which had not previously been considered:

- Bonneville Hot Springs Resort- A private development for 120 room resort, now under construction in the northeast corner of the sewered area of the City.

- Skamania County Business/Light Industrial Area- A proposed light industrial area north of the west-end of Evergreen Drive.

- Parcel 2 - This is a newly proposed light industrial area south of State Route 14 and east of the Cascade Drive entrance from State Route 14.

- Parcel C - This is a potential multi-family residential area east of the wastewater treatment plant.

- New mobile home and residential subdivisions, which have been submitted to the City for, review.

- Approximately 160 acres of land along the north side of the city, presently owned by Richard Beckman adding up to 410 residential units and two commercial sites.

- Approximately 21 acres of land located at the northwest corner of the City which had a zone change from Timber Conservancy to Single Family adding 60 single family units.

The possible future development north of Greenleaf Lake was not considered in original version of this plan because it was zoned as a "timber conservancy" and was excluded from the project's scope-of-work. The property has subsequently been rezoned to Single family and Commercial Recreational designations and the owners are proceeding with plans to develop the property. Upon satisfactory submission of the necessary permits and planning information, the developer will be required to update the relevant City planning studies information (the General Sewer Plan and Water System Plan, for example). This modification to the General Sewer Plan is to determine the impact on the wastewater facilities due to the change in land-use status of the properties that are being added to the service area and what steps will be necessary to assure that the functionality and integrity of the wastewater collection and treatment facilities is maintained.

Table 1-2 presents a comparison of the results of the two population projection methods as well as a prediction of buildout population. Appendix B contains a tabulation of how the population projections were developed.

Table 1-2. Comparison of Population Projections

Year	Four Percent Growth	City Planning Staff Estimate
1997	539	550*
2002	656	1,050
2007	798	1,150
2012	971	1,261
2017	1181	1,381
2023	1494	1,543
2025	1616	1,600
Buildout	2,875	2,875

* Includes 11 commercial population equivalents.

In order to develop the new buildout population for the City due to the zoning change to the properties on the north side of the City an average household size of 2.3 individuals per dwelling unit was used. The number of dwelling units was determined from property zoning and developer submitted information showing development plans on the specific parcels of property. By multiplying the total number of possible/proposed lots by 2.3 a number of additional population was determined. The increase in buildout population is 1081 individuals. That number was added to the buildout population number stated in the 1998 General Sewer Plan to get the new buildout population.

Figure 1-2 presents a graphical comparison of the two population projection methods. The two methods result in population projections within 15 percent for the year 2017 and less than 1% in 2025. The straight line four percent growth method shows a uniform increase in population. The “City” method shows a short term increase in growth and then a more uniform rate of increase. The latter short term increase is more realistic because it accounts for the near term development of the Bonneville Hot Springs Resort, which adds 240 persons when the resort is fully occupied.

Figure 1-2

A public presentation was made to the North Bonneville City Council and Planning Commission on October 30, 1997 on the alternative population projections. The "City" method of population projections was adopted by the City Council and Planning Commission. The projections presented at the public meeting were subsequently modified and the most current are presented herein.

1.4 Background

The current City of North Bonneville was constructed in 1976. The old town was demolished and residents relocated in a new town to allow construction of the second powerhouse for the Bonneville Dam.

The construction of sanitary sewers and wastewater treatment plant (WWTP) was by the US Army Corps of Engineers using the design drawings and specifications by Daniel, Mann, Johnson & Mendenhall/Hilton (US Army, 1976a, b, c) and the Corps of Engineers (US Army, 1978). The original construction cost of the system in 1976 was $2,434,700 (Beck & Associates, 1984).

The water system consists of a City well, reservoir, and water mains. In addition, there are five private wells, a County well, and four geothermal wells. The water system and wells are shown on the map in Appendix C.

1.5 Design Loading

Based upon current sewage flow and population data and industry-standard design criteria future WWTP loadings (flow, organic strength, and solids concentrations) were estimated using future population projections. These loadings were based upon the measured average per capita wastewater flow rate of 100 gallons per capita per day (gpcd). The maximum day flow rate was estimated using a peaking factor of 4.0 based upon Washington State Department of Ecology guidance (Ecology, 1985) and review of WWTP operating records. The maximum month flow rate peaking factor was estimated to be 1.55 based upon good engineering judgement given the maximum day peaking factor of 4.0.

The average BOD_5 loading observed for North Bonneville was 0.12 pounds per day per capita (ppd/cap). The maximum month BOD_5 loading will be based on the Washington Department of Ecology (WDOE) *Criteria for Sewage Works Design* (1998) recommended value of 0.20 ppd/cap. The maximum day BOD_5 loading was statistically estimated as 0.24 ppd/cap using two standard deviations from 1995-1997 data (one standard deviation was 0.06 ppd/cap).

The average total suspended solids (TSS) loading observed for North Bonneville was 0.126 pounds per day per capita (ppd/cap). The maximum month TSS loading will be based on the WDOE recommended value of 0.20 ppd/cap. The maximum day TSS loading was statistically estimated as 0.267 ppd/cap using two standard deviations from 1995-1997 data (one standard deviation was 0.07 ppd/cap).

Table 1-3 summarizes existing and future design loadings for flow, BOD_5, and TSS. The design loading for this General Sewer Plan is based upon 20 year projections, or the year 2017 and on to 2025:

Flow (2017) average of 138,160 gpd and peak of 552,400 gpd.
BOD_5 (2017) – average of 166 ppd, maximum month of 276 ppd, and maximum day of 331 ppd.
TSS (2017) – average of 174 ppd, maximum month of 276 ppd, and maximum day of 369 ppd.
Flow (2025) average of 160,000 gpd and peak of 640,000 gpd.
BOD_5 (2025) – average of 192 ppd, maximum month of 320 ppd, and maximum day of 384 ppd.
TSS (2025) – average of 202 ppd, maximum month of 320 ppd, and maximum day of 427 ppd.

Table 1-3. Existing and Future Design Loadings for Flow, BOD_5, and TSS

		Flow			BOD_5			TSS		
Year	**Population**	**Average, gpd**	**Max Month, gpd**	**Peak, gpd**	**Average, ppd**	**Max Month, ppd**	**Max Day, ppd**	**Average, ppd**	**Max Month, ppd**	**Max Day, ppd**
1995-96	550	54,000		278,000 (Feb 96)*	68	110	170 (Jan 95)	68	110	183 (Nov 96)
2002	1049	104,900		419,600	126	210	252	132	210	280
2007	1152	115,200	180,000	460,800	138	230	276	145	230	308
2012	1266	126,600	197,813	506,400	152	253	304	160	253	338
2017	**1381**	**138,100**	**215,781**	**552,400**	**166**	**276**	**331**	**174**	**276**	**369**
2023	1543	154,300	541,094	617,200	185	309	370	194	309	412
2025	1600	160,000	250,000	640,000	192	320	384	202	320	427
Buildout	2875	287,500	449,219	1,150,600	345	575	690	362	575	768

1.6 System Administration and Budget

The system is administered by the Mayor and City Council and with the assistance of the Clerk-Treasurer. System costs are paid with revenue collected from connection fees and rates for sewer service set in the City Code. The current rates are presented in Table 1-4. The total revenue for the sewer fund in the year ending December 31, 2005 was $67,241.

Table 1-4. Sewer Connection and Service Fees

User classification	Connection fees	Sewer service
Single family residential		Monthly, $30.00
4-inch service	$2,000 per dwelling unit	
6-inch service	$3,000 per dwelling unit	
8-inch service	$4,000 per dwelling unit	
Multy family residential		Monthly, $35.00
4-inch service	$2,000 per dwelling unit	
6-inch service	$3,000 per dwelling unit	
8-inch service	$4,000 per dwelling unit	
Commercial, industrial zones		Based on Water Meter Size
4-inch service	$2,000 per dwelling unit	Up to ¾" - $35.00/mo.
6-inch service	$3,000 per dwelling unit	1-in. - 40.00/mo.
8-inch service	$4,000 per dwelling unit	1-1/4 & 1-1/2" - 50.00/mo.
		2-in. - 100.00/mo.

		3-in. - 165.00/mo. 4-in. - 230.00/mo. 6-in. - 305.00/mo. 8-in. - 425.00/mo. 10-in. - 550.00/mo.
Commercial/Industrial Equivalent Service Charge	$8.00 per ESU	ESU Based on Use-Billed Monthly with Regular Sewer Service Fee

The treatment plant, pump stations, and collection system are operated and maintained by approximately 1 FTE. The City public works department has 4 FTEs with 3 certified plant operators. All the public works employees may work on the sewer system, but on average it amounts to approximately 1 FTE. See detailed breakdown from Utility Manager in Appendix J.

1.7 Environmental

The documentation for the State Environmental Policy Act (SEPA) for this General Sewer Plan was completed by the City of North Bonneville and is in Appendix H
.

2.0 WASTEWATER COLLECTION SYSTEM

2.1 Existing System – Description, Capacity, Future Needs

The system consists of approximately 3.9 miles of gravity sanitary sewer, four sewage pumping stations, 0.4 miles of pressure sewer (force main), and a secondary WWTP discharging to the Columbia River (U.S. Army Corps of Engineers, 1976a, b, c; U.S. Army Corps of Engineers, 1978; Tenneson, 1980). The system is described in detail by the Operations & Maintenance Manual (Anonymous, 1979).

The original gravity sanitary sewers, installed in 1976, were installed using "Truss pipe" manufactured of ABS plastic with a cementaceous fill within the "truss-like" structure between the inner and outer pipe walls. Joints were made with solvent-welded, slip-on collars. Ductile iron pipe with "push-on" joints was used for the gravity sewers under State Route 14 and railroads. Force mains were also installed using ductile iron pipe although some asbestos-cement pressure pipe was also used; both piping systems use push-on joints. Sewer laterals for individual connections were installed by the contractor, generally using ABS plastic pipe. The new extensions to the system have been installed using PVC plastic gravity sewer and PVC or ABS plastic sewer laterals. The vast majority of the gravity sewer consists of 8-inch pipe except for selected portions of 6 and 10 inch (Appendix C).

The existing system of sanitary sewers serves the entire developed area of the City. Future sewers will be installed by developers and connected to the existing system.

The sanitary sewers were installed approximately 20 years ago and remain in good condition and provide satisfactory service. New sanitary sewers are installed using rigid PVC sewer pipe. The force mains were installed using ductile iron pressure pipe and are in excellent condition. Manholes were installed using precast concrete sections and, except as noted below in the discussion of inflow and infiltration, are in satisfactory condition. The sanitary sewers and force mains all have satisfactory capacity for the anticipated future growth of the community and no new sanitary sewers are proposed in this plan. Any new sewers to be installed by developers or the City should be designed and constructed to meet the standards set by Ecology (1985).

There have been extensions to the system over the past 20 years to extend service to developing areas within the City and areas being annexed to the City. No service is provided to areas outside City limits. The current limits of the service area and existing facilities which constitute the publicly owned sewage disposal works are shown on the drawing in Appendix C. Record drawings of the system are available at the City WWTP. The minimum capacity is 0.5 mgd on 8-inch diameter sewer and 0.8 mgd for the 10-inch diameter truck sewers. Storm water is collected separately and discharged to local creeks, ditches, and the Columbia River. The use of the sanitary sewer system is regulated by the City of North Bonneville Municipal code (North Bonneville, 1997). The City does not permit sewer service outside the City limits. There are no other WWTPs within 20 miles or in the same drainage basin as the City which could connect to this system (Meyer, 1997). Sewage from the power house area is treated on-site near the dam.

The service area is primarily residential but also serves local commercial and small industrial customers. The number of users in each service classification is presented in Table 2-1. The volume of sewage discharged from the individual commercial and industrial customers was estimated as approximately equal to the volume of water used at each site. There is no significant industrial processing wastewater discharged to the system, and there is no requirement for specific pretreatment of industrial wastewater. (Ray Hays, 1997). A description of the individual customers in these classifications are presented in Table 2-2 along with their estimated individual discharge volumes over the most recent billing period.

Table 2-1. Number of Services by Customer Class (North Bonneville, 1997, 2005)

Customer Class	Year					
	1992	**1993**	**1994**	**1995**	**1996**	**2005**
Residential	148 Water 145 Sewer	151 Water 148 Sewer	163 Water 160 Sewer	166 Water 164 Sewer	174 Water*	231 Water 223 Sewer
Multifamily	13 Both	13 Both	13 Both	15 Both	15 Both	14 Both
Commercial	14 Both	14 Both	14 Both	14 Both	11 Both	30 Both
Industrial	3 Water 2 Sewer	3 Water 2 Sewer	3 Water 2 Sewer	4 Water 3 Sewer	2 Water 1 Sewer	2 Water 1 Sewer
* No data provided for sewer services in 1996.						

There are 11 units in the City served by septic tanks and subsurface disposal fields (Hays, 1997). Chapter 19.04.230 of the City Code regulates these systems. Septage from these systems is removed by private haulers and the majority is taken to the wastewater treatment plant in Goldendale, Washington. Septage haulers are not allowed to discharge at the North Bonneville WWTP or into the collection system.

The collection system has no bypasses or overflows.

2.2 Infiltration and Inflow

Sanitary sewers were designed to collect only wastewater, but rain water and groundwater also enter the system of almost all cities. Water from these sources reduces the capacity of the sewers and treatment facilities to carry and treat the flow they were intended to handle. As a result, the systems may be overloaded and unable to work properly. Water from these sources is generally referred to as "inflow" or "infiltration." Inflow is stormwater that enters a sanitary sewer system through roof leaders, access fitting for cleaning sewer lines ("cleanouts"), foundation drains, sump pumps, and cellar, yard, and area drains. It may also include water entering through older connections between sanitary and storm sewers and through defective manhole covers and frame seals (WEF, 1994). Infiltration is water that enters a sewer system from the ground through defective pipes, pipe joints, damaged lateral connections, or manhole walls. Infiltration most often is related to a high groundwater level but can also be influenced by storm events or leaking water mains (WEF, 1994).

Both inflow and infiltration (I/I) are present in almost all sewer and treatment plant systems but may not be present in such large volumes as to interfere with the operations. In such cases, they are considered as "non-excessive" inflow or "non-excessive" infiltration. Excessive I/I is where pumping stations and the treatment plant operations are impaired, either as evidenced by obvious flooding of the processes units, unsatisfactory quality, or by causing premature need for construction of larger facilities to carry the larger flows.

Table 2-2. Descriptions of Commercial and Industrial Customers (North Bonneville, 1997)

Customer	**Description**	**Billing Period**	**Average Daily Water Use, in gallons (Calendar Day Basis)**
Bonneville Hot Springs Resort	Residential/Recreational Community. 3 (Under construction, completion estimated 1998.)	7-31-96 to 9-30-97	124
Bonneville Power Authority	(Water customer but on own sewage system.)	9-30-96 to 9-30-97	76
Mini-Storage	Office.	7-30-96 to 9-30-97	65
Disc. Grocery & Quick Stop	Sanitary facilities.	7-30-96 to 9-30-97	28
Side Track Tavern	Sanitary facilities.	7-30-96 to 9-30-97	824
Dept. of Wildlife (Two services)	Sanitary facilities	7-30-96 to 9-30-97	4 22
Mansfield	Sanitary facilities (Not currently connected)	5-29-97 to 9-30-97	0
US Post Office	Sanitary facilities	9/30/96 to 9/30/97	2
Beacon Rock Golf Course	(Water customer but on own sewage system.)	7-30-96 to 9-30-97	1,198
K.W. Peterson	(Not currently connected))	7-30-96 to 9-30-97	6
Port – Thorsen	(Not currently connected))	7-30-96 to 9-30-97	4
H.H.Eco System	Produces bacterial cultures for wastewater and sludge treatment. (Two services)	7-30-96 to 9-30-97 7-30-96 to 9-30-97	67 79
US Army Corps of Engineers, Waterways	Sanitary facilities and marine biological laboratory.	1-10-97 to 9-30-97	27

The sewers in North Bonneville are capable of collecting and transporting flows of 0.5 mgd based upon a minimum slope for the 8-inch sewers. This capacity exceeds the present sewage flows from all sources prior to the inclusion of the properties to the north of Greenleaf Lake. If all of the flow from the properties North of Greenleaf Lake were to be directed into either pump station 2 or 4 the collection system downstream of these pump stations would not have the capacity to convey the flows to Pump Station Number 1. If the flows are split evenly between the two pump stations the existing piping system will have the capacity to convey the flows without surcharging the existing system. The pumping stations are also generally capable of the capacity needed for the flows, when the properties north of Greenleaf Lake are not included. However all of the pump stations in the system will require upgrading in order to handle the projected flows from the properties added to the system north of Greenleaf Lake regardless of how the flow is split between Pump Stations 2 and 4. The impact on the pump station due to the zone changes of the property north of Greenleaf Lake will be addressed in Chapter 3 of this report. The WWTP also has apparent capacity to treat the flow from all sources without violating the NPDES discharge permit conditions (North Bonneville, 1995, 1996, 1997) when not considering the properties north of Greenleaf Lake. By these standards, the system appears relatively unaffected by inflow or infiltration but other information is available for consideration. The increase in flows due to the zone changes of the properties north of Greenleaf Lake will significantly impact the existing facilities at the WWTP. This impact will be addressed in Chapter 4 of this report.

Evaulation from June 1998 General Sewer Plan

The City measures the sewage flow only at the WWTP where there is a Parshall flume and meter in the inlet structure. No other flow records, apart from readings from the newly installed water meters, are available to estimate sewage flow. The total 24-hour daily flow, from approximately 9:00 AM on the previous day, is read at about 9:00 AM on the day of the record. The flows are summarized in Figures 2-1, 2-2, and 2-3 for the calendar years of 1995, 1996, and the first nine months of 1997, respectively. These records reveal that high daily flows appear to be seasonally related, with lowest flows occurring regularly through the summer months, with higher flows during the wet winter months, and with extremely high flows during or following heavy rainfall events.

Where I/I is present, it is appropriate to consider implementing a sewer system evaluation survey. A survey of this type (USEPA, 1975) can be expensive, however, and the USEPA has developed guidelines, since adopted by Ecology, for determining if a survey is warranted. In brief, the relative flow per person from the affected sewer system is compared with the standards developed by USEPA (USEPA, 1991). Daily 24-hour flows recorded at the City treatment plant over the three year period, 1995-1997, indicate the summer dry weather flow averages approximately 0.45 mgd, or 83 gpcd. Wet weather daily flows, from periods not affected by direct precipitation but with high ground water conditions, are generally less than 120 gpcd except during the period from June 5 to June 17, 1997, when flows averaged 0.10 mgd, or approximately 200 gpcd. This is over the 120 gpcd USEPA limit for non-excessive infiltration. There is, therefore, presumptive evidence of excessive infiltration by the USEPA and Ecology standards.

To estimate the affect of inflow, pump cycling was examined on the influent pumps in the WWTP equalization basin. There is no current information available on flow over the daily diurnal cycles or on variations of flow within the day because the strip chart recorders at the WWTP have not been used during that time. Review of data from 1987, is presented in Figure 2-4. The charts show the reoccurring on-off operation of the pumps in Pump Station No. 1 against a chart with the daily time and a vertical scale having a maximum range of 0.7 mgd. Both sections of chart indicate the pumps are discharging approximately 130 gpm, as compared with their rated capacity of 200 gpm (the pumps have since been rebuilt and and then in 1998 upgraded to a pumping capacity of 540 gpm). The charts also show the pumps alternate (one pump has a slightly greater capacity than the other and there is regular alternation of high and low peaks). The operating cycles occur more frequently during the day when the sewage flow is at a higher rate than at night.

Figure 2-1

Figure 2-2

Figure 2-3

Figure2-4

The section of chart for the summer dry weather period shows a period of approximately two hours between pump operation cycles, corresponding to the time it took to fill the 1,500 gallon storage volume in the wet well. The flow during that period therefore was approximately equal to 1,500 gallons, divided by 120 minutes, or 12 gpm. The flow during the winter wet season was higher as it took only 20 minutes to fill the same 1500 gallon storage volume, and the flow was 75 gpm. The day time flow was also higher, exceeding the capacity of one pump requiring the second pump to operate; this left no reserve pumping capacity available at the pump station.

Review of flow data for several periods affected by direct precipitation (i.e. November 11, 1995; January 20, 1996; November 19 and 20, 1996; December 5, 8, 25,and 31, 1996; January 1 to 4, 1997; and January 31 to February 4, 1997) showed daily flows exceeding the limit of 275 gpcd developed by USEPA for non-excessive inflow, reaching a peak of approximately 430 gpcd. Based upon the above analysis, there is also presumptive evidence of excessive inflow by the USEPA and Ecology standards.

The City has previously recognized the presence of excessive I/I. An internal inspection of the sanitary sewers was made in December 1984 and early January1985 using a closed-circuit television camera (Gelco Grouting Services, 1985). The inspection report indicated 41 of the 83 manholes south of State Highway 14 were leaking and should be sealed. Only 6 of the 39 manholes north of the Highway showed similar leaks. Fifteen sections of the sewers [1] of the 122 sections in the system, showed leakage from up to 10 percent of the joints in the section of sewer [2]. Only a few sections of sewer were misaligned, were separated, or had improper service connections that need be repaired by excavation.

The City staff has also recognized surface water entering the system though vent holes in the manhole covers and has installed plastic sheets below the cover to reduce the flow from this source. Continued attention needs to be given to limiting flows from these I/I sources and to implement the recommendations of the 1985 Gelco report. Recommendations are presented in Section 6.0 of this report.

Follow-up evaluation

Based primarily on the information in the Gelco report, in 1999 and 2000 the City sealed all the known manhole section and rim joint leaks. In 2002 the City also completed a major street crack sealing and weed removal project. Together these activities were intended to significantly reduce the I/I in the collection system.

Figures 2.5, 2.6, and 2.7 present daily flow and rainfall information for 2005, 2006, and 10 months of 2007. The flow was measured at the wastewater treatment plant 45-degree v-notch

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[1] *A section of sewer is the sewer, of whatever length, between two consecutive manholes or cleanouts. Generally a section of sewer does not exceed 400-ft. in length.*

[2] *The truss-type sewer pipe used in most of the system has a length of 13-ft. per piece of pipe between joints. A 400-ft. section of sewer would have approximately 30 joints and 3 leaking joints would represent 10 percent of the total joints in the section of sewer.*

weir with a Stevens Axsys datalogger, which was most recently calibrated on February 23, 2007 by Dale R. Fraser. Typically the flowmeter is calibrated every 6 months.

Compared to figures 2.1 through 2.3, Figures 2.5 through 2.7 show slightly higher overall, but more consistent, plant influent flows. They also show a much lower correlation between rainfall events and increased flows. For example the rainfall patterns from spring of 1995 look similar to the rainfall from spring of 2007, but in 2007 the influent flow peaks deviated very little from their summer high of about 0.10 mgd while the 1995 peaks almost doubled their summer peaks.

The rainfall event in late November 2006 is the most significant high flow event in recent history and appears similar in magnitude to the event in February of 1996. For both events the rainfall slightly exceeded 5 inches in one day. The flow response in 1996 was approximately 0.28 mgd while the response in 2006 was only 0.23 mgd.

The graphs show that the I/I work completed in 1999 and 2000 appears to have been quite effective. City staff report that they continue to be progressive about sealing leaks as soon as they are discovered. In addition, the staff prepare a yearly I/I report evaluation report for the Annual Treatment Facility Review Report. The most recent of these evaluations in attached in Appendix K.

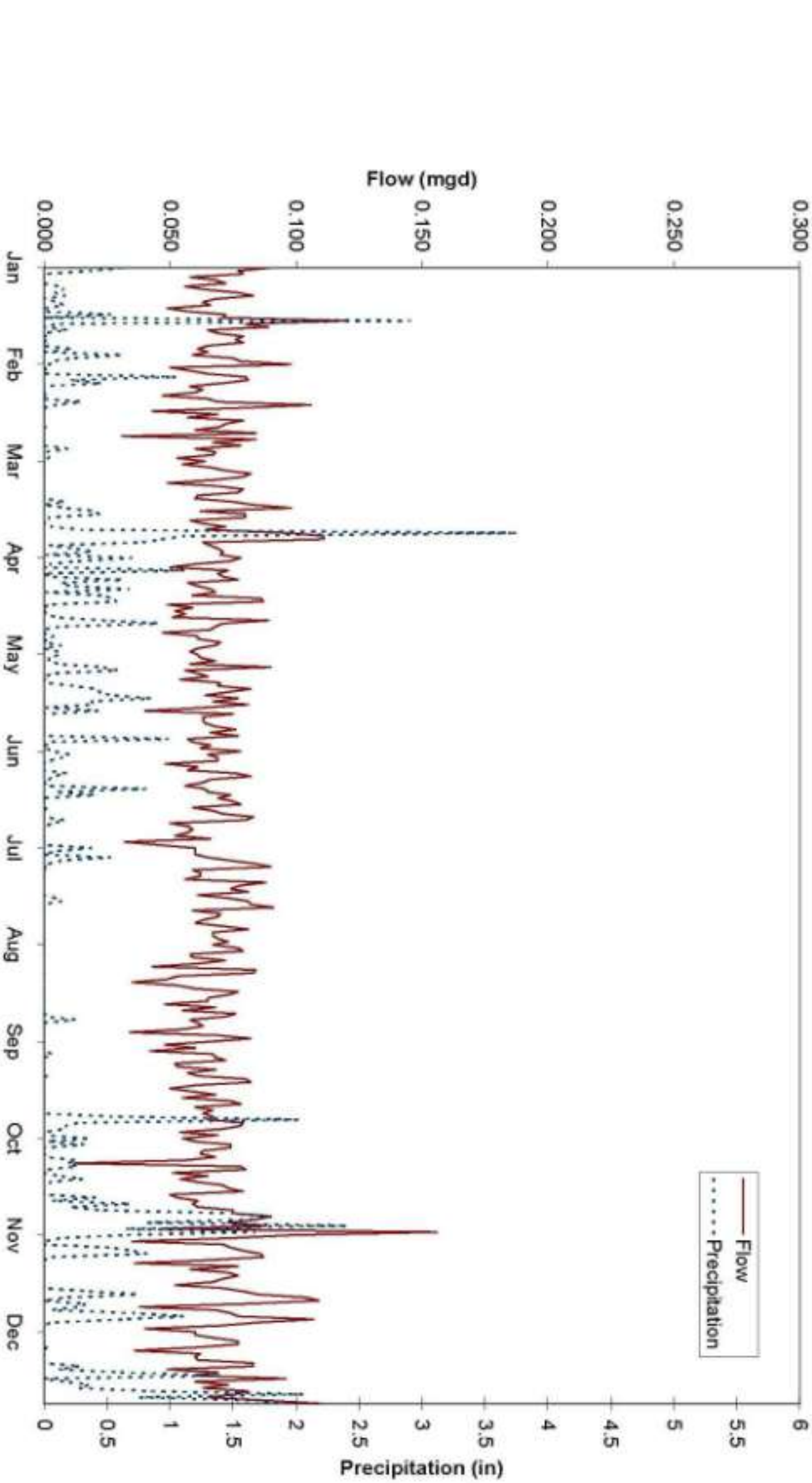

Figure 2-5
Sewage Flow Data - 2005
City of North Bonneville

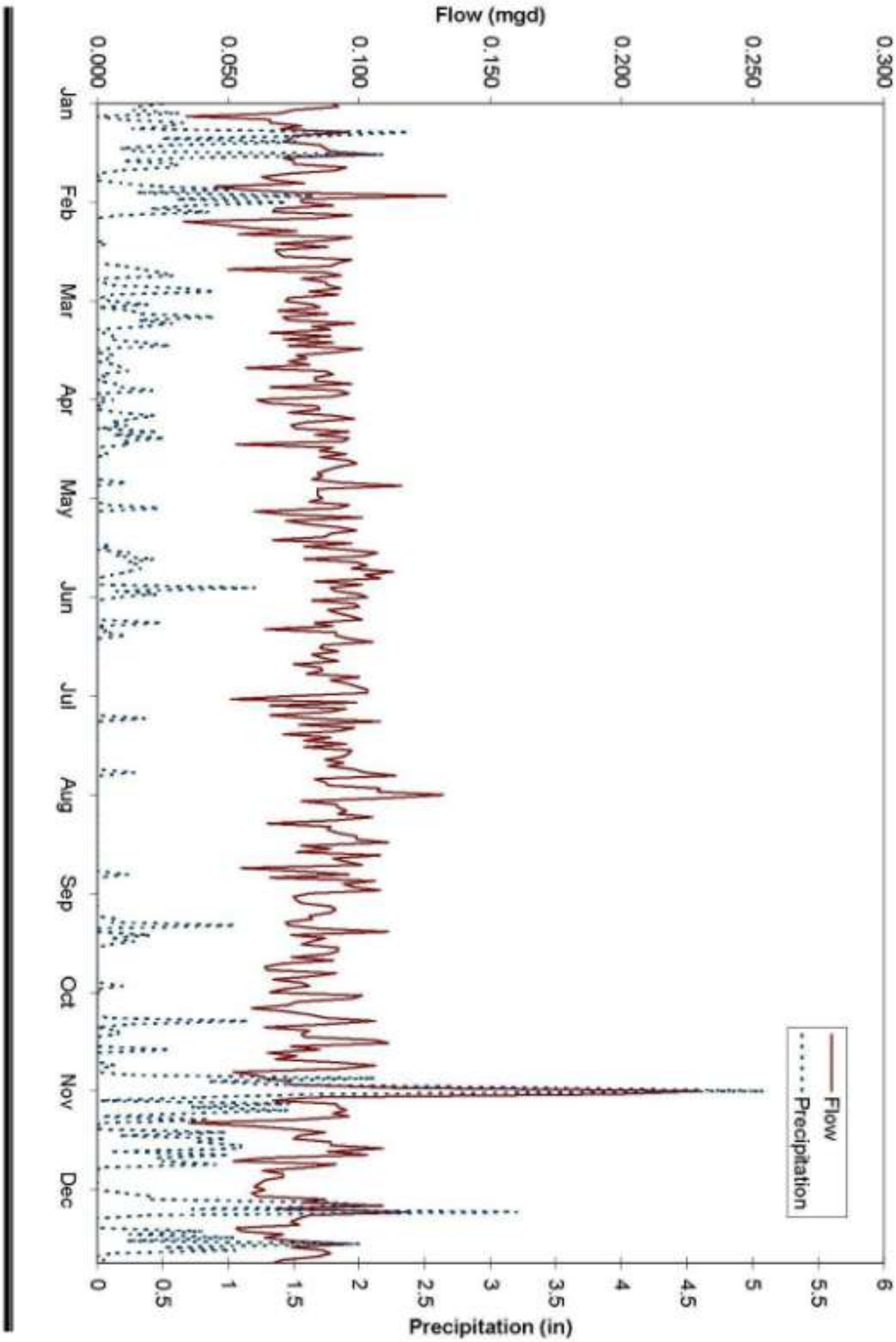

Figure 2-6
Sewage Flow Data - 2006
City of North Bonneville

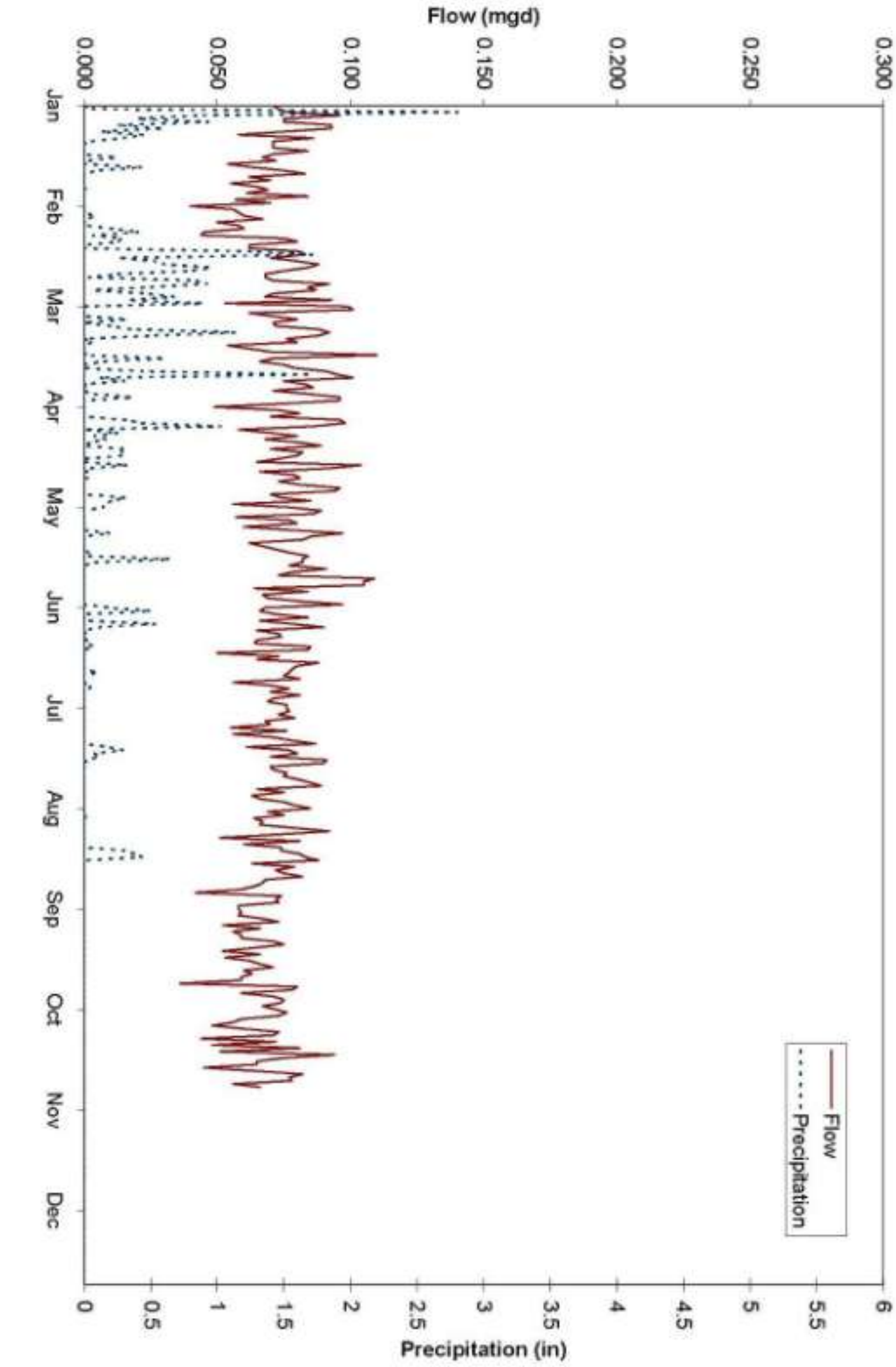

Figure 2-7
Sewage Flow Data - 2007
City of North Bonneville

3.0 SEWAGE PUMPING STATIONS

3.1 General

The City is located in the Columbia River valley in an area of level terrain and the wastewater must be pumped from low spots in the collection system to reach the wastewater treatment plant. Three sewage pumping stations were installed in 1976 with the original system and a fourth was installed in 1980 when sewers were extended to the northeastern corner of the service area. The pumping station locations are indicated on the sewer plan in Appendix C and summarized in Table 3-1. Further data on the Stations are presented in Appendix D.

Table 3-1. Sewage Pumping Stations

Pumping Station	Description of Service Area at Ultimate Development	Description of Station	Description of Force Main
No. 1- Pioneer Drive (1976)	Entire City area	Pumps (2) 540-gpm @ 31-ft. TDH	6-in. dia. Ductile iron pipe.
No. 2- Cascade Drive, West (1976)	78 SFR, 1 Commercial,	Pumps (2) 100-gpm @ 22-ft. TDH	4-in. dia. ductile iron pipe.
No. 3- Cascade Drive, East (1976)	PS-4 area & 41 SFR	Pumps (2) 200-gpm @ 27-ft. TDH	4-in. dia. ductile iron pipe.
No. 4- Bonneville Hot Springs Resort (1980)	21 SFR, 6 CR, & Bonneville Hot Springs Resort Development	Pumps (2), submersible. 100-gpm @ 27-ft. TDH	4-in. dia. ductile iron pipe.

SFR = Single family residences
CR = Commercial residential

3.2 Evaluation of Existing Pumping Stations

The sewage pumping stations have generally provided satisfactory service. The stations have remote alarm systems to indicate failure modes to the operators for high water levels and power failures. One stand-by portable generator is available for providing power to the pumping stations in the event of a power failure. Pumps have begun requiring significant repair and are being rebuilt over a three year period. All pump stations are expected to have been rebuilt by the end of 1998. Two pumping stations are expected to require more substantial improvements in the next year: Pump Station No. 1 that discharges directly to the WWTP and Pump Station No. 3 near the intersection of Cascade Drive and Bonneville Hot Springs Resort. These improvements have been made upgrading Pump Station Number 1 to a capacity of 540 gallons per minute and Pump Station Number 3 to a capacity of 200 gallons per minute in 1998.

Pump Station No. 1 contains two 540 gpm sewage pumps, with one intended to be used only as standby. The pump station has performed as intended since it has been upgraded. The capacity of a single pump is 0.77 mgd and the flow to the WWTP has been as high as 0.24 mgd over a 24-hour period on two occasions (both during high “inflow”). The flow to this pumping station is also affected on a short-term basis by the discharges from Pump Stations 2 and 3. The combined total flow from these stations is currently 300 gpm. As property north of Greenleaf Lake developes flows to Pump Stations 2 and 3 will increase requiring them to be upgraded further. The change in the flow to this station is anticipated to develop as shown in Table 3-2.

Table 3-2. Pump Station No. 1 Design Flows

Source of flow	Design Year						
	1997	**2002**	**2007**	**2012**	**2017**	**2023**	**2025**
Total tributary population	550	1,049	1,152	1,266	1,381	1,543	1,600
Net population served directly by PS No. 1	352	660	711	778	870	928	957
Est. Daily Flow, @ 100 gpcd	35,200	66,000	71,100	77,800	87,000	92,800	95,700
Est. Daily Flow, gpm	24	46	49	54	60	64	67
Est. Peak flow, at 4x daily flow (see footnote)	96	184	196	216	240	256	268
Flow, gpm, from PS No. 2	100	100	100	300	300	300	300
Flow, gpm, from PS No. 3	100	200	200	300	300	300	300
Est. Total flow, gpm	296	484	496	816	840	856	868
Actual Total flow	200 gpm	-	-	-	-		
Present capacity, gpm	200	540	540	900	900	900	900

Footnote: Ecology (1985) requires a minimum peak design flow of not less than 250 gpcd for design of main and trunk sewers when there is no other information available. The peak rate of flow now in the North Bonneville system is 4.3 times the average annual rate before any inflow/infiltration study and remediation is carried out. A preliminary design peaking factor of 4 is being used for the purposes of estimating in the General Sewer Plan.

It is recommended that the pumps in Pump Station No. 1 be replaced with pumps having a capacity in the range of approximately 900 gpm. The low speed for the “lead” pumps should be set for at least 200 gpm, however, to ensure solids are retained in suspension in the forcemain. It is also recommended that the pumps be installed with variable speed controls to permit efficient operation when the flow rate to the station is lower. This feature will reduce the rate of pumping to the wastewater treatment plant, reducing the loading on the grit chamber, comminutor, Parshall flume, and especially the equalization basin. Pump Stations 2, 3 and 4 should be upgraded to 300 gallons per minute ultimate capacity. This work should be completed as dictated by development of land north of Greenleaf Lake. Careful planning and coordination with the developers of property north of Greenleaf Lake will be required to assure that adequate pumping capacity does exist for the developments. It will be important for the City to work with the developers to assure that flows generated north of Greenleaf Lake are evenly split between Pump Station 2 and 3 to assure that there is not a capacity problem in the gravity collection system. Pump Station 1 will be upgraded to a capacity of 900 gallons per minute to have the capacity to convey the projected increased flows to pump stations 2, 3, and 4.The success or failure of measures to locate and eliminate inflow and infiltration will also significantly impact the actual flow that will come to this station..

The City was concerned that the new pumps recommended for Pump Station No. 1 would overload the treatment plant and “wash out” the clarifiers. If this pumping station were to operate continuously at 540 gpm and exceed the ability of the flow equalization basin, then the flow rate could “wash-out” the clarifier. Eventually, this is expected to occur and two clarifiers will be justified (but not yet, however). Until then, the equalization basin will equalize flows and reduce surges to prevent washing out the single clarifier. In addition, by increasing pump capacity and using variable speed drives, the pump will typically pump at rates less than 500 gpm.

Pump Station No. 3 is affected by the discharge of upstream Pump Station No. 4. Both stations have two pumps with the capacity of Pump Station No. 4 being 100 gpm per pump and the pumps in Pump Station No. 3 having a capacity of 200 gpm per pump. The completion in 1998 of the Bonneville Hot Springs Resort development increased the flow through Pump Station No. 4, but did not overload that station capacity. With future development of the property north of Greenleaf Lake flows will increase to Pump Station No. 4 to the point that the pump size will require upsizing. The increased time Pump Station No. 4 is in operation will result in overloading the capacity of Pump Station No. 3 by the incremental flow from the drainage area to Pump Station No. 3. The change in the flow is anticipated to develop as shown in Table 3-3.

Table 3-3. Pump Station No. 3 Design Flows

Source of flow	**Design Year**						
	1997	**2002**	**2007**	**2012**	**2017**	**2023**	**2025**
Old “west end” population	29	31	38	48	48	48	48
Parcel H population	0	0	19	28	26	28	28
Subtotal	29	31	57	76	74	76	76
Est. Daily Flow, @ 100 gpcd	2,900	3,100	5,700	7,600	7,400	7,600	7,600
Est. Daily Flow, gpm	2	2	4	5	5	5	5
Est. Peak flow, at 4x daily flow	8	8	16	20	20	20	20
Flow, gpm, from PS No. 4	100	100	100	100	100	300	300
Total flow, gpm	108	108	116	120	120	320	320
Present capacity, gpm	100	200	200	200	200	300	300

Pump Station Number 3 will require upgrading of the pumps to accommodate the future development of the property north of Greenleaf Lake.

Pump Stations No. 2 and 4 are not expected to need increased capacity except as might be required due to routine maintenance or unforeseen equipment failures until the property north of Greenleaf Lake is developed. As stated earlier, the owners of these parcels have proceeded with plans to develop their property and have contracted with engineers to develop designs for them. Approximately ½ of the property that had a zoning change from Timber Conservancy north of Greenleaf Lake will be directly served by Pump Station No. 4 which pumps flow to Pump Station No. 3. The other ½ of the flow will be directed to Pump Station No. 2. The estimates on flows to these stations are presented in Tables 3-4 and 3-5.

Table 3-4. Pump Station No. 2 Design Flows

Source of flow	Design Year						
	1997	**2002**	**2007**	**2012**	**2017**	**2023**	**2025**
Population served by PS. No. 2	151	217	228	256	279	318	334
Est. Daily Flow, @ 100 gpcd	15,100	21,700	22,800	25,600	27,900	31,800	33,400
Est. Daily Flow, gpm	10	15	16	18	20	22	23
Est. Peak flow, at 4x daily flow, gpm	40	60	64	72	80	88	92
Present capacity, gpm	100	100	100	100	100	100	100

Table 3-5. Pump Station No. 4 Design Flows

Source of flow	Design Year						
	1997	**2002**	**2007**	**2012**	**2017**	**2023**	**2025**
Population served by PS. No. 4 including Bonneville Hot Springs Resort development	18	261	276	276	276	276	276
Parcels North of Lake				40	80	120	134
Total Population	18	261	276	316	356	396	410
Est. Daily Flow, @ 100 gpcd	1,800	26,100	27,600	31,600	35,600	39,600	41,000
Est. Daily Flow, gpm	1.5	18	19	22	25	28	29
Est. Peak flow, at 4x daily flow, gpm	6	72	76	88	100	112	116
Present capacity, gpm	100	100	100	100	100	300	300

The timing of pump upgrades in all of the pump stations will be dependent on the timing and rate of the development within the City. It will be important to work with individual developers as they submit their plans for review to fully understand the impact of the proposed development on the pump stations. The growth projections reflected in this report will have to be reviewed with each proposed development and the timing of improvements to the pump stations considered with each review.

4.0 WASTEWATER TREATMENT PLANT

4.1 Overview

The Corps of Engineers constructed the WWTP in 1976. The WWTP provides secondary level treatment of the flow pumped directly to the WWTP site by Pumping Station No. 1. The site is separated from the nearest residential areas by over 400-ft. of undeveloped open and partially wooded areas. The site elevation is above the 100-year flood level and does not include any designated wetlands. There are no wells within 100-ft. of the site.

The influent flow to the WWTP passes through a "headworks" where grit is settled and large suspended solids are cut into small pieces before they might interfere with operation of downstream equipment. The flow rate is measured at this point with a Parshall flume. From the headworks, the wastewater flows to an aerated flow equalization basin (FEB) where the wastewater is mixed and kept "fresh." From the FEB, the wastewater is pumped at a more uniform rate through the remaining process units. Secondary treatment is provided in a single circular "package plant" with exterior concrete tank walls and central clarifier. Settled solids are returned to the aeration zone with an airlift pump. Waste activated sludge is also airlifted to an aerated holding tank. The clarified and treated wastewater flows from the clarifier to a flash mixing chamber where gaseous chlorine is added to control pathogenic (disease causing) organisms. After the mixing chamber, the wastewater flows to a chlorine contact tank before being discharged to the Columbia River. The plant does not have the piping to allow any bypassing of treatment processes.

The individual treatment process units are shown schematically in Figure 4-1 and listed in more detail in Appendix E. The hydraulic profile through the WWTP is shown in Figure 4-2 (US Army, 1976b).

4.2 Discharge Monitoring and Operations

The discharge limits for the treated effluent from the WWTP, established by the NPDES Permit, are summarized in Table 4-1 with a summary of operating results for the 1995-6 operating years.

In addition to the parameters listed in Table 4-1, the permit requires monitoring for dissolved oxygen, settleable solids, food/microorganism ratio, and sludge volume index. All of these parameters are within normal ranges (North Bonneville, 1996). There are no other discharges from the system than from the WWTP outfall sewer to the Columbia River. The WWTP received 1995, 1996, and 1997 Ecology commendations for exemplary effort in WWTP operations.

The existing permit expires in December 31, 2008. Discussions with Ecology has not indicated any significant changes to the permit limits but, at a minimum, the new permit will include the following language: "Total residual chlorine shall be maintained which is sufficient to attain the fecal coliform limits specified above. Chlorine concentrations in excess of that necessary to reliably achieve these limits shall be avoided." The new permit will also include the following language regarding ammonia removal: "Optimize plant operation for nitrification and monitor for compliance." (Meyer, 1998).

Figure 4-1

Figure 4-2

Table 4-1. Selected NPDES Discharge Limits and WWTP Loading

Parameter	Discharge Permit	1995-1996 Operating Year	Comments
Flow, monthly average Instantaneous peak flow	0.125 mgd 0.317 mgd	0.054 mgd 0.278 mgd (peak daily flow)	Satisfactory (no record of actual peak flow rate)
Biochemical oxygen demand, 5 day Influent, ppd avg. Effluent, monthly Weekly	 175 ppd 32 ppd, 30 mg/L, or less than 15% of influent concentration 47 ppd, 45 mg/L	 62 ppd 2 ppd 8.5 ppd	 Satisfactory Satisfactory Satisfactory
Total suspended solids Influent, ppd avg. Effluent, monthly Weekly	 200 ppd 32 ppd, 30 mg/L, or less than 15% of influent concentration 47 ppd, 45 mg/L	 68 ppd 2 ppd 6.5 ppd	 Satisfactory Satisfactory Satisfactory
Chlorine residual	Sufficient to attain Fecal coliform limits	-	Satisfactory
Fecal coliform bacteria Effluent, monthly Effluent, Weekly	 200/100 mL 400/100 mL	 6 31	 Satisfactory Satisfactory
PH	6.0 to 9.0	6.1-8.2	Satisfactory

The WWTP laboratory received “scope of accreditation” from Ecology to perform the laboratory analyses for BOD_5, Cl_2, DO, pH, TSS, and fecal coliforms (effective through July 10, 1998. Staff are certified as shown below:

Title	Name of Person	Certifications
Public Works Superintendent	Ray Hays	Group II Wastewater Operator
Wastewater Operator	Bryan Henrichsen	Group II Wastewater Operator
Maintenance	Steve Hichey	Pestified certified Group I Wastewater Operator

Individual responsibilities for system operation are as follows:

Activity	Name of person responsible
Normal operations	Ray Hays
Preventative maintenance	Ray Hays
Field engineering	Ray Hays
Operational and discharge water quality monitoring	Ray Hays
Troubleshooting	Ray Hays
Implementation of improvement program	Ray Hays
Budget formulation	Ray Hays and City Clerk
Response to complaints	Ray Hays
Public/press contact	Mayor and City Council
Billing	Deputy Clerk

All staff members are notified if there is an emergency. The Utility Manager is working with the County Emergency Management Coordinator to complete a contingency operational plan. Safety equipment has been listed and is available at the WWTP. No vulnerability analysis has been prepared.

4.3 Evaluation of Wastewater Treatment Plant Processes

The WWTP records were reviewed for the years 1995, 1996, and through September 30, 1997. The flow records were summarized in Figures 2-1 through 2-3 discussed earlier in relation to I/I. The records for organic loading, as BOD_5 and total suspended solids (TSS) are presented in Figures
4-3 and 4-4.

Figure 4-3

Figure 4-4

The individual unit processes in the WWTP perform specific functions. Dimensions and descriptions of unit processes are presented in more detail in Appendix E. The performance of each unit is summarized in Table 4-2 and discussed below:

4.3.1 Grit chamber. The grit chamber is a single longitudinal, controlled velocity gravity separator. The rate of flow to the unit is controlled by Pump Station No. 1, which now has a capacity of 540 GPM (0.77 mgd). The depth of flow is controlled by the Parshall flume and the pressure differential through the comminutor. Grit is removed weekly, usually on Tuesday, in the amount of 5 to 6 gallons. Grit is buried on site.

The grit chamber is operating satisfactorily at this time but consideration should be given to a second and larger unit when the WWTP is expanded. The present unit meets regulatory requirements, but is hydraulically overloaded when both pumps are operating and will be further overloaded when the proposed pumps are operating at full capacity during peak wet weather flow conditions. It is during such conditions that the maximum amount of grit is expected to be in the raw sewage. Failure to remove grit at this point results in its deposition in the FEB or aeration tanks later in the system.

4.3.2 Comminutor. This unit is operating satisfactorily at this time and has sufficient capacity for the proposed increased flow to the WWTP. This type of unit typically has a relatively high wear rate and becomes increasingly expensive to maintain with age. When the unit approaches the end of its service life and repair parts become unavailable, it should be replaced with a unit that provides better solids cutting.

4.3.3 Bypass bar screen. This unit is used only when the comminutor is out of service. It has adequate capacity for the present and proposed sewage flows.

4.3.4 Parshall flume/flow meter. This system consists of three separate parts: the fixed Parshall flume with a 3-inch throat, through which the sewage flows, the water level sensing float, and the indicator/totalizer/recorder device that converts the level signal into a number related to the rate of the sewage flow. The flume and float devices are satisfactory at this time, but the recorder is not functioning. This limits the value of WWTP records for analysis of I/I characteristics as there is no record of peak flow rates. The unit should be repaired or replaced. The system is checked annually for metering accuracy and the records of total flow are reliable (Hays, 1997). When Pump Station No. 1 was upgraded to 540 gallons per minute the capacity of the parshall flume was almost reached. For the flow to be increased at the plant this flume will have to be replaced with a flow element with greater capacity.

When the unit is replaced, it is recommended that a new sensor and indicator/totalizer/recorder device be installed with the readout chart inside the control building so that this information is available without going outside. A local indicator would be sufficient at the point of measurement. A signal from these units should be provided for flow proportioned sampling of the raw sewage flow at the headworks. Although the present sampling system at the FEB is adequate, the data is biased by the organic removal that appears to be taking place within this aerated tank.

Table 4-2

Table 4-2 cont'd

Table 4-2 cont'd

4.3.5 Flow equalization basin (FEB). This unit provides a necessary function, "smoothing" the alternating maximum and minimum flow rates resulting from the operation of Pump Station No. 1. The effectiveness of this operation is visible by comparing the alternating maximum and minimum flow rates shown earlier in Figure 2-4 with those in Figure 4-5 taken from the WWTP effluent flow meter. The alternating high and low into the FEB are pumped out at a more uniform rate to the rest of the WWTP. The air supplied to the FEB keeps the sewage "fresh" by flushing out odors in the raw sewage and adding oxygen to permit aerobic bacteria to begin breaking down and stabilizing the organic matter in the sewage. Air mixing also promotes separation of small sand and grit particles which can settle in the FEB and the downstream aeration tanks with less entrained putrescible organic material.

Review of the effluent flow charts in Figure 4-5 indicates the FEB is either too small to hold the flow long enough for this purpose or the pumps are too small for the present equalization system to even the peak and minimum flow rates. Either a larger tank, larger pumps, or a combination of the two could be installed, but larger pumps are recommended to avoid continued high flows to the downstream treatment process when two pumps operate simultaneously and when even a single pump comes on quickly or often. This factor will be aggravated if the decant from the aerobic digester is returned to the FEB as recommended in the discussion of that unit.

4.3.6 Flow equalization pumps. The pumps were accidentally flooded in 1997 when the dry well sump pump failed and the dry well filled with water (requiring one motor to be rebuilt). A second sump pump or a high water alarm should be installed to reduce the probability of a repeated failure. Review of the effluent flow charts from 1987 (see Figure 4-5), the last year during which these records were kept, indicates that the second pump is needed to keep the FEB dewatered during high flow periods.

The proposed system, including Pump Station No. 1 and the recycle from the aerobic sludge digester is expected to have a peak combined flow of approximately 500 gpm. It is proposed the new flow equalization pump system consist of three pumps with individual capacities of approximately 250 gpm and be furnished with manual variable speed control. Normally a single pump will maintain flow from the FEB through the WWTP, and the pump capacity can be manually set relative to the dry weather flow. Upon the water level in the FEB reaching a high point set by the operator, the second pump would come on at a similar rate to control the water level in the FEB. Should the level not fall with the second pump on, an alarm would be activated and the third pump would come on. Normally, it should not be necessary to use more than a single pump at partial capacity to maintain a satisfactory flow through the WWTP. During wet weather flows, the second pump would be occasionally required and its capacity could be set manually for a higher rate relative to the flow to the WWTP, or as much as a combined peak flow. The actual design rates for the pumps should be selected at the time of detailed design.

4.3.7 Aeration basins. The existing aeration basins are separated by a structural steel wall which permits operation of two parallel basins. It is occasionally necessary to dewater one basin when the other is still in service for inspection, repainting, cleaning, or repairing aeration

Figure 4-5

equipment. The wall between the basins, however, is reported to be unable to withstand the full hydrostatic pressure of the adjacent basin when the other basin is empty. Otherwise, however, the basins are adequate for present operation.

The air diffuser systems in the basins are of the coarse bubble-type. These type diffusers require more air than would be necessary if newer fine bubble diffusers were installed. The two positive displacement blowers have adequate capacity through 2017.

Ecology regulations require redundant aeration basins so the WWTP can operate with one out of service. This is not presently possible because the divider wall between the basins is not strong enough to allow one basin to be dewatered when the adjacent basin is full. Not being able to drain a basin limits the ability to maintain the diffusers or basin for inspection, cleaning, or repainting. It is proposed that the divider wall be inspected and if feasible reinforced. Otherwise, it will be necessary to provide an additional basin to provide redundancy.

Although the basins appear to be in generally satisfactory condition, they have been in service for over 20 years. Coated steel tanks have a limited service life, particularly if the protective paint is not maintained. It is estimated that this tanks will reach the end of that life in 10 years.

The tanks should be inspected for structural condition, an estimate made of the cost to reinforce the divider wall, and to determine if repainting is desirable to extend the life of the existing tanks. Appendix F contains some suggested approaches for reinforcing the aeration basin wall.

4.3.8 Clarifier. The existing clarifier is providing satisfactory service. Ecology regulations require that the WWTP have two clarifiers so the WWTP can operate with one out of service. This is not presently possible. It is proposed that an additional clarifier be provided for redundancy and to provide additional capacity during maximum day, wet weather flows.

4.3.9 Airlift pumps. The WWTP currently has four airlift pumps for sludge transfer. All are in different states of disrepair but are providing satisfactory service. An unfortunate characteristic of airlift pumps is that it is very difficult to estimate or measure flow because the flow varies with the difference in water levels between the suction basin and discharge point, rate of air flow, variable solids content of the sludge being pumped, and partial plugging. Further, the pumps operate at varying rates of flow over time with considerable splashing that interferes with downstream flow measurement.

The present units should continue to be used. However, centrifugal pumps should be provided for sludge return and wasting when WWTP improvements are implemented.

4.3.10 Chlorine feed system. This unit has adequate capacity at the present flow rates but will need to have a new metering tube installed to provide a maximum capacity of 12 mg/L (Ecology, 1985) at the future flow rates. Chlorine feed systems do not have a long service life and it is proposed the capacity of this unit be increased when it is replaced.

4.3.11 Chlorine flash mixing tank. This tank has adequate capacity at both the present and anticipated future flow rates.

4.3.12 Chlorine contact tank. This tank has adequate capacity at both the present and anticipated future flow rates.

4.3.13 Effluent flow meter. This system, like that at the Parshall flume, consists of three separate parts: the fixed 45 degree "V-notch" weir, the water level sensing float, and the indicator/totalizer/recorder device that converts the level signal into a number related to the rate of the sewage flow. The weir and float devices are satisfactory. The system is checked annually for metering accuracy, and the records of total flow are reliable (Hays, 1997). A signal from this unit also controls the rate chlorine is fed to the chlorine flash mixing tank. The chart drive has been required and rates-of-flow are again being recorded continuously.

There is no current record of hourly flow variations at this meter to evaluate the effectiveness of flow equalization through the WWTP. Records from the last year of record, 1987, were reviewed and two typical periods, one dry weather and one wet weather, are presented in Figure 4-5. Those records show the FEB was generally effective in eliminating the effects of on-off operation of pumps in Pump Station No. 1. The records also show FEB pump operation itself produces a hydraulic surge through the WWTP when the second FEB pump is activated. The records also show a significant impact on the rate of WWTP flow when the aerobic digester is decanted into an aeration basin. The flow produces an immediate increase in flow of MLSS solids into the clarifier, and a subsequent decrease in flow when excess MLSS solids are "wasted" from the aeration basins into the digester.

It is recommended that a new sensor and indicator/totalizer/recorder device be installed with the readout chart inside the control building so this information is available without going outside. A local indicator would be sufficient at the point of measurement. Signals from this unit should be provided for controlling the rate chlorine is fed to the chlorine flash mixing tank and for flow proportioned sampling of the treated sewage flow at the chlorine contact tank.

4.3.14 Outfall. The outfall pipe is a 12-inch gravity line approximately 1,400 feet long. The pipe extends into the Columbia River about 250 feet. The outfall pipe terminates at two 6-inch diffusers, 20 feet apart, which extend up from the 12-inch pipe and then turn downstream with 90-degree elbows. The centerline of the elbows was constructed 12 inches above the bottom of the river and the invert of the 12-inch pipe was set at elevation 18.0. The outfall pipe has adequate capacity to carry the present and anticipated flow from the WWTP. No changes are proposed.

In 2007 the City completed an professional outfall inspection. The results of the inspection are included in Appendix L. During the time of the inspection the outfall was 30 feet below water surface.

Based on current FIRM maps, the 100-year flood level for the Columbia River adjacent to North Bonneville is 37.5 feet above MSL. Low water level is difficult to determine, as the water level is dependent on how the water is being controlled at the Bonneville Dam, just upstream of the City. In addition the river profile is steep here and the water is very swift. Less water through the Dam would not necessarily translate into a lower water level.

4.3.15 Aerobic sludge digester. Once each week the air lift pumps transfer sludge from the aeration basin to the aerobic digester. The level in the aeration basin is reduced by two-feet. The sludge concentration varies from 1,500 to 3,000 mg/L MLSS. Space within the digester would be soon filled by this practice, but the staff first "decant" approximately an equal volume of clarified liquid from the top of the aerobic sludge digestion tank (after settling the sludge solids overnight). This practice affects other WWTP operations, however, as discussed earlier in relation to the effluent flow meter.

It is proposed that the decanted liquid from the digester be returned to the FEB to eliminate the flow surge through the aeration tanks/clarifier/effluent disinfection system. There is no reason to change the general practice of sludge wasting and decanting, however, until more information can be obtained on the sludge characteristics and control of vector attraction reduction. This is discussed below under ultimate sludge disposal.

4.3.16 Sludge pumps. There is only one sludge pump. A portable pump could be used to transfer the sludge from the digester to the beds if the existing pump were to fail.

4.3.17 Sludge drying beds. The existing drying beds are satisfactory. Sludge is pumped onto the beds twice annually and dried until it is convenient to remove it. No records are kept of the volumes, concentrations, or volatile content of sludge added to the beds or removed from them. Pumping is normally in the fall and spring. Sampling, analyses, and record keeping were started in November 1997.

4.3.18 Ultimate sludge disposal. Dried sludge is loaded from the beds and piled at the WWTP site. In the past, dried sludge has been used as a soil amendment for planting ornamental trees on City lands but this has not been done in 8-10 years.

Both Ecology and the USEPA regulate sludge disposal in Washington. Ecology is currently in the public comment period for review of new regulations applicable to "biosolids" to be beneficially applied to the land (Ecology, 1997). Municipal sewage sludge, as the material is presently classified, is regulated by the USEPA under 40 CFR Part 503 (USEPA, 1992). The principal intent of both the current federal and proposed state regulations is to control and reduce pathogens in sewage sludge. The regulations also require a reduction in the ability of the sewage sludge to attract vectors (insects and other living organisms that can transport sludge pathogens away from the site) by providing adequate volatile solids reduction. The mass of metals that may be land applied is also regulated.

The regulations apply different standards for sludge treatment depending on the ultimate disposal method chosen by the City. Class A requirements apply if the sewage sludge is to be sold or given away for application to a lawn or home garden. Class B requirements are less stringent and apply to sewage sludge to be applied to agricultural land, a forest, a public contact site, or a reclamation site. The City presently stores its sludge at the WWTP site or previously used it when planting on City lands. These practices, storage on the site for over one year, or use on agricultural land require a permit which the City does not have. It is recommended that the City begin the application process for biosolids disposal using one of the permitted processes. In that the WWTP more nearly complies with the Class B requirements, it is recommended that improvements be directed toward processes meeting those standards. The choice of processes should be made by the City after full consideration of the benefits of alternative uses and the costs of achieving standards required for those uses. A full discussion of all these factors is beyond the scope of a General Sewage Plan. The discussion in this report will be limited with those uses more readily achieved by the existing WWTP facilities and in agreement with previous City activities.

The existing WWTP uses aerobic digestion and air drying to reduce pathogens and control vector attraction for Class B purposes. These are essentially application to agricultural land where some pathogens may be permitted and where the danger of contact is controlled by restricted crop harvesting, animal grazing, and public access. This allows adequate time for environmental factors to reduce pathogens. The processes must meet the following standards, however, if they are to comply without requiring laboratory testing:

<u>Pathogen reduction requirements</u>:
Aerobic digestion- Sewage is agitated with air or oxygen to maintain aerobic conditions for a specific mean cell residence time at a specific temperature. Values for the mean cell residence time and temperature shall be between 40 days at 20 degrees C and 60 days at 15 degrees C. *Note: The City WWTP has a single aerobic digestion basin and, as sludge is wasted to this tank at least once weekly, there is less than 40 days detention time for at least some of the sludge at all times. A second aerobic digestion tank would have to be constructed to permit separate storage of the sludge during the final 40 days at 20 degrees C to meet the federal standards for pathogen reduction. As discussed below, air drying allows compliance without necessitating the additional tank.*

Air drying- Sewage sludge is dried on sand beds or on paved or unpaved basins. The sewage sludge dries for a minimum of 3 months. During 2 of the 3 months, the ambient average daily temperature is above 0 degrees C. *Note: The City WWTP practices meet this storage time and, as shown in Table 1-1, the ambient average daily temperature is above 0 degrees C throughout the year. Therefore, the WWTP is presumed to meet the standards for pathogen reduction for Class B purposes.*

<u>Vector attraction reduction requirements</u>:
Table 4-3 lists the 12 options to meet the Federal regulations:

Table 4-3. Summary of Requirements for Vector Attraction Reduction Under Part 503 (USEPA, 1992)

Requirement	What is Required	Most Appropriate for:
Option 1 **503.33(b)(1)**	At least 38% reduction involatile solids during sewage sludge treatment	Sewage sludge processed by: • Anaerobic biological treatment • Aerobic biological treatment • Chemical oxidation
Option 2 **503.33(b)(2)**	Less than 17% additional volatile solids loss during bench-scale anaerobic batch digestion of the sewage sludge for 40 additional days at 30°C to 37°C (86°F to 99°F)	Only for anaerobically digested sewage sludge that cannot meet the requirements of Option 1
Option 3 **503.33(b)(3)**	Less than 15% additional volatile solids reduction during bench-scale aerobic batch digestion for 30 additional days at 20°C (68°F)	Only for aerobically digested sewage sludge with 2% or less solids that cannot meet the requirements of Option 1—e.g., sewage sludges treated in extended aeration plants.

Table 4-3 (cont'd). Summary of requirements for Vector Attraction Reduction Under Part 503 (USEPA, 1992)

Requirement	What is Required	Most Appropriate for:
Option 4 **503.33(b)(4)**	Specific oxygen uptake rate at 20°C (68°F) is ≤1.5 mh oxygen/hr/g total sewage sludge solids	Sewage sludges from aerobic processes (should not be used for composted sludges)
Option 5 **503.33(b)(5)**	Aerobic treatment of the sewage sludge for at least 14 days at over 40°C (104°F) with an average temperature of over 45°C (113°F)	Composted sewage sludge (Options 3 and 4 are likely to be easier to meet for sludges from other aerobic processes)
Option 6 **503.33(b)(6)**	Addition of sufficient alkali to raise the pH to at least 12 at 25°C (77°F) and maintain a pH ≥12 for 2 hours and a pH ≥11.5 for 22 more hours	Alkali-treated sewage sludge (alkalies include lime, fly ash, kiln dust, and wood ash)
Option 7 **503.33(b)(7)**	Percent solids ≥75% prior to mixing with other materials	Sewage sludges treated by an aerobic or anaerobic process (i.e., sewage sludges that do not contain unstabilized solids generated in primary wastewater treatment)
Option 8 **503.33(b)(8)**	Percent solids ≥90% prior to mixing with other materials	Sewage sludges that contain unstabilized solids generated in primary wastewater treatment (e.g., any heat-dried sewage sludges)
Option 9 **503.33(b)(9)**	Sewage sludge is injected into soil so that no significant amount of sewage sludge is present on the land surface 1 hour after injection, except Class A sewage sludge which must be applied to or placed on the land surface within 8 hours after the pathogen reduction process.	Sewage sludge applied to the land or placed on a surface disposal site. Domestic septage applied to agricultural land, forest, or a reclamation site, or placed on a surface disposal site
Option 10 **503.33(b)(10)**	Sewage sludge is incorporated into the soil within 6 hours after application to land or placement on a surface disposal site, except Class A sewage sludge which must be applied to or placed on the land surface within 8 ours after the pathogen reduction process.	Sewage sludge applied to the land or placed on a surface disposal site. Domestic septage applied to agricultural land, forest, or a reclamation site, or placed on a surface disposal site
Option 11 **503.33 (b)(11)**	Sewage sludge placed on a surface disposal site must be covered with soil or other material at the end of each operating day.	Sewage sludge or domestic septage placed on a surface disposal site
Option 12 **503.33(b)(12)**	pH of domestic septage must be raised to ≥12 at 25°C (77°F) by alkali addition and maintained at ≥12 for 30 minutes without adding more alkali.	Domestic septage applied to agricultural land, a forest, or a reclamation site or placed on a surface disposal site

There is presently insufficient laboratory information to determine if Options No.1 or 4 is being met, and whether Option No. 3 is possible. Sludge testing will also be required to determine the metals content of the sludge solids to determine if land application of the biosolids would be permissible under the requirements of the proposed Ecology regulations.

4.3.19 WWTP utilities. The WWTP is presently served by the public electric, water, and telephone utilities. These systems area reported adequate to meet the demands of the WWTP. The water system is separated from the WWTP water distribution system by a reduced pressure zone backflow preventer.

The existing telemetry system is essentially unchanged since the WWTP and pumping stations were originally built. Several functions of the system are reported to be no longer reporting correctly and require operator visits to determine the true cause of alarm warnings. The system for the water and the wastewater utilities need to be reviewed in conjunction with those of the other City functions and the telemetry system upgraded to meet current requirements. A portion of the cost of the telemetry system upgrade is already budgeted, but an additional $4,000 will be required to complete that work.

4.3.20 Control building. The building provides space for the office, laboratory, vehicle and generator storage, and rest rooms. Air blowers, chlorination feed equipment, electric panels, and record storage are also in this building. The building is in generally satisfactory condition and requires only regular maintenance.

It is proposed that a new equipment storage building be constructed. The adjacent storage building needs to have its' roof replaced, gutters installed, and a new entry door. These improvements are scheduled for 1999. A building is also needed to store the backhoe, dump truck, and sand for road maintenance. That building is proposed for 2002.

4.3.21 Site. The roads and general site are satisfactory. Continued regular maintenance should adequately meet the needs for future requirements.

Based on the existing city limits, the current site is adequate for all future development. However, should additional land become necessary there is adequate adjacent land for expansion of the plant,.

4.3.22 Monitoring and Records

Monitoring of the sludge digester, decant, and drying beds for total and volatile solids should be consistently conducted. The following minimum monitoring is recommended to enhance process control, monitor performance, and meet regulatory requirements:

- Waste activated sludge, digester, and digester decant – twice monthly each.
- Sludge drying beds – influent and effluent twice annually.

The records of the system are stored at both City Hall and the WWTP. The records include the record drawings of the sewage collection system, pumping stations, and WWTP; operating manuals for the system; equipment inventories; maintenance schedules; and monthly operating records.

4.3.23 Potential for Effluent Reuse

According to RCW 90.46, it is highly desirable to develop facilities to provide reclaimed water to replace potable water in nonpotable applications, to supplement surface and groundwater supplies, and to assist in meeting future water requirements.

Potential reuse of wastewater effluent includes the following applications:

- Irrigation for agriculture.
- Industrial process water.
- Architectural features, landscape enhancement, or ornamental fountains.

- Recreational uses.
- Groundwater recharge.
- Fish and wildlife habitat creation or enhancement.

Any of these applications would require costly capital expenditures to provide piping from the treatment plant and appropriate equipment to implement the application. For the most part, effluent requirements would be more stringent, requiring additional, more costly, tertiary treatment at the treatment plant.

In addition, there would be comprehensive monitoring required to ensure that the health and safety of citizens is protected and to comply with Federal and State regulations. There would be increased monitoring requirements and accompanying laboratory analyses that would also increase annual costs.

Of these alternatives, a potential application in North Bonneville is the irrigation of the golf course. However, the costs for the pipeline to transport the effluent, additional tertiary treatment requirements, and additional monitoring make this option cost prohibitive. The golf course has a dedicated well, not connected to the City's potable water system.

Finally, the North Bonneville wastewater treatment plant is only a few hundred feet from the Columbia River. Any alternative water use would be hard pressed to select wastewater effluent in lieu of drawing water directly from the Columbia River, which would not have the accompanying treatment needs, regulatory restrictions, potential hazards, and perception issues to overcome.

Because of the limited opportunities and the high capital and operating costs, the reuse of effluent is impractical for the North Bonneville wastewater treatment plant.

5.0 ALTERNATIVES

To provide wastewater treatment capacity to accommodate future population growth, three alternatives were evaluated:

- Upgrade the existing WWTP.
- Provide a new parallel treatment train.
- Provide a new WWTP.

Redundancy will be provided for each of these alternatives to meet Ecology's requirements. The existing equalization basin and disinfection systems would be utilized with each of the proposed alternatives. It is assumed that needed improvements identified for the grit chamber, comminutor, Parshall flume, influent and effluent flow meters, equalization pumps, and chlorine feed system in the current WWTP will be done for all of the alternatives. If the second or third alternative is selected, the second clarifier and new blowers and aeration system would not be needed.

The soils at the site are adequate for any of the types of structures that are normally constructed at wastewater treatment plants. The soils are wet silty sands. In the winter of 1996 and summer of 1997, 10 foot deep test holes were dug at the treatment plant site to determine if there are groundwater issues. No groundwater was found. These results are similar to those found in the geotechnical explorations from the original plant construction in 1975. Those results further showed that the loose sand layer is 7 to 10 feet deep and is underlain by a medium dense to very dense sand layer.

Order of magnitude opinions of probable cost were prepared for each alternative. These costs were based upon selected vendor "budget" prices for equipment, gross unit costs for piping and structures, with installation costs being a percentage of the equipment. The costs should be conservative. Costs for electrical and site work, engineering, contingencies, and other costs were not included in this alternative comparison but will be added to the recommended alternative.

5.1 Alternative No. 1 - Upgrade the existing WWTP

For this alternative, the existing WWTP would be retained and selected unit processes upgraded as needed to provide for future growth and meet redundancy requirements. As discussed in previous sections, the existing WWTP would require improvements to include removal of the grit chamber and comminutor, upgrading the Parshall flume, influent and effluent flow meters, equalization pumps, and chlorine feed system and installation of a new fine wastewater screen. In addition, repairs being made to the telemetry system must be completed. These are common to all alternatives including this one.

The major shortcoming with the existing system (after the above improvements are implemented) is the lack of redundancy in the following units:

- Aeration Basin – Although there are two aeration basins currently, the structural ability of the dividing wall is reportedly insufficient to allow one tank to be dewatered while the adjacent tank is full of liquid. To allow needed redundancy, the dividing wall needs to be reinforced. Appendix F contains a preliminary review of reinforcing methods.
- Clarifier – A second clarifier is needed.

A schematic of the proposed system is shown in Figure 5-1. The design criteria for the upgraded existing WWTP is listed in Table 5-1

Table 5-1. Design Criteria for Alternative No. 1 - Upgrade the Existing WWTP To 2025

Feature	Average Capacity	Max Day Capacity	Approximate Dimensions
Flow	160,000 gpd	640,000 gpd	
Aeration Basin	10.5 ppd BOD/ 1000 cf	384 ppd BOD	Two 61,000 gallon basins
Clarifier	340 gpd/sf	1,550 gpd/sf	Existing at 18 ft diameter, New at 25 ft diameter
Return/Waste Sludge Pumps	100 gpm		Two pumps at rated capacity

Figure 5-1

The new secondary clarifier would be mounted in a circular concrete basin. The 2007 costs for the parallel train alternative are listed in Table 5-2.

Table 5-2. Opinion of Probable Construction Cost for Alternative No. 1 - Upgrading the Existing WWTP

Item	Description/Notes	2007 Cost	Year Needed
Work common to all alternatives -			
1 Headworks	Replace existing Headworks	$153,000	2008
Fine Screen			
Parshall Flume			
2 New equalization basin pumps	3@250 gpm New Sump pump and high-water alarm (work in existing confined space)	$15,000	2008
3 Chlorine feeder	Increased max capacity to 40 ppd	$4,000	2008
4 Telemetry	Upgrade to match improvements	$20,000	2008
Work specific to this alternative			
5 Improvements to existing aeration basin	Strengthen interior Partitions between aeration basin to permit individual tank dewatering, Replace Mechanical Systems, Rehabilitate Steel Surfaces, Repainting	$200,000	2008
6 Improvements to existing aeration basin	Replace diffusers	$65,000	2008
7 New parallel clarifier	Addition of 25 ft. dia. Clarifier in concrete tank	$400,000	2008
8 Electrical	Electrical Wiring and Panels	$100,000	2008
9 Yard piping and Improvements	Decant return piping to FEB Sludge recycle and waste pumping and piping, and Effluent piping Gravel and Asphalt Concrete	$7,000 $33,000 $30,000	2008 2008
	Total	**$1,027,000**	

One drawback to this alternative is that the age of the original steel structure that makes up the working mechanisms within the “Donut” treatment plant are in excess of 30 years old. It is intended that the systems will be rehabilitated during this process to extend the life of the existing units for at least 10 years. The systems have not been inspected since they have been put into service because of the lack of redundancy within the system. When the system is taken down during the upgrade process there is a possibility that what we find is that the steel components are so corroded that they may not be able to be rehabilitated and replacement will be necessary in a timeframe much shorter than is being anticipated in this analysis. If that does occur the cost of this alternative would increase by $250,000.

5.2 Alternative No. 2 - Provide a New Parallel Treatment Train

For this alternative, a parallel treatment train would be provided to closely match the existing aeration basin and clarifier treatment system. A similar package type WWTP is proposed. A schematic of the proposed system is shown in Figure 5-2. The design criteria for the parallel train is listed in Table 5-3.

Table 5-3. Design Criteria for Alternative No. 2 - New Parallel Treatment Train

Feature	Average Capacity	Max Day Capacity	Approximate Dimensions
Flow	160,000 gpd	640,000 gpd	
Aeration Basin	10.5 ppd BOD/ 1000 cf	384 ppd BOD	Two 61,000 gallon basins
Clarifier	340 gpd/sf	1,550 gpd/sf	25 ft diameter
Return/Waste Sludge Pumps	100 gpm		Two pumps at rated capacity

The parallel train would be mounted in a circular concrete basin. The 2006 costs for the parallel train alternative are listed in Table 5-4.

Figure 5-2

Table 5-4. Opinion of Probable Construction Cost for Alternative No. 2 - Parallel Treatment Train

Item	Description/Notes	1998 Cost	Year Needed
Work common to all alternatives -			
1 Headworks	New Headworks	$153,000	2008
Wastewater Screen			
Parshall Flume			
2 New equalization basin pumps	3@250 gpm New Sump pump High-water alarm (work in existing confined space)	$15,000	
3 Chlorine feeder	Increased max capacity to 40 ppd	$4,000	
4 Telemetry	Upgrade to match improvements	$20,000	
Work specific to this alternative -			
5 Improvement to existing Treatment Unit	Repainting, Structural Improvements, Replace Mechanical, Diffusers	$265,000	2008
6 Electrical	Electrical Wiring and Panels	$100,000	2008
7 New package WWTP	Additional 52-ft. diameter concrete tank with steel partitions between aeration, clarifier, and aerobic digestion sections, with clarifier mechanism, bridge, blowers, diffusers, and controls.	$870,000	2008
8 Yard piping and Improvements	Decant return piping to F.E.B. Sludge recycle and waste pumping, and Effluent piping. Gravel and Asphalt Concrete	$7,000 $33,000 $30,000	2008 2008 2008
	Total	**$1,496,000**	

The same issue that exists for alternate number 1, pertaining to the physical condition of the existing treatment unit, also exists with this alternative. If the condition of the existing treatment unit has deteriorated to a degree greater that would allow it to be rehabilitated then replacement of that that unit in a shorter time frame would be required. This would increase the cost of this alternative by $250,000.

5.3 Alternative No. 3 - Provide a New Sequencing Batch Reactor Treatment Plant

For this alternative, a new WWTP would be provided to replace the package treatment system. A sequencing batch reactor (SBR) type WWTP will be evaluated. An SBR will be evaluated for new construction because it will eliminate the need for separate tankage for aeration and clarifiers and thus reduce the volume of tanks for construction. The SBR will also reduce the size of pumps needed because no return sludge pumps are necessary. The SBR technology also inherently provides for flow equalization during the fill cycle. The SBR units discharge during only part of the cycle, however, and the chlorination facilities must be designed to accommodate that discharge pattern. Two parallel SBRs would be provided – each in a rectangular concrete tank with a common wall. The existing aeration basin and clarifier could be retained for sludge storage.

A schematic of the proposed system is shown in Figure 5-3. The design criteria for the new SBR WWTP is listed in Table 5-5.

Table 5-5. Design Criteria for Alternative No. 3 - New SBR WWTP

Feature	Average Capacity	Max Day Capacity	Approximate Dimensions
Flow	160,000 gpd	640,000 gpd	
Aeration Basin	166 ppd BOD	384 ppd BOD	Two parallel units
Clarifier			No separate tank needed, incorporated into SBR
Waste Sludge Pumps	25 gpm		Two pumps at rated capacity
Aerobic digester	3cf/capita or 34,000 gal		Two parallel units – reuse existing 61,000 gal aeration basin – install divider wall.

The SBRs would be mounted in one rectangular concrete basin, with a common wall separating the each SBR. Due to the batch process of an SBR and the high discharge flows from the decanter an exterior equalization basin will be necessary to allow metered flow to enter the chlorine contact basin. The metered flow would be accomplished by pumping from the equalization basin to the chlorination facilities. The existing blowers and controls will all be replaced to facilitate the operation of the treatment facility. The 2007 costs for the parallel train alternative are listed in Table 5-6.

Figure 5-3

Table 5-6. Opinion of Probable Cost for Alternative No. 3 – New SBR WWTP

Item	Description/Notes	1998 Cost	Year Needed
Work common to all alternatives -			
1Headworks	ınits	$153,000	2007
Wastewater Screen			
Parshall Flume			
2New equalization basin pumps	3-250 gpms New Sump pump High-water alarm (work in existing confined space)	$15,000	2007
3 Chlorine feeder	Increased max capacity to 40 ppd	$4,000	2007
4Telemetry	Upgrade to match improvements	$20,000	2007
Work specific to this alternative -			
5 Convert existing unit to Digester	Drain and clean existing tanks	$67,000	2007
6 New SBR facilities	Parallel concrete tanks for aeration/decanting/EQ basins with blowers, diffusers, decanting mechanisms, valves, controls, and piping.	$1,216,000	2007
7 Electrical	Electrical Wiring, Controls and Panels	$200,000	2007
8 Yard piping and Improvements	Decant piping Influent and effluent piping, and effluent pumping. Gravel and Asphalt Concrete	$10,000 $60,000 $30,000	2007 2007 2007
	Total	**$1,775,000**	

5.4 Alternative No. 4 - Provide a New Treatment Plant (Biolac)

For this alternative, a new WWTP would be provided to replace the package treatment system. A process manufactured by Parkson Corporation called "Biolac" will be evaluated. The "Biolac" system is an extended aeration activated sludge process that is well represented throughout the Pacific Northwest and has been shown to be easy to use, reliable and affordable. The "Biolac" system is a complete primary treatment system including aeration basins using fine bubble diffusion and rectangular hopper type clarifiers. The system is a "flow-through" system and will not require that the existing chlorination facilities be upgraded at this time. Two parallel aeration basin/clarifiers would be provided . The existing aeration basin and clarifier could be retained for sludge digestion and storage.

A schematic of the proposed system is shown in Figure 5-4. The design criteria for the new "Biolac" WWTP is listed in Table 5-6.

Table 5-7. Design Criteria for Alternative No. 4 - New "Biolac" WWTP

Feature	Average Capacity	Max Day Capacity	Approximate Dimensions
Flow	160,000 gpd	640,000 gpd	
Aeration Basin	166 ppd BOD	384 ppd BOD	Two parallel units
Clarifier			Two patallel units
Waste Sludge Pumps	25 gpm		Comes as part of the Biolac Package
Aerobic digester	3cf/capita or 34,000 gal		Two parallel units – reuse existing 61,000 gal aeration basin – install divider wall.

The "Bio-Lac" system would be located just south of the existing treatment building and headworks. It would be fully operational prior to taking any of the existing treatment facilities off ling. The aeration basins and clarifiers would be elevated 6 feet to allow gravity flow to enter the chlorine contact basin. The existing blowers and controls will all be replaced to facilitate the operation of the treatment facility. The 2006 costs for the parallel train alternative are listed in Table 5-8.

Figure 5-4

Table 5-8. Opinion of Probable Cost for Alternative No. 4 – New "Biolac" WWTP

Item	Description/Notes	1998 Cost	Year Needed
Work common to all alternatives -			
1 Headworks	New Headworks	$153,000	2002
Fine Wastewater Screen			
Manual Bar Screen By-pass			
Parshall Flume			
2 New equalization basin pumps	3-250 gpms New Sump pump High-water alarm (work in existing confined space)	not needed	2002
3 Chlorine feeder	Increased max capacity to 40 ppd	$4,000	2002
4 Telemetry	Upgrade to Match Improvements	$20,000	1998
Work specific to this alternative -			
5 Convert Existing Unit to Digester	Drain and clean existing tanks, remove all materials inside and install aeration/mixing	$67,000	2007
6 New Biolac Treatment System Including All Process Piping	Parallel aeration basins and clarifiers and aerobic sludge digestion, with blowers, diffusers, decanting mechanisms, valves, controls, and piping.	$725,000	2007
7 Electrical	Wiring and Panels	$185,000	2007
8 Yard and Site Improvements	Gravel and AC Surfacing, Fencing, Landscape and Erosion Control	$59,000	2007
	Total	**$1,213,000**	

5.5 Alternate Disenfection System

The City of North Bonneville has been experiencing delivery problems associated with Chlorine gas due to transportation problems. They have also experienced an extended period of increasing cost for the chlorine gas. They are also concerned with the inherent dangers associated with the presence of such a toxic substance at there place of work. There is also the possibility that regulations will change and the City will be required to eliminate the free chlorine from their wastewater effluent.

The existing tankage is of sufficient size to meet their long term flow projections but there is still the desire, due to the issues stated above, to look at alternate methods of disinfection of the treatment plant effluent.

The alternatives to upgrading the existing chlorine feed equipment would be to go to a chlorine solution system with the understanding that there may be a need to dechlorinate in the future or use ultraviolet light technology.

The cost comparison for the alternatives is shown below:

Table 5-9. Opinion of Probable Cost for Alternate Disinfection System

Alternative	Description/Notes	Cost
1 Chlorine feeder	Increased max capacity to 40 ppd	$4,000
2 Chlorine Solution	New Chlorine Solution Feed System	$30,000
3 Ultraviolet	Double Bank of Open Channel UV Systems	$90,000

The least cost alternative is obviously to stay with the existing system but it does not resolve any issues that the City has to contend with and does not address the escalating problems that the City will have to address in the future. Chlorine solution feed also leaves the problem of having to possibily address the free chlorine issue in the effluent in the future which will carry a cost similar to the cost to switch from chlorine gas to chlorine solution.

Ultraviolet light has been used in Oregon and Washington for more than 20 years now and is a well accepted and mature process alternative for disinfection of wastewater treatment effluent.

5.6 Comparison of Alternatives

Table 5-10 contains a comparison of each treatment alternative based upon descriptive criteria. Table 5-11 contains a comparison of each of the disinfection alternatives based on descriptive criteria. The costs shown are present worth (PW) costs assuming a 5 percent rate of return. Future costs were also inflated at 2.6 percent per year. Because Alternatives 1 and 2 are using an increasingly aging system, an expected increase in O&M costs was accounted for by increasing the O&M costs by 1 percent per year. Appendix G contains a tabulation of the PW analysis and shows the projected increases in O&M costs.

Table 5-10. Comparison of Treatment Alternatives

Criteria	1. Upgrade Existing WWTP	2. New Parallel Train	3. New WWTP-SBR	4. New WWTP-"Biolac"
PW Capital Cost ($1,000)	Least - $1,027	Moderate - $1,496	Greatest - $1,775	Least - $1,213
Operating Cost ($1,000)	Greatest – $1,046	Greatest - $1,046	Least - $953	Least - $953
PW Total ($1,000)	$2,051	$2,542	$2,728	$2,166
Operation	Low complexity – staff have greatest familiarity.	Moderate complexity because have duplicates of all units to operate and maintain.	After initial learning of new treatment process, have moderate complexity – SBRs are generally easy to operate.	Low complexity- extremely stable process and easy to operate
Construction	Low complexity and straight forward construction. However, continuing operation during reinforcing of aeration basin walls may be difficult.	Easy construction because existing WWTP can treat all flows during construction. However, will disrupt large area during construction.	Easy construction because existing WWTP can treat all flows during construction. However, will disrupt large area during construction.	Easiest of all systems to construct. Earth fill structures minimize the expensive construction materials required with other systems. However, will disrupt large area during construction.
Ability to meet Discharge Requirements	Have lowest aeration basin volume, so most susceptible to upset. However, based upon loading calculations, capacity is adequate.	Doubles aeration and clarification capacity so should greatly increase stability.	Integral equalization of SBR should maintain stable operation.	Extended aeration system and conservative clarifier design has documented history of very high quality effluent, and very low O&M requirements
Other Features	Simplest approach. Retains older "package" treatment technology.	Retains older "package" treatment technology.	Reduces need for recycle sludge pumps and need for separate clarifier tank. Does have lot of moving parts.	Will provide long-term treatment with little change in operating requirements.

Table 5-11. Comparison of Disinfection Alternatives

Criteria	1. Upgrade Existing Feed Equip	2. Change to Chlorine Solution Feed Equip	3. Change to Ultraviolet Light
PW Capital Cost	Least - $4,000	Moderate - $30,000	Greatest - $100,000
Operating Cost (annual)	Greatest – $15,000	Greatest - $15,000	Least - $7,000
PW Total (4%, 20 yrs))	$208,000	$234,000	$195,000
Operation	Low complexity – staff have greatest familiarity.	Moderate complexity, learn new process, deal with liquid solutions	Low complexity, minimal staff requirements to operate
Construction	Will need to provide for temporary facilities during construction	Will need to provide for temporary facilities during construction	Easy construction, no need for temporary facilities
Ability to meet Discharge Requirements	Will meet existing requirements, may need to address free chlorine issues in future	Will meet existing requirements, may need to address free chlorine issues in future	Will meet existing and future requirements
Safety Issues	Hazardous material issues remain	Still have corrosive liquid to work with.	Need to address the issue of ultraviolet light when working on equipment.

6.0 CONCLUSIONS AND RECOMMENDATIONS

This section summarizes the needed improvements to the collection and treatment system discussed in previous sections of this report. The costs for the recommended improvements are listed in Table 6-1. Costs for contingencies, engineering, administrative and legal fees have been included. These were estimated on a percentage basis. The contingency is meant to cover items such as normal construction dewatering, miscellaneous items, and unknown factors. Extensive dewatering during construction is not included.

Once detailed planning and design of these improvements is initiated, these costs should be refined. These generalized costs are useful for considering the alternatives and for budgeting. However, it must be recognized the estimating data and methods presented cannot in any way be used as a substitute for detailed cost estimating based upon design drawings, current labor costs, attitudes of proposed contractors regarding their need for work at the time of construction bidding, availability of materials, climate and seasonal factors, local site conditions, and other variables which may affect actual construction costs.

6.1 Wastewater Collection System

The City has been pursuing an aggressive program to minimize infiltration and inflow (I/I) into the collection system. The sewers were experiencing "excessive" inflow as compared with USEPA and Ecology criteria. Based upon one period, June 1997, the sewers may also be experiencing "excessive " infiltration by those same criteria. The major source of the flow may be from defective joints in the manhole barrels, based on the 1985 television inspection. Work performed to cut down on the I/I has been successful in reducing the amount of I/I entering the system as is evident from the comparison of plant flow data of two extreme rainfall events defined in Section 2.2 of this document.

The City should continue the I/I reduction program and periodically re-inspect the manholes during high inflow and high infiltration periods to determine the overall condition of the system and identify which manholes need to be sealed. Those manholes most in need of sealing should be sealed. Sewage flow records should then be analyzed to determine if further work is needed and provide a "red flag" to indicate when a more intensive effort should be scheduled. The estimated annual cost for this continuing work, to include sealing approximately 5 manholes per year and analyzing the results, is $10,000.

No new sanitary sewers are expected to be necessary except as may be constructed by developers for service to their specific projects. Those sewers should be designed and constructed by the developers in accordance with Ecology criteria and standards. If the sewers would serve the long-term interests of the City, the City should take ownership and provide the future maintenance of those sewers.

6.2 Pump Stations

In 1998, Pump Station No. 3 had its pumps replaced with the pumps from Pump Station No. 2 having capacities of approximately 200 gpm. The costs for these improvements was $16,000.

Also during 1998, Pump Station No. 1 had its pumps replaced with pumps having capacities of approximately 540 gpm. Controls for these pumps included automatic variable speed control to minimize the impact of the pump discharge on the WWTP headworks and FEB. The size of the new pumps did impact the existing system and additional detailed pre-design engineering is warranted. The costs for these improvements was $50,000. The total cost for pumping station improvements including contingencies, engineering, legal, administration, and taxes was $102,000.

6.3 Wastewater Treatment Plant

The short-term improvements identified in the 1998 General Sewer Plan have been completed. They included upgrading pump stations number 1 and 3, the chlorine feed system, telemetry system, and constructing a new storage building.

Increased testing of sludge should be conducted for total and volatile solids of the waste activated sludge, in the digester, and the digester decant twice a month and the sludge into and out-of the drying beds twice a year. The cost for increased testing is approximately $1,000 per year, based upon an additional 76 total and volatile solids tests per year.

Although any of the treatment alternatives would meet the City's needs, Alternative 4 has the least short-term cost impact. Based upon the evaluation conducted in Section 5, Alternative 4, constructing a "Biolac" secondary treatment facility with Ultraviolet light disinfection presently appears to be the most appropriate option for North Bonneville. Planning should begin toward upgrading the existing WWTP. The costs and timing for upgrading the WWTP are presented in Table 6-1.

In 2007 improvements to the headworks, existing treatment unit, a new secondary treatment facility, and associated yard piping totals $1,228,000.

If the City constructs the new treatment facility as called for it will serve the City for the design life of the facilities which will be in excess of 20 years.

6.4 Summary of Costs

A summary of the opinion of probable costs for construction of the recommended improvements are presented in Table 6-1, along with the probable changes to system operating costs. It is assumed that no additional staff will be necessary to operate the improved WWTP. Additional operating costs are for power only.

These costs are presented in 2008 dollars and include allowances for project construction contingencies, engineering design services, construction administration services, as well as owner inspection and owner administrative expenses. Sales taxes on work performed in City of North Bonneville are also shown at the current rate of 7.5 percent. It is not expected that there will be any legal fees associated with the project.

Table 6-1. Opinion of Probable Capital Costs and Increase in Annual Operating Costs in 2007 Dollars

Year	Activity	Capital Costs			Increase in annual operating costs
		Collection System	Pump Stations	WWTP	
2009	New Headworks with Wastewater Screen and Parshall Flume Convert Existing Treatment Unit to Digester .Construct New Secondary Treatment Facility (Biolac) Including Process and Yard Piping Construct New UV Disinfection system Yard and Site Improvements .Electrical and Telemetry Improvements Associated with Plant Up-Grade Construction Construction Contingency (15%) Taxes (7.5%) Total Construction Engineering Design, Const Admin On Site Inspection, Owner Admin			$ 153,000 $ 67,000 $725,000 $100,000 $59,000 $209,000 $ 1,313,000 $197,000 $113,000 $1,623,000 $225,000 $50,000	
Total Costs				**$1,875,000**	**+$5,000**

6.5 Rates

The City of North Bonneville sanitary sewer rates are presently at $35/month per Equivalent Residential Unit. This amount is sufficient covers the costs of operating the sanitary sewer facilities. The Sewer Utility has no outstanding debt.

The City is investigating different funding alternatives for the improvements to the wastewater treatment facility. Alternatives are Rural Development (RD) and the State of Washington Public Works Trust Fund for financing of this project. The two alternatives have significantly different conditions associated with the funding possibilities. RD has an interest rate of 4.75% and can fund a loan with a repayment period of up to 40 years. There is also the possibility of grant money being available through RD. The Public Works Trust Fund has 20 year loans at an interest rate of 1%. Due to the uncertainty of the funding application and the conditions of any approved funding alternative a number of possible funding alternatives were analyzed to determine the impact on the City of North Bonneville sanitary sewer user fees . That different analysis are included in Appendix I of this Report. The analysis assumed that there would be no grant funds available. One alternative was for a loan from RD in the amount of $1,500,000. The interest rate was set at 4.75% (the interest rate for RD loans last year). Three different loan periods were analyzed which included 20, 30 and 40 year loan periods. Another analysis was performed on a possible Public Works Trust Fund loan. The loan amount was $1,500,000, the interest rate was set at 1% and the loan period was 20 years. The results of the different analysis are included in Appendix G.

7.0 IMPLEMENTATION

The City should continue its program to locate manholes leaking groundwater into the system during wet weather condition and seal those manholes where rehabilitation is cost effective. The work would also include monitoring sewage flow conditions in the separate sections of the system to identify sections with apparently excessive flows. As the City has already done with the inspection of the manholes they should seal the leaking joints where rehabilitation is cost effective. A typical approach to a full analysis and evaluation of sewers for inflow and infiltration is presented in Figure 7-1. The City should consider implementing the early portion of this approach with its own staff, using consulting engineering assistance in an advisory capacity. This phase of study is important and should be initiated now, even though the manhole program is still in progress.

The WWTP staff should also continue the sampling and analysis on the sludge system to determine whether the existing system complies with the current Federal and proposed Ecology sludge regulations.

The next major step should be to initiate design for modifications to the wastewater treatment plant. The City has acknowledged the need to upgrade or replace the existing facility. The age of the facility is showing and significant repairs as well as mechanical equipment replacement is needed in the short term just to keep the existing facility operational. Upon approval of the General Sewer Plan Update preliminary and final design should proceed for the plant improvements.

With the plant improvements the biological and hydraulic capacity of the secondary wastewater treatment system will be of adequate size to serve the projected population for the City for the design period of 20 years at a minimum. The life of the newly constructed facilities can be expected to be in excess of 40 years.

The improvements recommended in this plan were reviewed under the requirements of the State Environmental Policy Act (SEPA). A determination of nonsignificance was prepared. Appendix H contains the SEPA documentation.

Figure 7-1

8.0 REFERENCES

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APPENDIX A

NPDES Discharge Permit

APPENDIX B

Population Projections

APPENDIX C

Sewer and Water System Maps

APPENDIX D

Description of Pumping Stations.

Table D-1. Description of Pumping Stations

Pumping Station	Description of Service Area, Ultimate Development	Description of Station	Description of Force Main	Records Available
P.S. No. 1- Pioneer Drive (1976) Pump 1 rebuilt in 1996, pump 2 in 1997.Pumps replaceed in 1998	Entire City area	Pumps (2) 540-gpm @ 40-ft. TDH 20-HP, Hydromatic 40 MPC 1000 Wet well dimensions are: 5-ft. dia. by 16–ft. deep. 5.0-ft. alarm on. 4.0-ft. lag pump on. 3.0-ft. lead pump on. 1.5-ft. pumps off.	6-in. dia. Ductile iron pipe, 800-ft. to WWTP Min. WS El. at PS = 17.0-ft. WS El. at out= 44.9-ft	Design calculations. Bid drawings and specifications. Operations manual with pump curves.
P.S. No. 2- Cascade Drive, West (1976) Pump 1 rebuilt in 1997.	78 SFR, 1 Commercial,	Pumps (2) 100-gpm @ 22-ft. TDH 2-HP Wet well dimensions are: 6-ft. dia. by 8.0–ft. deep. 5.5-ft. alarm on. 4.5-ft. lag pump on. 3.5-ft. lead pump on. 1.5-ft. pumps off.	4-in. dia. ductile iron pipe, 560-ft. to WWTP Min. WS El. at PS = 17.0-ft. WS El. at outlet = 33.75-ft. Air release valve on force main.	Design calculations. Bid drawings and specifications. Operations manual with pump curves.
P.S. No. 3- Cascade Drive, East (1976) Pump 1 rebuilt in 1997.	PS-4 area & 41 SFR	Pumps (2) 200-gpm @ 31-ft. TDH 5-HP Wet well dimensions are: 4-ft. dia. By 9.5–ft. deep. 4.5-ft. alarm on. 3.5-ft. lag pump on. 3.0-ft. lead pump on 1.5-ft. pumps off.	4-in. dia. ductile iron pipe, 360-ft. to WWTP Min. WS El. at PS = 34.0-ft. WS El. at outlet = 54.30-ft.	Design calculations. Bid drawings and specifications. Operations manual with pump curves.
P.S. No. 4- Bonneville Hot Springs Resort (1980)	21 SFR, 6 CR, & Bonneville Hot Springs Resort Development	Pumps (2), submersible. 100-gpm @ 27-ft. TDH 2-HP Wet well dimensions are: 5-ft. dia. By 15–ft. deep. 3.9-ft. alarm on. 3.4-ft. lag pump on. 3.2-ft. lag pumps off. 3.1-ft. lead pump on. 2.7-ft. lead pump off.	4-in. dia. ductile iron pipe, 260-ft. to WWTP Min. WS El. at PS = 32.0-ft. WS El. at outlet = 47.50-ft.	Bid drawings and specifications.
SFR = Single family residences CR = Commercial residential				

APPENDIX E

Wastewater Treatment Plant Unit Processes.

Table E-1. Treatment Plant Unit Processes

Unit	Description
Grit chamber	Single 14.2-ft. long by 1.5-ft wide longitudinal, controlled velocity gravity separator. Manually cleaned. Water surface elevation is controlled by downstream Parshall flume. Flow velocity 0.7 fps at design avg. flow
Comminutor,	Worthington Model 12-C-4 comminutor. Min. upstream channel velocity of 0.6 fps at design avg. flow
Bypass bar screen	Fabricated bar screen, ¼-inch bars at 1.5-in. on center, with 45-degree slope. Min. velocity through bars 0.02 fps.
Influent flow monitoring	3-in. fiber-reinforced polyester Parshall flume liner with Stevens Model 61R float-operated flow recorder. Allowable flow range = 0-0.7 mgd.
Flow equalization basin	16-ft. square by 15-ft. side water depth (SWD) concrete tank. Volume = 28,723 gallons. Mixed with diffused air, but air rate is not measured. Detention time at mean depth 3.2 hr.
Flow equalization pumps	Two (2) vertical, pedestal-mounted, centrifugal pumps in subsurface wet well structure. Capacity, ea. = 87 gpm at 7.5-ft. TDH 0.5 HP 96 % time in operation at design avg. flow
Aeration basin	Two (2) parallel compartments in a single 45-ft. diameter “package plant” structure using “extended aeration” activated sludge process. The exterior tank wall is reinforced concrete and interior walls are steel. Volume = 61,000 gallons, each. SWD = 15.0-ft. Nominal detention time 24 hr Organic loading of 10.5 ppd/1000cf at design avg. flow Aeration equipment is submerged, coarse bubble diffused air. Air supply is from a 20 HP, 2 speed, Sutorbilt 7MVB rotary positive displacement blower, rated at 260/488 cfm free air at 7 psig, with a standby unit. Coarse bubble air diffusers
Clarifier	Single circular 18.25-ft. diameter , 13-ft. SWD clarifier with 4.1-ft. dia. center feed, peripheral weir overflow, scraper and skimmer solids collection and parallel airlift pumps for solids return to the aeration basins. Overflow rate 460 gpd/sf at design avg. flow Overflow rate at Max. daily flow 1212 gpd/sf at design avg. flow Nominal detention time 5.1 hrs at design avg. flow. Weir loading. 2350 gpd/lf at design avg. flow Solids loading rate 9.6 ppd/sf at design avg. flow
Return sludge pump	Two 4 inch airlift pumpsm 90 gpm at 15 cfm air
Effluent chlorination	Chlorine supply is 150-lb. Gas cylinders. Chlorine feeder is Fischer & Porter Chloralert, 0-3 ppd gas feeder. Max. rate at peak hydraulic flow is 1.1 mg/L. Chlorine feed control is controlled by the effluent flow meter and a manual rate setting.

Flash Mixing	Mixing is provided in a 4-ft. square by 4-ft. SWD concrete tank with a ½ HP Eastern RG portable mixer.. Detention time 5.7 min at design avg flow. Mixer speed is 400 rpm.
Chlorine Contact Tank	Single, baffled, 18.7-ft. by 13-ft. by 8.0-ft. SWD reinforced concrete tank Nominal detention time is 2.9 hr at design avg flow. Nominal detention time at peak daily flow is 1.1 hr. Approx. length to width ratio = 18:1
Effluent flow monitoring	Single, 45 V-notch weir with a Stevens Model 61R float-operated flow recorder. Allowable flow range = 0 - 0.35 mgd Automatic sampling equipment
Outfall	1,395-ft. of 12-in. diameter "boltless" ductile iron flexible pipe extending 275-ft. into the Columbia River with an approximate depth of 2.8-ft. at average water surface elevation. High water surface El = 54.25 Low water surface El = 18.25
Sludge holding tank	Single aerated compartment in circular "package plant" structure. Volume = 27,900 gallons Storage capacity is 3.1 cf/capita Aeration system is two coarse bubble diffuser assemblies. Air supplied from common blower serving all other aeration uses. Air supply = 20 cfm/1,000 c.f. Two adjustable airlift pumps @ 35 gpm each provided for decanting supernatant.
Sludge removal pump	Single, vertical, pedestal-mounted, centrifugal pump in subsurface dry well. Capacity = 89 gpm at 11.5–ft. TDH ½ HP
Sludge drying beds	Six (6) parallel 24.8-ft. by 14.7-ft. open, asphalt-paved sand drying beds. Area = 2,187 S.F. 1.8 sf/capita Subsurface flow is pumped to the influent flow equalization basin. Single, submersible, non-clog pump in 4-ft. diameter, 15.0-ft. deep wet well, with 4.5-ft. operating depth. Capacity = 100 gpm at 11.5–ft. TDH ¾ HP
Standby power	WWTP - 60 KW generator set. Sewers- Portable 30 KW generator set.

APPENDIX F

Proposed Reinforcing Methods for the Aeration Basin Walls.

Proposed Reinforcing Methods for the Aeration Basin Walls.

1. The following discussion provides alternatives on how the aeration tank could be inspected and repaired without being removed from the process and dewatered.

   Planning for the reinforcement should begin with a structural analysis based on plant drawings and such field measurements as can be obtained without full dewatering. Dewatering for even a short period will require substantial advance planning, but it would be possible to dewater the tank during a summer, dry weather period when the plant flow is approximately 50,000 gpd. The existing tanks hold approximately 122,000 gallons which, if pumped out over a 5-day period, would result in an additional flow of approximately 24,000 gpd. Approximately 24,000 gallons of aeration can be provided in the equalization tank, but an additional 40,000 to 50,000 gallons of aeration would be needed to provide treatment equivalent to the present system. Three alternative approaches are presented:

   A. A temporary aertion tank could be provided and aerated with the existing blowers and temporary diffusers. The flow would be discharged to the temporary tank from the equalization tank and then flow to the existing clarifier for separation. Sludge would be returned to the equalization tank. The aeration tank could then be inspected. The system could be returned to normal operation while design documents were prepared, the work plan reviewed by the regulatory agencies, the work bid, contracts awarded, and the tanks again dewatered for the repairs. The costs for this work would include the temporary tank and yard piping. There would likely be no salvage value for the tank, but it could be retained for future additional sludge storage.

   B. Instead of constructing a temporary aeration tank, the structure for the clarifier proposed for 2007 could be constructed early and used for the temporary aeration tank. The volume of that tank, together with that of the equalization tank, would total approximately 52,000 (vs. 61,000 gallons for a cell in the existing tanks). Floating aerators could be used for aeration. This alternative would require either that the clarifier struture be built early or that the repairs to the aeration tanks be deferred.

   C. Instead of dewatering the aeration tanks, divers experienced in structural work in highly turbid water conditions would perform the inspection. The only interruption to plant operations would be to close off the cell being inspected. The level would be lowered so far as possible and blower operation ceased to the other aerated cells for the duration of the dive to reduce underwater noise levels. The turbidity of a cell could also be reduced somewhat by diluting the mixed liquor suspended solids in that cell by diverting incoming sewage to the other cell several weeks in advance and returning plant effluent to the cell to be inspected.

A decision between these alternatives is beyond the scope of this plan and should be made only after a structural analysis has been completed on the basis of the plans and more readily available information.

APPENDIX G

Summary of Sewer Capital Improvements Proposed Financing Approach.

Summary of Capital Improvements
Proposed Financing

North Bonneville has a specifically dedicated sewer fund. To assist in paying for the proposed improvements, The City is investigating the possibility of acquiring funding for the project from the State of Washington Public Works Trust Fund. The funding will be in the form of a loan. The loan application is proposed to be $1,500,000. The interest rate of the loan is 1% APR for 20 years.

The City is also investigating the possibility of acquiring financing from Rural Development (RD). This program does have the possibility of providing grant funds in the funding package. The interest rate for borrowed funds has been 4.75% in the recent past. Loans can be funded for up to 40 years in this program.

Different methods of financing the remainder of the improvements costs have been investigated and the alternatives include:

- Working with the Skamania County Economic Development Council to acquire possible grants.
- Initiate an Inter-Fund loan from the City General funds and utilizing sewer connection fees to repay the loan. Increase the Sewer Connection Fee from $2,000 to $4,500 per single residential unit or equivalent. The fee would increase proportionately based on water service connection size for commercial and industrial users.

The attached table shows a cash flow forecast based on conservative criteria associated with the growth within the City and the growth of costs associated with operating and maintaining the sanitary sewer system.

The main assumptions associated with the development of the cash flow forecast were:

1) Connections Will Increase at 3%/Year
2) Operating and Maintenance Expenses Will Increase at 3%/year
3) Monthly Sewer Rates Are Adjusted to Assure the City Operating Costs are Met
4) The Sewer Connection Fee has increased from $2,000 to $4,500 for a customer with a ¾" water service. Fee increase for services larger water services will increase at a higher rate based on meter size but that increase was not included into the spreadsheet (also makes the estimate conservative)
5) Interest Rate is set at 4.75% for RD and 1% for Public Works Trust Fund

The historical growth rate in the City has been greater than 4% and there is very strong evidence that the growth in the City will be higher than has been experienced in the past. The proposed improvements to the wastewater treatment plant will be able to serve the projected population for the City for the 20 years time frame evaluated without additional upgrades at the wastewater treatment plant.

The spread sheets shows that the City will need to adjust the user fee rates to meet the operating costs and loan costs. The amount of fee increase will depend on the condition and amount of the loan.

APPENDIX H

SEPA Documentation.

APPENDIX I

2006-2007 Inflow/Infiltration Evaluation.

APPENDIX J

2007 Outfall Inspection Report