City of Molalla
Wastewater Facilities Plan

April 2000

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City of Molalla
WASTEWATER FACILITIES PLAN

April 2000

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EXECUTIVE SUMMARY

PURPOSE AND SCOPE

The City of Molalla Wastewater Facilities Plan was prepared to evaluate the City's wastewater collection system and treatment facilities. The study scope includes evaluation of current and future wastewater facility needs, infiltration and inflow (I/I) monitoring, a capital improvement program, and a funding plan.

SERVICE AREA AND POPULATION

The City of Molalla wastewater system serves much of the developed area within the City's urban growth boundary (UGB). The existing UGB is not likely to have enough capacity for the 20-year planning period. As a result, this plan addresses expected growth outside the existing UGB.

The City's 1998 population was approximately 5,395. The City has experienced explosive growth in excess of 9 percent annual growth during the past three years. Growth is expected to continue at a rate higher than the county average. Molalla's growth potential is related in part to its proximity to the Portland metropolitan area. Based on the population projection assumptions used in this plan, the projected 10-year (2009) population is 9,950, or 1.8 times the existing population, and the projected 20-year (2019) population is 13,370, or 2.5 times the existing population.

WASTEWATER FLOWS AND LOADS

Wastewater unit flows and loads were estimated based on wastewater treatment plant flow records from January 1996 through 1998. Based on these records and the expected growth in the City, the wastewater flows and loads to the plant were projected. These projections are shown in Table ES-1 for existing (1999), 10-year-planning (2009), and 20-year-planning (2019) conditions. Flows are shown in million gallons per day (mgd) and loads are shown in pounds per day (ppd).

REGULATORY REQUIREMENTS

Regulatory requirements for the wastewater system include treatment plant effluent disposal and sludge management regulations. Currently, the City disposes of effluent using land application during dry-weather months and a stage-based surface water discharge to Bear Creek the rest of the year. The City's National Pollution Discharge Elimination System (NPDES) permit regulates both disposal methods. Sludge management is regulated by federal code.

EXISTING FACILITIES AND CONDITIONS

The existing wastewater system facilities include a gravity collection system and a lagoon treatment plant. The existing facilities were investigated using site visits, interviews with City staff, and review of available records, mapping, and regulations.

TABLE ES-1.
WASTEWATER FLOW AND LOAD PROJECTIONS

	1999	2009	2019
Population	5,770	9,950	13,370
Projected Flows (mgd)			
Average dry weather flow (ADWF)	0.92	1.58	2.12
Maximum month dry weather flow (MMDWF)	1.70	2.94	3.95
Average wet weather flow (AWWF)	1.85	2.61	3.23
Maximum month wet weather flow (MMWWF)	3.10	4.37	5.41
Peak daily average flow (PDAF)	6.22	8.77	10.86
Peak instantaneous flow (PIF)	10.59	14.93	18.49
Projected Loads (ppd)			
Biochemical oxygen demand (BOD): Average	1,330	2,300	3,090
BOD: Maximum	2,990	5,160	6,940
Total suspended solids (TSS): Average	940	1,610	2,170
TSS: Maximum	2,070	3,570	4,790

Collection System

The collection system has approximately 100,000 feet of piping and over 250 manholes. Most of the system was installed after 1955 and uses piping made of concrete, asbestos cement (AC) or polyvinyl chloride (PVC). The 3,700 feet of pre-1955 sewer lines are open-jointed concrete pipe. Much of the system drains to the north and then follows Toliver Road west to the treatment plant. A recent trunk installed in the south end of the City diverts some of the flows along Highway 211 and Bear Creek to the plant. There are also four small collection system pump stations.

Though there are areas of newer sewers, the collection system in general is aging. The system experiences levels of infiltration and inflow (I/I) well beyond the per capita flow guidelines developed by the Environmental Protection Agency (EPA). The major deficiencies associated with the existing collection system are as follows:

- High I/I indicates problems in the integrity of the collection system piping. Over half of the collection system is over 40 years old.
- There is approximately 3,700 feet of open joint pipe.
- A significant portion of the previous I/I reduction work (grouting) work has exceeded its useful life and has failed.
- Facilities are generally hydraulically overloaded during winter storm events, which causes I/I-related surcharging.

Treatment Plant

The existing plant was constructed in 1980. The plant has a headworks, which includes comminution (grinding) and flow measurement using a Parshall flume. Influent flows by gravity from the headworks to an aeration basin. A pump station is required to transfer the wastewater from the aeration basin to the first of two facultative lagoons, which provide both treatment and storage. Disinfection is accomplished using gaseous chlorine. Dry-weather effluent is disposed of by land application on the plant site and on adjacent land owned by a farmer. Excess dry weather effluent is stored in the lagoons. Wet-weather flows and stored effluent are further treated using dissolved air flotation (DAF) and gravity filters prior to a stage-based surface water discharge to Bear Creek.

The existing treatment facilities were found to have a number of deficiencies for existing and future operation:

- The existing headworks is hydraulically undersized for existing flows.
- The headworks configuration is such that the Parshall flume does not provide accurate readings.
- Because there are no grit removal facilities, grit accumulates in the aeration basin and facultative lagoons.
- The transfer pump station and associated force main are hydraulically undersized.
- The chlorine contact tank is hydraulically undersized for existing flows.
- Mass load limits to Bear Creek exceed permitted limits during peak wet-weather flows.
- Application of effluent on land currently exceeds defined agronomic rates.

COLLECTION SYSTEM ANALYSIS

The analysis of the collection system includes an evaluation of I/I, estimations of future system expansion, and evaluation of the existing system relative to future flow projections.

Infiltration/Inflow

Investigation into current levels of I/I consisted of flow monitoring. It was determined that I/I is fairly widespread and will be difficult to remove cost-effectively. As a result, no specific I/I removal projects are recommended. The recommendation for addressing I/I is to implement an I/I reduction program to minimize further collection system degradation and subsequent I/I increases. This I/I reduction program includes the following:

- Smoke testing, cleaning and video survey of the sanitary sewer collection system
- Correction of problems identified by the video survey
- Cleaning of the storm sewer system.

Future Expansion

Areas of potential expansion of the system were investigated and the impact of the expansion on the existing system was assessed. It was assumed that future development will be required to provide adequate facilities to convey flows to the existing system. There are, however, two system extension projects that are recommended to provide service to the predominantly undeveloped commercial and industrial area in the south end of the City. These projects are necessary to encourage economic development and are as follows:

- Industrial (South) Trunk extension to connect to the upstream end of the existing Bear Creek Trunk at Highway 211.
- Highway 213 (South) Trunk extension to connect to the existing Bear Creek Trunk.

Existing System Evaluation

The collection system was evaluated by comparing the full flow gravity capacity of the sewers with the PDAF. This approach assumes that if the sewers can accommodate the PDAF by gravity, then the PIF can be accommodated by surcharging. Evaluation of the collection system under existing conditions indicates that some trunks are currently under-capacity and must therefore convey peak flows to the plant by surcharging. Evaluation of the collection system under future conditions indicates that several trunks will be severely under-capacity. Upgrades and improvements are recommended for the following collection system components:

- The Treatment Plant Trunk from the intersection of Toliver Road and Highway 213 to the plant.
- The Toliver Road Trunk from Molalla Avenue and Heintz Street to the intersection of Toliver Road and Highway 213.
- Molalla/Highway 211 improvements including connecting the manhole at the intersection of Molalla Avenue and Highway 211 to the north. Also included is plugging the existing line downstream of the manhole (to the west).
- Bear Creek Trunk from Highway 211 and Bear Creek to the Treatment Plant Trunk.

WASTEWATER TREATMENT PLANT ANALYSIS

Wastewater treatment plant design criteria were developed for two phases: Phase 1, based on the 2009 flow and load projections, and Phase 2, based on the 2019 flow and load projections. Treatment facilities must generally accommodate an MMDWF of 2.9 mgd for Phase 1 and 4.0 mgd for Phase 2.

Six general approaches for wastewater treatment improvements were considered and three were evaluated in detail—upgrade of the existing plant treatment processes; converting the plant to an oxidation ditch plant; and converting the plant to a sequencing batch reactor (SBR) plant. The alternative to upgrade the existing plant treatment processes is the recommended alternative.

Three solids management alternatives were evaluated in detail: a facultative sludge lagoon (FSL); aerobic digestion followed by an FSL; and aerobic digestion followed by lime stabilization. The alternative to add an FSL is the recommended alternative.

The recommendations for Phase 1 facilities are as follows:

- Preliminary Treatment—Installation of facilities for fine screening with washing and compaction and flow measurement.
- Transfer Pumping—New pump station with vertical turbine solids handling pumps.
- Secondary Treatment—Construction of two aeration basins within the existing Lagoon No. 1 and installation of aeration equipment.
- Advanced Treatment—Upgrade and expansion of the dissolved air flotation and gravity sand filters.
- Effluent Disinfection—Upgrade of chemical equipment and addition of a chlorine scrubber at the existing plant site.
- Other—Miscellaneous improvements to the office/laboratory.
- Solids Management—Construction of diking in the existing Lagoon No. 1 for the Phase 2 facultative sludge lagoon to be completed as part of the Phase 2 treatment plant improvements.

Implementation of Phase 2 facilities is dependent on actual long-term population growth and industrial development. The recommendations for Phase 2 facilities are as follows:

- Secondary Treatment—Add secondary clarification and return activated sludge (RAS) and waste activated sludge (WAS) facilities. Add additional aeration to the aeration basins.
- Solids Management—Install a liner and associated piping in the facultative sludge lagoon cell.
- The Biosolids Management Plan will have to be updated for the recommended solids management facilities.

EFFLUENT DISPOSAL

Various methods of effluent disposal were evaluated and a combination of summer reuse and winter discharge to both Bear Creek and the Molalla River is recommended. The design of the initial facilities to implement the effluent disposal recommendations was developed concurrently with this plan and is detailed in the *City of Molalla Preliminary Design Report; Effluent/Irrigation Pump Station, Forcemain, and Outfall, TetraTech/KCM, October 1999*. The overall effluent disposal recommendations are summarized as follows:

- Effluent/Irrigation Pump Station—Installation of effluent/irrigation pump station and flow control facilities at the wastewater treatment plant. This project also includes a standby generator sized to run the plant during a power outage.

- Effluent/Irrigation Forcemain—Installation of effluent/irrigation forcemain to Feyrer Park Road.
- Effluent/Irrigation Forcemain Extension and Outfall—Installation of effluent/irrigation forcemain extension from Feyrer Park Road to an outfall to the Molalla River.
- Phase 1 Effluent Storage Facilities—Purchase of property adjacent to plant, installation of half of the required effluent storage volume, and installation of additional irrigation facilities.
- Phase 2 Facilities—Installation of the second half of the required effluent storage volume, installation of additional irrigation facilities, and addition of effluent/irrigation pump.

The following issues must be addressed in addition to the design of the recommended effluent disposal facilities:

- An Effluent Reuse Plan will be required prior to irrigation at the Coleman Ranch.
- The City must apply for an effluent discharge permit to the Molalla River from the DEQ.

IMMEDIATE RECOMMENDATIONS

The wastewater facility evaluations and recommendations focus on the requirements for the 20-year planning period. These recommendations will take a minimum of two to three years to implement. During this interim period, increasing flows and loads will exceed the capacity of the existing treatment plant and disposal facilities. This was demonstrated by the recent Notice of Noncompliance (NON) issued to the City by DEQ for exceeding permit limits during the first three months of 1999. To address this shortcoming, immediate recommendations were developed to provide short-term capacity at the existing plant prior to implementation of the long-term recommendations. The immediate recommendations are included in the capital improvement program and are as follows:

- Removal of the sludge accumulation in Lagoon No. 1.
- Addition of dechlorination facilities.
- Addition of aeration at the inlet end of Lagoon No. 1.
- Improvements on the piping out of Lagoon No. 2.

CAPITAL IMPROVEMENT PROGRAM

Recommendations for the wastewater collection system and the treatment facilities were compiled into the capital improvement program shown in Table ES-2.

TABLE ES-2.
CAPITAL IMPROVEMENT PROGRAM SUMMARY

Project	Chapter Discussed	Year (Range)	Project Budget
Collection System			
C1. Treatment Plant Trunk Upgrade	6	2002-2003	$600,000
C2. Toliver Road Trunk Upgrade	6	2003-2006	$2,200,000
C3. Molalla/Hwy 211 Improvements	6	2003-2006	$50,000
C4. Bear Creek Trunk Upgrade	6	2004-2008	$450,000
C5. Industrial (South) Trunk Extension	6	2005-2009	$860,000
C6. Highway 213 (South) Trunk Extension	6	2005-2009	$310,000
Wastewater Treatment Plant			
T1. Effluent/Irrigation Pump Station	8	2000	$1,410,000
T2. Effluent/Irrigation Forcemain	8	2000	$1,900,000
T3. Effluent Reuse Plan	8	2000	$20,000
T4. Effluent Discharge Permit to Molalla River	8	2000	$80,000
T5. Add Dechlorination Facilities	9	2000	$20,000
T6. Add Aeration in Lagoon No. 1	9	2000	$50,000
T7. Improve Piping out of Lagoon No. 2	9	2001	$60,000
T8. Phase 1 Preliminary Treatment Upgrades	7	2001	$630,000
T9. Phase 1 Transfer Pumping Upgrades	7	2001	$1,380,000
T10. Phase 1 Advanced Treatment Upgrades	7	2002	$2,140,000
T11. Effluent/Irrigation Forcemain Extension and Outfall	8	2003	$1,270,000
T12. Phase 1 Effluent Storage Facilities	8	2004-2006	$3,030,000
T13. Phase 1 Secondary Treatment Upgrades	7	2005-2008	$1,490,000
T14. Phase 1 Solids Management Upgrades	7	2005-2008	$350,000
T15. Phase 1 Effluent Disinfection Upgrades	7	2005-2008	$380,000
T16. Phase 1 Miscellaneous Upgrades	7	2005-2008	$50,000
T17. Phase 2 Effluent Storage and Irrigation Facilities	8	2010-2014	$2,630,000
T18. Phase 2 Treatment Upgrades	7	2010-2014	$3,280,000
T19. Biosolids Management Plan	7	2010-2014	$20,000

FUNDING PLAN

Funding sources were reviewed and the most feasible sources for the City of Molalla were identified. The City will prepare a cost-of-service and SDC analysis as a document separate from this report.

CHAPTER 1.
INTRODUCTION

1.1 AUTHORIZATION

In December 1998, the City of Molalla authorized KCM, Inc. to prepare this Wastewater Facilities Plan.

1.2 PURPOSE AND SCOPE

The purpose of this study is to evaluate the City's wastewater system, assess current facility conditions, identify future wastewater facility needs for a 20-year study period (through 2019), compile a capital improvement plan, and develop a funding plan.

This Wastewater Facility Plan generally meets the 1998 guidelines for facilities planning outlined by the Oregon Department of Environmental Quality (DEQ), but is not intended to meet the requirements of the DEQ State Revolving Fund (SRF) program. This plan provides a firm basis for design of the needed improvements. Preparation of the plan involved the following steps:

- Review data and documents available from the City—including as-built drawings, mapping, and operational data—in order to describe and evaluate existing facilities.
- Project future conditions by updating population projections, evaluating existing flows and loads to the wastewater treatment plant, and projecting future flows and loads.
- Summarize regulatory standards and determine whether they are being met.
- Evaluate the collection system through infiltration and inflow (I/I) monitoring that includes flow monitoring during the winter. Identify areas with high I/I and evaluate I/I removal alternatives.
- Determine the size and location of trunk sewers that would be needed to extend service to undeveloped areas and the upsizing requirements for existing trunk sewers.
- Develop and evaluate alternatives for addressing growth requirements and existing deficiencies in the City's treatment and disposal systems.
- Review the alternatives from a qualitative perspective. Determine whether I/I removal or transportation and treatment of flows is more cost-effective.
- Prepare preliminary layout drawings showing the recommended improvements in schematic form.
- Develop a capital improvement plan.
- Evaluate funding sources and develop a funding plan.

1.3 PLANNING BACKGROUND

The following documents describe previous wastewater system planning and development for the City of Molalla service area:

- *Wastewater System Facility Plan, Review Draft, City of Molalla, Oregon*, EAS, November 1996.
- *Highway 211 Sanitary Sewer Improvements—Bear Creek to Molalla Avenue "As-Built," City of Molalla, Oregon*, DeHaas, August 1995.
- *Toliver/Bear Creek Sanitary Sewer Diversion "As-Built," City of Molalla, Oregon*, DeHaas, December 1991.
- *Bear Creek Interceptor "As-Built," City of Molalla, Oregon*, DeHaas, September 1988.
- *Comprehensive Plan, Molalla, Oregon*, 1987.
- *East Main Street Sewer Extension, City of Molalla, Oregon*, Westech, October 1979.
- *Sewage Treatment Plant (Schedule T) and Trunk Sewer (Schedule S), City of Molalla, Oregon*, Westech, May 1977.
- *Sewage Collection System, City of Molalla, Oregon*, Clark & Groff, October 1954.

1.4 PUBLIC INVOLVEMENT

In addition to presentations to the City Council, two public involvement meetings were held to inform interested parties of the recommendations of this plan. The meetings were held on November 17, 1999 at 7:00 p.m. and on December 8, 1999 at 12:00 p.m. The agendas and minutes for these meetings are included in Appendix H.

CHAPTER 2.
SERVICE AREA, LAND USE, AND POPULATION

2.1 SERVICE AREA

The City of Molalla is in Clackamas County, approximately 30 miles south of Portland (see Figure 2-1). Highway 213 runs north-south through the west end of the City and Highway 211 runs east-west through the middle. The Molalla River is located just east of the City.

The service area for this study is defined by the existing Urban Growth Boundary (UGB), which is shown in Figure 2-2. Currently, the Molalla wastewater system serves much of the developed area within the City limits.

The existing UGB is expected to reach buildout (the maximum amount of development allowed by zoning) within the 20-year planning period. For this study, the City directed KCM to plan sanitary sewer facilities for the 20-year population projections and provided general information on how the UGB may be expanded in the future. This information is discussed in more detail below.

2.1.1 Topography

The UGB is located on level to gently sloping land with the center of the City (the intersection of Highway 211 and Molalla Avenue) at an approximate elevation of 375 feet. Elevations within the City range from approximately 300 feet to 420 feet. Ground slopes range from nearly flat to approximately 10 percent.

2.1.2 Geology

Predominant soils in the study area are alluvial silt deposits of the Concord-Clackamas-Amity and Briedwell associations. These soils have high seasonal water tables and a depth to hard rock of 20 to 40 inches or more. Although classified as silts, these soils contain areas of clay, gravel, or loam and are somewhat poorly drained. Septic tank limitations in the area are classified as moderate to severe. The soils, however, are classified as having fair stability for building sites with slight to moderate restrictions.

2.1.3 Water Resources

The Molalla River runs generally south to north just east of Molalla; the City is located at approximately river mile 20. Two branches of Creamery Creek flow through the north end of the City and run generally from the southeast to the northwest. These branches meet east of Highway 213, and Creamery Creek flows into the Molalla River several miles outside the UGB. Another City creek is Bear Creek, which runs generally parallel to and south of Creamery Creek in the vicinity of Molalla. Bear Creek eventually flows into the Pudding River. The Pudding River flows into the Molalla River at approximately river mile 1, just before the Molalla River enters the Willamette River.

During this century, Molalla has met the needs of municipal water users by withdrawing, treating, and distributing water from the Molalla River and Trout Creek, which is a tributary of the Molalla River. The City has also pursued the use of groundwater wells to supplement the water supply. However, wells in the Molalla area tap a variety of rock units and generally yield small to modest quantities of water.

2.1.4 Climate

The climate of the study area includes dry, moderately warm summers and mild, wet winters. The temperature ranges from an average high of 81°F in July to an average low of 33°F in January. The average annual rainfall is 47 inches, over 75 percent of which falls from November to April on average.

2.2 LAND USE

Molalla's Comprehensive Plan was published in 1987 and is implemented by the City's zoning code and other plans and ordinances. Figure 2-3 shows the Molalla zoning map from the 1987 Comprehensive Plan.

The City is primarily zoned residential, with a downtown commercial center and an industrial area in the southwest. Wood-product mills are the largest industries in Molalla; however, significant industrial land exists within the UGB for diversified industrial growth in the future.

There are no designated floodplains in Molalla, but Bear Creek lies within the UGB. Bear Creek's floodplain has never been defined, as it was outside the corporate limits when the U.S. Army Corps of Engineers last mapped floodplains in Oregon. Bear Creek should be studied in the near future because annexations will bring it into the City within the next few years.

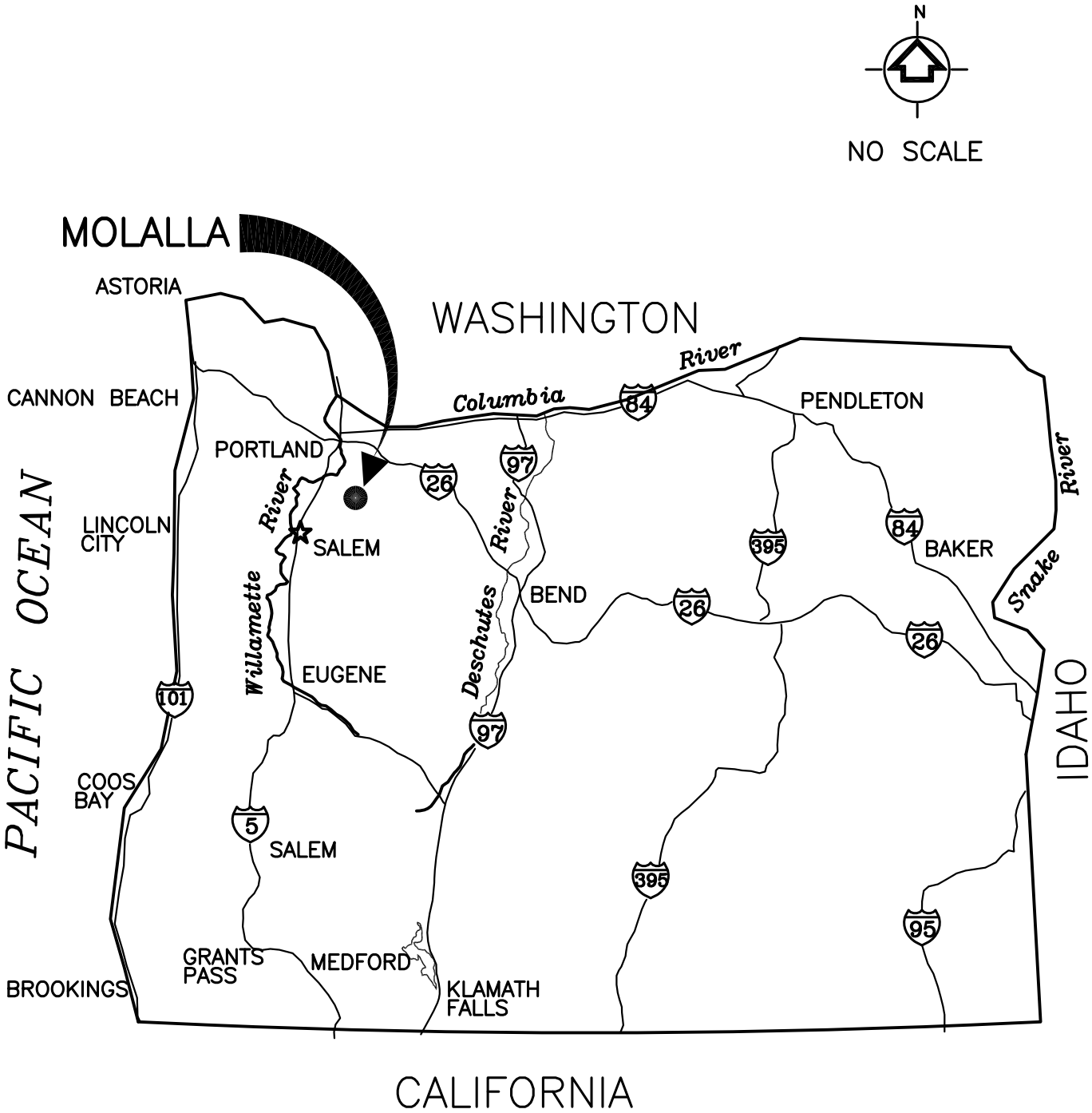
Lands surrounding Molalla are predominantly used for agricultural purposes. Significant stands of timber are located nearby to the east in the Cascade Range foothills.

2.3 POPULATION

Historical population records and expected land development were used to project future populations through the 20-year study period.

2.3.1 Historical Population

Table 2-1 summarizes historical population and average annual growth rates for the City of Molalla. Population data are from Portland State University's (PSU) Population Research and Census Center.



2140078FIG2-1.DWG 1=1 2-6-2002

Tetra Tech/KCM, Inc.
7080 SW Fir Loop
Portland, Oregon 97223
503-684-9097 Fax: 503-598-0583

City of Molalla, Oregon	FIGURE 2-1
STORMWATER MASTER PLAN	LOCATION MAP

2140078 FIG2-2 1=10002-6-2002

Tetra Tech/KCM, Inc.
7080 SW Fir Loop
Portland, Oregon 97223
503-684-9097 Fax: 503-598-0583

City of Molalla, Oregon	FIGURE 2-2
STORMWATER MASTER PLAN	URBAN GROWTH BOUNDARY

Tetra Tech/KCM, Inc.
7080 SW Fir Loop
Portland, Oregon 97223
503-684-9097 Fax: 503-598-0583

City of Molalla, Oregon STORMWATER MASTER PLAN	FIGURE 2-3 ZONING MAP

TABLE 2-1.
HISTORICAL POPULATION

Year	Population	Annual Growth Rates			
		1-year Average	5-year Average	10-year Average	20-year Average
1965	1,599				
1970	2,005		4.6%		
1975	2,760		6.6%	5.6%	
1980	2,992		1.6%	4.1%	
1985	3,100		0.7%	1.2%	3.4%
1990	3,637		3.2%	2.0%	3.0%
1991	3,650	0.4%	2.8%	1.6%	2.6%
1992	3,680	0.8%	2.7%	1.8%	2.4%
1993	3,820	3.8%	3.4%	2.4%	2.3%
1994	3,915	2.5%	3.9%	2.4%	2.0%
1995	4,045	3.3%	2.1%	2.7%	1.9%
1996	4,505	11.4%	4.3%	3.5%	2.4%
1997	4,920	9.2%	6.0%	4.3%	2.6%
1998	5,395	9.7%	7.1%	5.3%	3.2%

PSU calculated a population density value from the 1980 census of 2.80 people per dwelling. For subsequent population estimates, PSU has assumed a slightly lower density. PSU assumed 2.73 people per dwelling for its 1998 population estimate.

2.3.2 Growth Potential

The following are key factors considered in estimating growth for the Molalla area:

- Growth in surrounding communities and other similar areas, including the suburbs of the Portland Metropolitan area
- Previous population projections
- Available land for development
- Job opportunities.

Growth in Surrounding Communities

In general, northwest Oregon is experiencing strong growth with a healthy economy. The strongest growth in the region is in the Portland Metro area and its fringes, including Molalla. Many communities around the Metro area are experiencing overflow from Portland, with people willing to commute longer to gain a better quality of living. Molalla fits this description, as recent growth within the City suggests.

Previous Population Projections

Population projections in the 1987 Molalla Comprehensive Plan assumed strong growth during the 1980s (7 percent annual average) and slower growth during the 1990s and early 2000s (2 to 3 percent annual average). The resulting population projection for 1998 was 7,317, which is 36 percent higher than the actual 1998 population of 5,395. The strong growth predicted in the 1980s, however, did occur during the mid-1990s.

A more recent population projection was performed for the Water System Master Plan (EAS, 1996), which assumed a steady 5 percent average annual growth rate over a 20-year planning period. Based on the 1995 population of 4,000, the population projection for 2016 was 11,144.

Metro, the planning agency for the Portland metropolitan area, has performed population projections throughout its four-county area (Clackamas County, Multnomah County, Washington County, and Columbia County). The Metro projections are based on 1995 population and are delineated by transportation analysis zone (TAZ). Molalla is included in a very large TAZ, TAZ 535, that is roughly bounded by the divide between the Molalla and Clackamas River basins on the east, the Clackamas County boundary on the southwest, and the Cities of Canby and Estacada on the north. The 1995 population estimate for this TAZ was 25,963, of which Molalla's population made up 15.4 percent. The Metro projections for this TAZ assume 1.8 percent average annual growth between 1995 and 2000 and somewhat declining growth rates through 2020, for an overall average annual growth rate between 1995 and 2020 of 0.9 percent. The resulting 2000 and 2020 projected populations for the TAZ are 28,345 and 32,593, respectively.

PSU has also made countywide population projections based on the estimated 1995 population (312,294 for Clackamas County, with Molalla contributing 1.3 percent). PSU estimates that the county population will grow at average annual rates of 1.7 percent through 2000, 1.6 percent from 2000 to 2005, and 1.5 percent from 2005 to 2010. The resulting 2000 and 2010 projected county populations are 339,451 and 395,138, respectively.

Table 2-2 summarizes the previous population projections. The annual rates shown represent the average annual growth rate between the corresponding population projection and the previous (five year earlier) population projection.

Available Land for Development

The City currently has a number of proposed housing units. A December 28, 1998 City listing indicates that 267 subdivision housing units have been approved but are not yet built and another 475 subdivision housing units are proposed. Assuming 2.7 people per dwelling (somewhat less than the 1998 PSU estimate of 2.73, to account for the continuing declining trend in this value), development of these homes will accommodate an additional population of 2,006. At the current average annual growth rate near 9 percent, this capacity would be filled in less than four years.

TABLE 2-2.
PREVIOUS POPULATION PROJECTIONS

	1985 Comp Plan – City of Molalla		1996 Water Plan – City of Molalla		Metro Projections - TAZ 535		PSU Projections - Clackamas County	
Year	Population	Annual Rate	Population	Annual Rate	Population	Annual Rate	Population	Annual Rate
1980	3,663							
1985	5,136	7.0%						
1990	5,952	3.0%						
1995	6,897	3.0%	4,000		25,963		312,294	
2000	7,645	2.1%	5,105	5.0%	28,345	1.8%	339,451	1.7%
2005	7,940	0.8%	6,516	5.0%	30,187	1.3%	367,332	1.6%
2010			8,316	5.0%	31,479	0.8%	395,138	1.5%
2015			10,613	5.0%	32,032	0.4%		
2020					32,593	0.4%		

To accommodate additional growth, remaining areas within the UGB will be annexed to the City and developed. Beyond the sites of the development already proposed, however, there is little undeveloped residential land left within the UGB. Once the land within the UGB is near full development, infilling in predominantly developed areas and redevelopment of existing neighborhoods to higher densities are likely to occur. These development patterns, however, will not sustain the level of growth that the City is currently experiencing.

The City expects that the UGB will be expanded in the near future to accommodate a continued demand for growth. A likely area of expansion for additional residential development is to the north to Vick Road in the west end of the City. Other options are to the west of Highway 213, to the north in the east end of the City, and to the southeast. The addition of residential land within the UGB is likely to allow for continued growth at or near the rates recently experienced in Molalla.

Job Opportunities

Clackamas County, including the City of Molalla, has historically depended on the forest products industry. As old growth forests have been harvested, growth of this industry has slowed, and this is a possible cause for the City's lower-than-expected overall growth in the late 1970s and early 1980s. However, the City has seen a return of accelerated growth from 1995 to 1998, and it is reasonable to assume increased population and business growth over the next 20 years for several reasons:

- Molalla has excellent highway access to the greater Portland Metro area, which has continued to experience growth in employment opportunities. As such, Molalla is developing into a "bedroom community" for those who live in Molalla and work in the Portland Metro area.

- There are available tracts of vacant commercial and industrial land within the UGB. Development on this land should increase employment opportunities within the City.
- Molalla's small commercial areas have historically struggled to compete with the larger Portland commercial areas, but population and employment growth in the City should enhance the future well-being of its commercial areas.

Industrial development in Molalla has never been a significant factor in growth of the City, with the exception of the Avison lumber mill. The City has no large tracts (greater than 100 acres) of open land zoned for industrial development that are not currently occupied; and the City's water supply is not adequate to accommodate large industries with high water needs. For these reasons, growth projections assume only minor industrial development. In calculating wastewater flow projections (see Chapter 3), it is assumed that the ratio of industrial sewage to residential sewage will remain constant.

2.3.3 Population Projections

The City expects its historically strong growth to continue, despite the fact that PSU projects Clackamas County to experience growth in the range of 1.6 percent over the next decade and Metro projects growth in Molalla's TAZ in the range of 1.0 percent over the next 20 years. The PSU and Metro projections were made prior to the last three years of high growth in Molalla, so they do not account for the recent data. Additionally, Molalla is a very small component of the much broader projections by PSU (all of Clackamas County) and Metro (all of the four-county area) and the potential for growth within Molalla did not likely impact these more macroscopic projections. As a result, population projections for Molalla are based on its own historical growth rates and potential.

For near-term (five-year) planning, it is assumed that growth will closely match the current five-year average annual growth. Growth over the subsequent five years is assumed to closely match the current 10-year annual average; and growth over the following decade is assumed to closely match the current 20-year annual average. The assumed rates for the 20-year study period, therefore, are as follows:

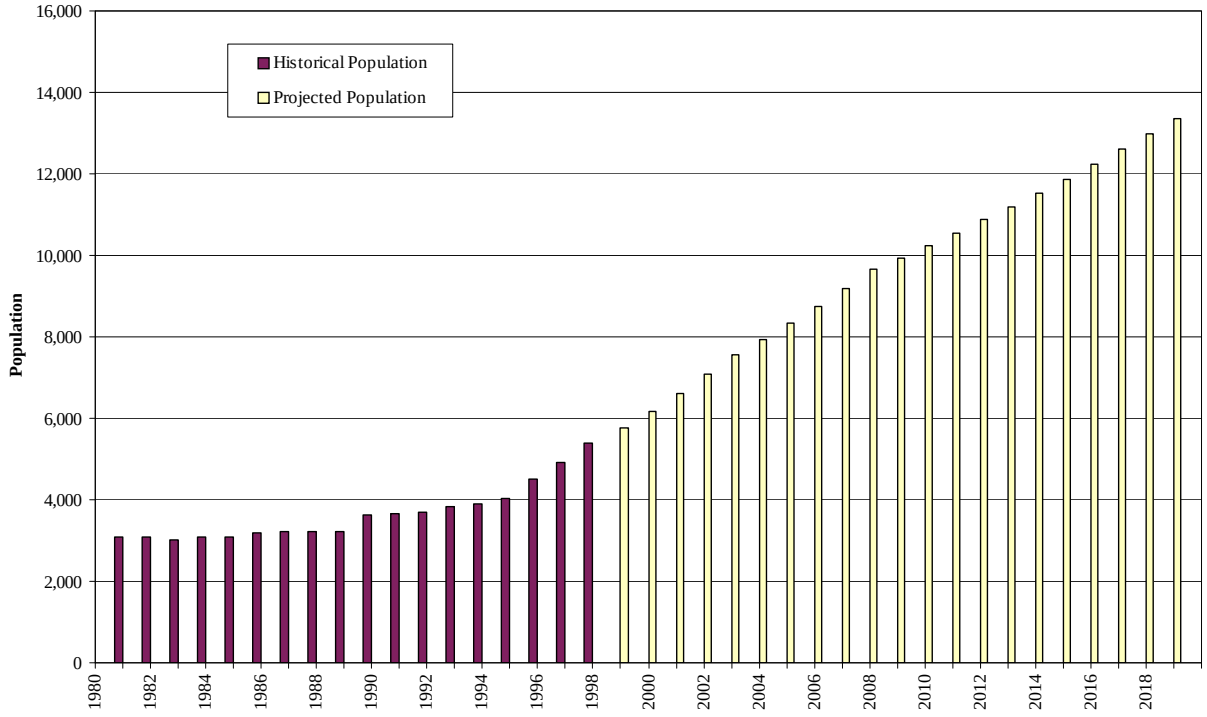
- 7 percent during the first five years (1999 to 2003)
- 5 percent during the next five years (2004 to 2008)
- 3 percent during the remaining 10 years (2009 to 2019)

The resulting population projections based on the 1998 population of 5,395 are shown in Table 2-3. The historical population within the City of Molalla between 1990 and 1998 (shown in Table 2-1) and the population projections (shown in Table 2-3) are shown in Figure 2-4.

TABLE 2-3.
POPULATION PROJECTIONS

Year	Population
1999	5,770
2004	7,950
2009	9,950
2014	11,530
2019	13,370

FIGURE 2-4
POPULATION CHART



CHAPTER 3.
WASTEWATER FLOWS AND LOADS

3.1 WASTEWATER FLOW PROJECTIONS

Wastewater flow projections for the 20-year study period are based on historical flows to the wastewater treatment plant and the population projections described in Chapter 2.

3.1.1 Wastewater Flow Classifications

The following classifications are used in determining design flows:

- Average Dry Weather Flow (ADWF)—Average daily wastewater flow during the dry weather season (from May through October).
- Maximum Month Dry Weather Flow (MMDWF)—Average daily flow for the highest-flow month during the dry weather season.
- Average Wet Weather Flow (AWWF)—Average daily wastewater flow during the wet weather season (from November through April).
- Maximum Month Wet Weather Flow (MMWWF)—Average daily flow for the highest-flow month during the wet weather season.
- Peak Daily Average Flow (PDAF)—Highest average flow for a 24-hour period.
- Peak Instantaneous Flow (PIF)—Highest instantaneous flow, often defined as the sustained one-hour peak.

Under DEQ guidelines, the design MMDWF is to be calculated assuming a summer storm with a 10-year recurrence interval (i.e., a storm with a 10 percent probability of occurring in any given dry-weather season), and MMWWF, PDAF, and PIF are to be calculated assuming a winter storm with a 5-year recurrence interval (*Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF*, Rev. IV.96).

3.1.2 Historical Wastewater Flow Records

A listing of the historical monthly wastewater influent flows to the Molalla treatment plant for January 1996 through December 1998 is included in Appendix A. Table 3-1 summarizes flow monitoring records to the plant during this period. The table shows flows to the plant in million gallons per day (mgd), as well as unit flows in gallons per capita per day (gpcd), which were determined using the historical flow and population records.

The MMWWF and the PDAF shown for 1996 do not include three days during February 1996 when flows to the plant could not be measured because they exceeded the capacity of the influent Parshall flume. Since the February 1996 storm was much greater than a 5-year event, the absence of data for this storm does not result in underestimation of the MMWWF and PDAF.

TABLE 3-1.
HISTORICAL WASTEWATER FLOWS TO THE TREATMENT PLANT

Year	Population	ADWF (mgd)	ADWF (gpcd)	MMDWF (mgd)	MMDWF (gpcd)	AWWF (mgd)	AWWF (gpcd)	MMWWF (mgd)	MMWWF (gpcd)	PDAF (mgd)	PDAF (gpcd)
1996	4,505	0.83	184	1.33	295	1.84	396	2.68	553	5.00	1,110
1997	4,920	0.69	141	0.94	191	1.52	308	2.23	453	4.64	943
1998	5,395	0.82	151	1.36	252	1.55	287	2.00	371	4.78	886
		Avg.=159 gpcd		Max.=295 gpcd		Avg.=331 gpcd		Max.=553 gpcd		Pk=1,110 gpcd	

The records show that 1996 flows were generally high relative to 1997 and 1998. The nearest currently active National Climatic Data Center is in Oregon City, approximately 13 miles north of Molalla. Records of total rainfall at that station indicate that 1996 had the highest annual rainfall amount between 1948 and 1998. Though the total annual rainfall does not necessarily have a direct impact on the amount of flow to the treatment plant, these records provide an indication of why flows to the plant were higher during 1996. Because a wetter weather trend is expected to continue, it is reasonable to expect high precipitation similar to that experienced in 1996.

3.1.3 Infiltration and Inflow Reduction Considerations

High groundwater (infiltration) and stormwater runoff (inflow) components of wastewater flow enter a sanitary collection system primarily during wet weather. Under Environmental Protection Agency (EPA) guidelines, an in-depth infiltration and inflow (I/I) investigation is required for wastewater systems that do not meet both of the following criteria:

- Combined flows from domestic wastewater and infiltration are less than 120 gallons per capita per day (gpcd) during periods of high groundwater, based on the highest 7- to 14-day average.
- During a storm, the total daily flow is less than 275 gpcd, there are no surcharges or bypasses, and treatment performance is adequate.

The highest 7- to 14-day average flow during periods of high groundwater is generally represented by AWWF. Molalla's historical records show that the average AWWF between 1996 and 1998 was over 300 gpcd. The historical records also indicate that PDAF, which is essentially equivalent to the total daily flow during a storm, ranged from approximately 900 to 1,100 gpcd over the past three years. According to the records, therefore, I/I in the City's system is excessive per EPA criteria and further investigation is required. Chapter 6 describes the I/I analysis performed for this study.

3.1.4 Influent Measurement Considerations

The configuration of the existing headworks raises concerns about the accuracy of flow measurements made at the influent flume. For accurate measurement with a Parshall flume, the approaching flow should be relatively free of turbulence. At the Molalla

treatment plant, however, there is a steep slope in the channel from the comminutors and bar screen just upstream of the 9-inch Parshall flume. This slope is shown on the construction plans as a 12-inch drop over a 24-inch length. This slope, in combination with the convergence of three channels just upstream of the slope, creates considerable turbulence at the upstream end of the flume.

The City has performed an investigation into the effect of the headworks channel configuration on the flow measurements at the flume. The City installed a Marsh-McBirney flow meter in the influent line upstream of the plant. The meter readings were compared to the flow measurements at the flume. In general, influent flows less than 2.5 mgd were measured accurately at the flume, but influent flows greater than 2.5 mgd were measured approximately 10 to 15 percent high at the flume. Apparently, under higher flow conditions, the existing headworks configuration creates a hydraulic jump at the upstream end of the flume, causing the head measurements to be high.

For the wastewater flow projections, no adjustments were made to the historical records to account for the effects of the headworks configuration. Because the measurements taken at the flume are somewhat high, using the historical records without adjustments provides conservative estimates. In addition, PIF, whose measurement would be most affected by the headworks configuration, exceeds the 5.75-mgd capacity of the flume, and therefore is not included in the summary of historical flows in Table 3-1.

3.1.5 Unit Wastewater Flows

Projections of future wastewater flows are developed using population (and sometimes land use information) together with unit values that define wastewater flows per capita (or per acre for commercial and industrial areas). Unit wastewater flows were calculated using historical plant records ADWF, AWWF, MMDWF, MMWWF, PDAF and PIF. In addition, procedures outlined by the DEQ were used to calculate MMDWF, MMWWF, PDAF, and PIF. The analysis and results using the DEQ procedure are described in Appendix A.

As discussed in Chapter 2, industrial development within Molalla has never been a significant factor in the growth of the City, with the exception of the Avison lumber mill, and only minor future industrial growth is expected. For the wastewater flow projections, the ratio of commercial and industrial sewage flow to residential flow is assumed to remain constant. This allows the use of City-wide wastewater unit flows for the wastewater flow projections.

Unit Flows for Existing Population

The following unit flows were established for the existing system:

- Based on population records and treatment plant flow records for 1996 through 1998, ADWF and AWWF unit flows were calculated as 159 gpcd and 331 gpcd, respectively.
- Unit flows for MMDWF, MMWWF, PDAF, and PIF were found to be higher when calculated using historical records than when calculated using DEQ procedures. To be conservative, the higher, historical-based unit flow values are used:

- 295 gpcd for MMDWF
- 553 gpcd for MMWWF
- 1,110 gpcd for PDAF
- 1,890 gpcd for PIF (because historical records for PIF have exceeded the capacity of the influent Parshall flume that measures the flows, this value was determined by extrapolating from other historical measurements).

Unit Flows for Additional Future Population

As the population in Molalla grows, new sections will be added to the collection system. These new sections should be more watertight than the older portions of the system, thus allowing less I/I. Also, future infill development requiring no additional main sewer lines or manholes will contribute less I/I per capita. For these reasons, historical wet weather unit flows based on the existing system with high I/I contribution will not accurately represent wet-weather unit flows for the new sections of the collection system.

To account for this, this study assumes lower wet-weather unit flows for the City's new population than for the existing population. The lower wet-weather unit flows are assumed to be slightly more than existing dry-weather unit flows. For 1996 through 1998, ADWF was about 50 percent of AWWF, and MMDWF was about 53 percent of MMWWF. As a conservative estimate, the unit flows for the added population are assumed to be 55 percent of existing-system wet-weather flows. It is important to note that the entire collection system—old and new—will become more susceptible to I/I over time unless the City makes efforts to minimize increases in I/I.

Summary

Table 3-2 summarizes wastewater unit flows used in this study for existing and future populations.

3.1.6 Flow Projections

The unit wastewater flows presented in Table 3-2 and the population projections discussed in Chapter 2 were used to project future wastewater flows. For example, the MMWWF in 2009 is equal to the existing population of 5,395 multiplied by the existing unit MMWWF of 553 gpcd plus the additional population of 4,555 multiplied by the future unit MMDWF of 304 gpcd. Table 3-3 summarizes the results. These projections assume no reduction in I/I (see Chapter 6 for discussion of I/I reduction).

There is an inherent difficulty in projecting peak wastewater flows due to the many variables involved that are difficult to predict. In particular, this includes the response of the existing system to aging and the resulting increases in I/I in the existing system, as well as the I/I that will enter newly constructed systems and the new trunk lines that transport flows to the existing system. Because of this, an alternative method of calculating the peak flow (PDAF and PIF) projections was performed and is summarized in Appendix. This check of the peak flow projections is an additional confirmation that the flows shown in Table 3-3 are reasonable for planning purposes.

TABLE 3-2.
DESIGN UNIT WASTEWATER FLOWS

Flow Condition	Unit Flow for Existing Population [a] (gpcd)	Unit Flow for Future Population [b] (gpcd)
ADWF	159	159
MMDWF	295	295
AWWF	331	182
MMWWF	553	304
PDAF	1,110	610
PIF	1,890	1,040

a. ADWF, MMDWF, AWWF, MMWWF, and PDAF based on historical plant records. PIF based on a projection of historical plant records.
b. For dry weather flows (ADWF and MMDWF), unit flows for future population same as for existing population. For wet weather flows (AWWF, MMWWF, PDAF, and PIF) unit flows for future population 55 percent of unit flows for existing population.

TABLE 3-3.
WASTEWATER FLOW PROJECTIONS

		Projected Flows (mgd)					
Year	Population	ADWF	MMDWF	AWWF	MMWWF	PDAF	PIF
1999	5,770	0.92	1.70	1.85	3.10	6.22	10.59
2004	7,950	1.26	2.35	2.25	3.76	7.55	12.85
2009	9,950	1.58	2.94	2.61	4.37	8.77	14.93
2014	11,530	1.83	3.40	2.90	4.85	9.73	16.57
2019	13,370	2.12	3.95	3.23	5.41	10.86	18.49

3.2 WASTEWATER LOAD PROJECTIONS

Wastewater load projections for the 20-year study period are based on historical loads to the wastewater treatment plant and the population projections in Chapter 2.

3.2.1 Wastewater Load Classifications

For the purpose of identifying design loads, the following classifications are used:

- Average Load—Average daily wastewater load.
- Maximum Load—Daily wastewater load for the peak week.

3.2.2 Historical Wastewater Loads

Historical wastewater loads to the treatment plant from January 1996 through December 1998 were reviewed. Table 3-4 summarizes historical 5-day biochemical oxygen demand (BOD) and total suspended solids (TSS) loading to the treatment plant. The maximum BOD load shown for 1996 does not include three days during February 1996 when flows to the plant far exceeded the capacity of the influent Parshall flume and could not be measured. Loadings are shown in pounds per day (ppd) and unit loadings, based on historical population records, are shown in pounds per capita per day (ppcd). BOD peaking factors shown in Table 3-4 represent the ratio of the maximum loading to the average loading.

TABLE 3-4.
HISTORICAL WASTEWATER LOADS

		BOD					TSS	
Year	Population	Average Load (ppd)	Average Unit Load (ppcd)	Maximum Load (ppd)	Maximum Unit Load (ppcd)	Peaking Factor	Average Load (ppd)	Average Unit Load (ppcd)
1996	4,505	1,058	0.23	3,704	0.52	2.2	673	0.15
1997	4,920	1,096	0.22	1,680	0.34	1.5	753	0.15
1998	5,395	1,273	0.24	2,095	0.39	1.6	994	0.18
			Avg=0.23		Max=0.52			Avg=0.16

Typical BOD unit loads for domestic wastewater range from 0.09 to 0.33 ppcd and average around 0.23 ppcd. Typical unit TSS loads for domestic wastewater are somewhat higher. For Molalla, the average historical BOD unit loads are therefore typical, however, the maximum historical BOD unit loads are somewhat higher than typical. Also, the average historical TSS unit loads are significantly smaller than the typical average for domestic wastewater.

The historical wastewater load records indicate that the ratio of average BOD to average TSS is approximately 1.4, whereas a ratio of 1.0 or slightly less is typical. This ratio, however, is relatively consistent throughout the evaluation period between 1996 and 1998 and is also consistent between summer and winter conditions. This ratio appears to be characteristic of recent influent flows to the plant despite the lack of an industry or other known contributor of wastewater high in soluble BOD.

3.2.3 Unit Wastewater Loads

Projections of future wastewater loads are developed using population (and sometimes land use information) together with unit loads that define wastewater loads per capita (or per acre for commercial and industrial areas). Unit wastewater loads for this study were determined using the historical plant records. Table 3-5 summarizes the unit wastewater loads.

Consistent with the wastewater flow projections, the industrial load component is assumed to grow with the population at the same ratio as current industrial loads to current population. This allows wastewater unit loads to be developed on a City-wide basis. It was also assumed that TSS loading will have the same peaking factor (the ratio of maximum load to average load) as BOD loading.

TABLE 3-5.
DESIGN UNIT WASTEWATER LOADS

Load Component	Average Unit Load (ppcd)	Maximum Unit Load (ppcd)	Peaking Factor
BOD	0.23	0.52	2.2
TSS	0.16	0.36	2.2

3.2.4 Load Projections

The wastewater unit loads presented in Table 3-5 and the population projections discussed in Chapter 2 were used to project future wastewater loads. Table 3-6 summarizes the resulting load projections. Loads are shown in pounds per day (ppd).

TABLE 3-6.
TOTAL WASTEWATER LOAD PROJECTIONS

		BOD (ppd)		TSS (ppd)	
Year	Population	Average	Maximum	Average	Maximum
1999	5,770	1,330	2,990	940	2,070
2004	7,950	1,840	4,130	1,290	2,850
2009	9,950	2,300	5,160	1,610	3,570
2014	11,530	2,670	5,980	1,870	4,130
2019	13,370	3,090	6,940	2,170	4,790

3.3 WASTEWATER FLOW AND LOAD SUMMARY

The wastewater flow and load projections discussed in this chapter are summarized in Table 3-7 for existing, 10-year, and 20-year planning conditions.

TABLE 3-7.
WASTEWATER FLOW AND LOAD PROJECTION SUMMARY

	1999	2009	2019
Population	5,770	9,950	13,370
Projected Flows (mgd)			
ADWF	0.92	1.58	2.12
MMDWF	1.70	2.94	3.95
AWWF	1.85	2.61	3.23
MMWWF	3.10	4.37	5.41
PDAF	6.22	8.77	10.86
PIF	10.59	14.93	18.49
Projected Loads (ppd)			
BOD: Average	1,330	2,300	3,090
BOD: Maximum	2,990	5,160	6,940
TSS: Average	940	1,610	2,170
TSS: Maximum	2,070	3,570	4,790

CHAPTER 4.
REGULATORY REQUIREMENTS

4.1 REGULATING AGENCIES

The U.S. Environmental Protection Agency (EPA) regulates disposal and/or reuse of sewage sludge and septage and, through the National Pollution Discharge Elimination System (NPDES) permit program, the discharge of wastewater effluent. The EPA is required to coordinate its requirements with other federal agencies—such as the National Oceanic and Atmospheric Administration (NOAA), the U.S. Army Corps of Engineers, and the U.S. Fish and Wildlife Service—and with state agencies such as the Department of Environmental Quality (DEQ), the Oregon Health Department (OHD), and the Oregon Department of Fish and Wildlife (ODFW).

4.2 EFFLUENT DISPOSAL REGULATIONS

Effluent can be disposed of by discharging it to surface water, such as a creek or river, or applying it to land. The DEQ regulates both methods of disposal. DEQ regulations require that non-discharging options be considered before discharge to a surface water can be approved and the Environmental Quality Commission (EQC) must approve of the discharge before DEQ can issue an NPDES permit. The City of Molalla, however, has already been issued a surface water discharge permit by the DEQ. Molalla also land-applies effluent during the dry season. Effluent reuse is typically regulated through a Water Pollution Control Facilities (WPCF) permit, but in Molalla the land application requirements are included in the City's NPDES permit.

4.2.1 Surface Water Discharge

The disposal of effluent to a surface water is governed by OAR 340-41, *State-Wide Water Quality Management Plan; Beneficial Uses, Policies, Standards, and Treatment Criteria for Oregon*. Discharging treatment plant effluent to a surface water (such as Bear Creek or the Molalla River) requires an NPDES Permit from the DEQ. OAR 340-41 outlines basin standards; NPDES permits regulate specific discharges.

Basin Standards

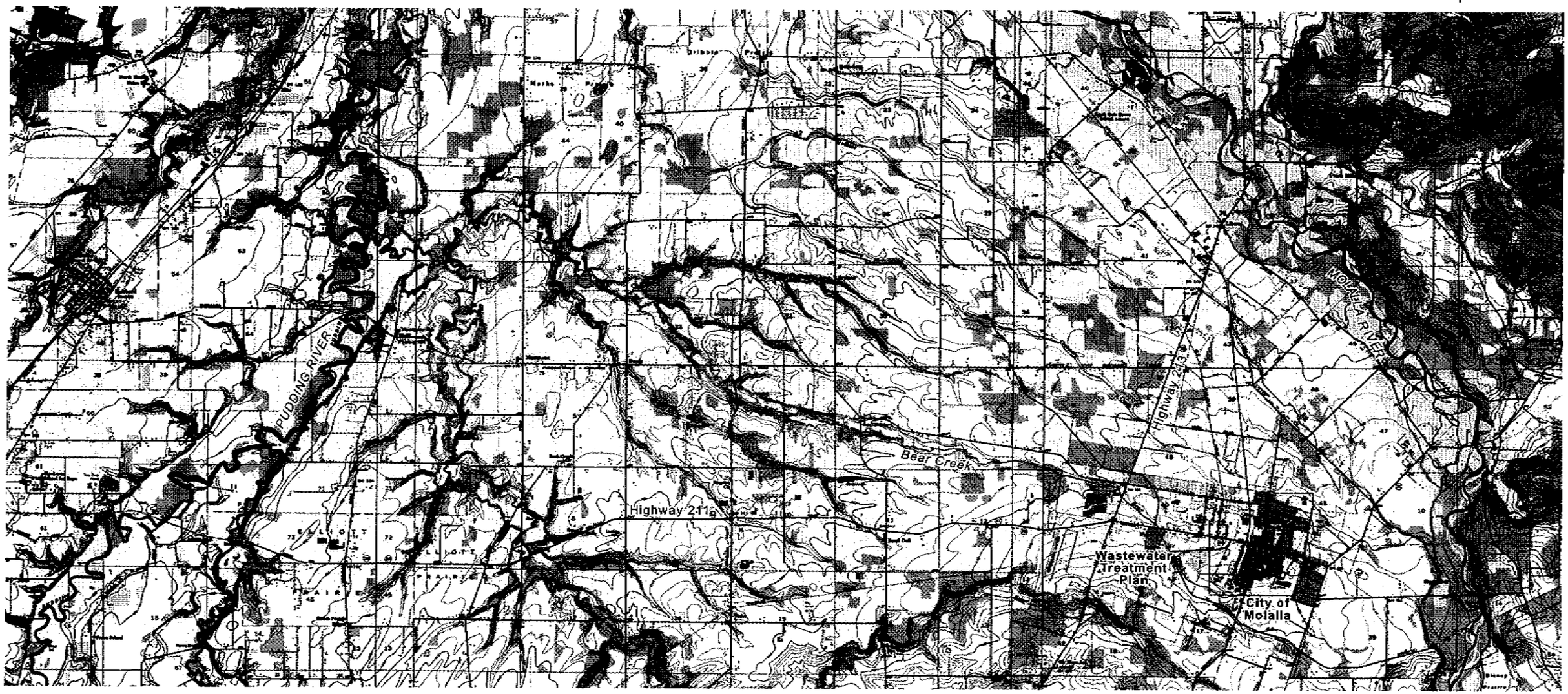
For surface water discharge, the City of Molalla is required to comply with Sections 442, 445, and 455 of OAR 340-41, which pertain to the Willamette Basin. Bear Creek and the Molalla River are potential salmonid passage/rearing waters, and the following basin standards apply to both:

- During periods of low stream flows (May through October), treatment must result in monthly average effluent 5-day biochemical oxygen demand (BOD) and total suspended solids (TSS) concentrations below 10 milligrams per liter (mg/l). During periods of high stream flows (November through April), a minimum of secondary treatment, or equivalent treatment, must be used.

- Effluent BOD concentrations (in mg/l), divided by the dilution factor (the ratio of receiving stream flow to effluent flow) shall not exceed 1 mg/l unless specifically approved by the EQC.
- Effluent must be disinfected after treatment with sufficient chlorine to provide a residual of 1 mg/l after 60 minutes of contact time.
- Discharge may not lower dissolved oxygen concentration in the stream below 8.0 mg/l as an absolute minimum. Where conditions of barometric pressure, altitude, and temperature preclude attainment of the 8.0 mg/l, dissolved oxygen shall not be less than 90 percent of saturation.
- No measurable surface water temperature increase resulting from anthropogenic activities is allowed in the following cases:
  - Where surface water temperatures exceed 64.0°F
  - Where dissolved oxygen levels are within 0.5 mg/l or 10 percent saturation of the dissolved oxygen criterion for the given stream.
- Discharge may not increase turbidity of the stream by more than 10 percent.
- Discharge may not cause the pH of the stream to go outside the range of 6.5 to 8.5.
- Effluent bacteria requirements will be based on the Escherichia coli (*E. coli*) standard and shall meet the following criteria:
  - The 30-day log mean shall not exceed 126 *E. coli* organisms per 100 milliliter (ml), based on a minimum of five samples.
  - No single sample shall exceed 406 *E. coli* organisms per 100 ml.
- Total dissolved solids shall not exceed 100 mg/l.
- Discharge cannot raise the level of toxic substances in the stream above the criteria listed in Table 20 of OAR Chapter 340 Division 41.

OAR 340-41 also regulates the development of fungi or other growths, the creation of tastes or odors, the formation of appreciable bottom deposits, discoloration and floating solids, offensive aesthetic conditions, and other extreme deleterious effects on the receiving water.

DEQ has designated both the Molalla River and the Pudding River (into which Bear Creek flows) on the 303(d) list of water quality limited streams. As a result, more stringent regulations than the basin standards described above may apply. The portions of these rivers so listed are shown in Table 4-1, with approximate river mile (R.M.) locations and the water quality parameters and associated seasons of concern. The locations of these rivers relative to the City of Molalla are shown graphically in Figure 4-1.



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City of Molalla, Oregon
WASTEWATER FACILITIES PLAN

FIGURE 4-1
RIVER LOCATION MAP

283010/USGSmap.fh8

TABLE 4-1.
303(d) LISTINGS FOR THE MOLALLA RIVER AND THE PUDDING RIVER

River	Water Body Boundaries	Parameter	Season
Molalla	Mouth to North Fork Molalla (R.M. 26)	Flow Modification Bacteria Temperature	 Fall, Winter, Spring Summer
	North Fork Molalla (R.M. 26) to Headwaters	Temperature	Summer
Pudding	Mouth to Little Pudding River (R.M. 36)	Bacteria Temperature Toxics	Year-Round Summer

NPDES Permit

An NPDES permit defines limits on BOD and TSS concentrations and mass loads, removal requirements, and criteria for pH levels and bacteria counts. It also regulates the discharge mixing zone and any other restrictions on the discharge. An NPDES also includes monitoring and reporting requirements and other applicable conditions. The City's current NPDES permit is included in Appendix F and Table 4-2 summarizes limitations defined for the discharge to Bear Creek, Outfall Number 001 (permit number 101514; expires November 30, 2002). The NPDES permit also states that the mixing zone shall not exceed beyond 50 feet downstream from the point of discharge.

TABLE 4-2.
CURRENT NPDES PERMIT DISCHARGE LIMITATIONS

Parameters	Limitations				
	Max Concentration (mg/l)		Max Mass Load (ppd)[a]		
	Monthly Average	Weekly Average	Monthly Average	Weekly Average	Daily
BOD	10	15	66	99	132
TSS	10	15	66	99	132
Dilution	Effluent BOD concentration in mg/l, divided by the dilution factor (ratio of receiving stream flow to effluent flow) shall not exceed 1 mg/l.				
pH	Shall be within the range of 6.0 - 9.0 standard units.				
E. coli	30-day log mean of 126 organisms per 100 milliliter (ml) and no single sample shall exceed 406 organisms per 100 ml. If a single sample exceeds 406 organisms per 100 ml, additional sampling per Schedule B is mandated.				
BOD and TSS removal	Shall not be less than 85 percent monthly average				

a. Based on an average dry weather design flow equaling 0.79 mgd.

4.2.2 Land Application

Reuse of effluent by land application is governed by OAR 340-55, *Regulations Pertaining to the Use of Reclaimed Water (Treated Effluent) from Sewage Treatment Plants*. Groundwater quality is governed by OAR 340-40, *Groundwater Quality Protection*. Land application of treatment plant effluent requires a Water Pollution Control Facility permit from the DEQ; however, Molalla's land application is permitted through the City's NPDES permit. The permit refers to the land application process as "Outfall Number 002, Reuse."

The land application requirements outlined in OAR 340-55 depend on the level of treatment provided and type of land on which the reused effluent is applied. Table 4-3 summarizes the treatment, monitoring, and application requirements for reuse of effluent for various reuse levels. Level I reuse has the least stringent treatment and monitoring requirements and therefore the most restrictive application requirements. Level IV reuse has the most stringent treatment and monitoring requirements and the least restrictive application requirements.

The City's current NPDES permit requirements for land application are as follows:

- All wastewater distributed on land during dry weather (typically between June 1 and October 31) shall dissipate by evapotranspiration and controlled seepage using sound irrigation practices so as to prevent:
  - Prolonged ponding of treated wastewater on the ground surface
  - Surface runoff or subsurface drainage through piping
  - The creation of odors, favorable conditions for fly and mosquito breeding or other nuisances
  - The overloading of the land with nutrients, organic materials, or other pollutant parameters.
- Prior to land application of the reclaimed water, it shall receive at least Level II treatment as defined in OAR 340-55 and accordingly, total coliform is limited to a 7-day median of 23 organisms per 100 ml with no two consecutive samples exceeding 240 organisms per 100 ml.

4.3 SLUDGE MANAGEMENT REGULATIONS

Sludge is a mixture of solids and liquids that is an end-product of wastewater treatment. It is commonly disposed of by applying it to agricultural or forest land after processing. Adequately processed sludge is classified in regulations as biosolids. The Code of Federal Regulations (40 CFR 503) defines standards for three measures of biosolids quality:

- Pathogens.
- Vector attraction (the tendency of the sludge to attract rodents, insects and other organisms that can spread disease).
- Trace elements.

TABLE 4-3.
TREATMENT, MONITORING, AND APPLICATION REQUIREMENTS FOR REUSE

Parameter	Level I	Level II	Level III	Level IV
Required Treatment				
Biological Treatment	X	X	X	X
Disinfection		X	X	X
Clarification				X
Coagulation/Filtration				X
Total Coliform				
Two Consecutive		240		
7-day Median		23	2.2	2.2
Maximum			23	23
Sampling Frequency		1 per week	3 per week	1 per day
Turbidity (NTU)				
24-hour Mean				2
5% of Time				5
Sampling Frequency				Hourly
Public Access	Prevented (fences, gates, locks)	Controlled (signs, rural or nonpublic lands)	Controlled (signs, rural or nonpublic lands)	No direct public contact irrigation cycle
Buffers for Irrigation	Surface: 10 ft. Spray: site specific.	Surface: 10 ft. Spray: 70 ft.	10 ft.	None required.
Agricultural				
Food Crops	Not Allowed	Not Allowed	Not Allowed	Unrestricted
Commercial Timber	Restricted	Restricted	Restricted	Unrestricted
Other	Not Allowed	Restricted	Restricted	Unrestricted
Parks, Playgrounds, Schoolyards, Golf Courses w/ contiguous residences	Not Allowed	Not Allowed	Not Allowed	Restricted
Golf Courses w/o contiguous residences, Cemeteries, Highway Medians, Landscapes w/o Frequent Public Access	Not Allowed	Restricted	Restricted	Restricted
Industrial or Commercial Use, Construction Use	Not Allowed	Restricted	Restricted	Restricted
Impoundments				
Unrestricted	Not Allowed	Not Allowed	Not Allowed	Restricted
Restricted	Not Allowed	Not Allowed	Restricted	Restricted
Landscape	Not Allowed	Restricted	Restricted	Restricted

Treated sludge (biosolids) that meets the higher of two standards for all three elements is designated exceptional quality (EQ) biosolids. EQ biosolids have fewer reporting and monitoring requirements and virtually no restrictions on use. Use is restricted for biosolids that do not meet the higher standard for any of the three elements.

4.3.1 Pathogen Requirements

Pathogen requirements define two classes of biosolids—Class A and Class B. Class A is the higher standard and requires complete destruction of pathogens before disposal. Class B requirements call for reducing pathogens before disposal and applying the biosolids to land in such a way that pathogens are further reduced.

To be classified as Class A, biosolids must be treated using one of the EPA's *processes to further reduce pathogens* (PFRP), or an equivalent process. These processes include composting, heat drying, heat treatment, thermophilic aerobic digestion, beta ray irradiation, gamma ray irradiation, and pasteurization. Regardless of the process used, Class A biosolids must not exceed maximum allowable fecal coliform density or salmonella bacteria density.

Class B biosolids must be treated using one of the EPA's *processes to significantly reduce pathogens* (PSRP), or an equivalent process. These processes include aerobic digestion, air drying, anaerobic digestion, composting, and lime stabilization.

4.3.2 Vector Attraction Requirements

Biosolids must meet one of the following requirements for reducing vector attraction if they are to be applied to land without restrictions:

- Volatile solids in the sludge shall be reduced by a minimum of 38 percent.
- The specific oxygen uptake rate for sludge treated by aerobic digestion shall be less than or equal to 1.5 mg oxygen per hour per gram of total solids at a temperature of 20°C.
- Aerobic processes shall treat the sludge for a minimum of 14 days with an average temperature of at least 45°C and a minimum temperature of 40°C.
- Alkali addition shall raise the pH of the sludge to a minimum of 12 for two hours and maintain the pH at a minimum of 11.5 for an additional 22 hours without additional alkali.

The use of the land where the biosolids is applied is restricted if vector attraction reduction is achieved by measures such as injecting the biosolids below the surface of the land or disposing of them on the surface and incorporating them into the soil within six hours.

4.3.3 Trace Elements

Ten elements typically found in biosolids have been identified as critical. Two limits have been set for each of these trace elements: EQ and a ceiling limit. If all the trace elements are below the EQ limit, then no restrictions are placed on loading rates. If any of the trace

elements are over the ceiling limit, then the biosolids are not suitable for land application. If the trace elements fall between these two limits, restrictions are placed on loading rates.

4.3.4 Use Restrictions Based on Quality

Restrictions on the use of biosolids depend on the quality of the biosolids as follows:

- **EQ rating for trace elements, vector attraction and pathogens**: No restrictions are imposed on application or use with regard to pathogens, vector attraction, or trace elements.
- **EQ rating for trace elements and vector attraction; Class B rating for pathogens**: Application is subject to EPA-defined waiting periods for crops, grazing, and public access. Biosolids cannot be distributed for home use, in bags, or in containers.
- **EQ rating for trace elements and pathogens but not for vector attraction**: Biosolids must be injected or tilled into the soil and cannot be distributed for home use, in bags, or in containers.
- **EQ rating for vector attraction and pathogens but not for trace elements**: Bulk application must not exceed EPA-defined cumulative loading rates. Biosolids distributed in bags or containers are subject to annual loading rate restrictions.
- **All other biosolids qualities**: Application is subject to trace loading requirements and pathogen waiting periods. Biosolids must be injected or tilled into the soil and cannot be distributed for home use, in bags, or in containers.

4.4 SEPTAGE MANAGEMENT REGULATIONS

If septic tanks are used to remove solids from wastewater flows, the septage sludge must be disposed of at a facility designed to treat septage. The 40 CFR 257 regulations outline the rules pertaining to the treatment and disposal of industrial/commercial/domestic septage.

Treatment and application must meet the same pathogen reduction standards for septage sludge as for sewage sludge. The use of septage sludge treated by a PFRP is not restricted; septage sludge treated by a PSRP is subject to waiting periods for crops, grazing, and public access. The regulations also require that the on-site population of disease vectors be minimized through the periodic application of cover material. Application to land used for crop production is restricted with regard to cumulative applications of cadmium and the concentrations of polychlorinated biphenyls (PCBs) in the septage sludge.

In the past, the City has accepted septage at a receiving station at the existing headworks, but the operators report that it has been approximately six years since septage has been received. DEQ states that sewage treatment plants are not allowed to accept septage without having special equipment and tankage design for septage receiving and handling. It was assumed that the existing septage receiving station does not meet current standards and that the City will not update these facilities to receive septage.

4.5 ON-SITE SEWAGE DISPOSAL

On-site sewage disposal systems are regulated by the DEQ and are governed by OAR 340-71 and 340-72, *On-Site Sewage Disposal*. These regulations outline minimum separation distances from items requiring setbacks, effective soil depths for different drainfield slopes, and minimum drainfield lengths with respect to soil type and depth.

CHAPTER 5.
EXISTING FACILITIES AND CONDITIONS

Prior to 1955, sewage collection and treatment in Molalla were primarily provided by individual septic tanks. In 1955, the City expanded its limited sewer system with construction of a piped gravity collection system and a treatment plant located south of Toliver Road approximately a half mile east of Highway 213. Since then, the collection system has been further upgraded and expanded. The original plant was abandoned and replaced with the current plant in 1980.

The wastewater system currently includes a gravity collection system with two small pump stations. The current lagoon treatment plant includes an aerated lagoon, a transfer pump station, two facultative treatment/storage lagoons, advanced treatment by dissolved air flotation and gravity filters, disinfection by chlorination, discharge of the treated effluent to Bear Creek during winter months, and land application of effluent during summer months. This chapter describes the existing condition of each of these wastewater system components and summarizes the historical performance of the treatment plant.

5.1 EXISTING COLLECTION SYSTEM

Most of the collection system consists of standard gravity sewers. The system has four small collection system pump stations. The collection system is shown on Figure 5-1 and a spreadsheet inventory of the trunk lines, including pipe capacities, is in Appendix B.

5.1.1 Gravity Sewer System

System Background

Prior to 1955, the City sewer system had approximately 3,700 feet of 6-inch and 8-inch piping and 10 manholes. The pre-1955 sewer lines were open-jointed concrete pipe. In 1955, an additional 47,200 feet of 8-inch to 15-inch piping was installed. Sewers constructed in 1955 are concrete, asbestos cement (AC) and polyvinyl chloride (PVC) piping. The main trunk sewer to the original treatment plant is a 15-inch line primarily along Toliver Road. In the existing sewer system, approximately 50,900 feet of sewers are now over 40 years old.

When the existing treatment plant was built in 1980 to replace the original plant, the main trunk sewer was extended west along Toliver Road and south to the plant. This trunk is mostly 12-inch and 15-inch piping with a final section of 21-inch piping. In 1988, the 15-inch Bear Creek sewer interceptor was constructed from the upstream end of the 21-inch trunk line southeast along Bear Creek. This interceptor was extended in 1995 with a 12-line trunk from Bear Creek east along Highway 211 to Molalla Avenue. The sewer system south of Highway 211 and west of Grange Street was intercepted and routed along this trunk. The remaining system continues to flow to the original trunk in Toliver Road. Other improvements since 1955 include the addition of 8-inch sewers to serve new subdivisions.

During the second half of the 1970s and in the early 1980s, the City invested considerable effort and money in removing excessive infiltration and inflow (I/I) from the collection system. The work included video inspecting, testing, grouting, replacing services and some 8-inch collection lines, and repairing manholes. The City believes that this sewer work had some success in lowering wet-weather flows. However, City staff has reported grout material entering the wastewater treatment plant from the collection system. It is unknown how much of the grout has been washed away, or how effective the remaining grout is.

Existing Conditions

The current collection system has approximately 100,000 feet of piping and over 250 manholes. Most of the gravity piping ranges from 8-inch to 15-inch; the trunk line just upstream of the treatment plant that collects flows from the entire City is 21-inch.

The collection system in general is aging and hydraulically overloaded. City staff also report that they are concerned about the quality of construction of the recently constructed piping along Highway 211 from Bear Creek east to Molalla Avenue. Excessive infiltration has been observed entering through cracks in the base of several manholes along this trunk extension. This line has not been video-inspected.

Diversions

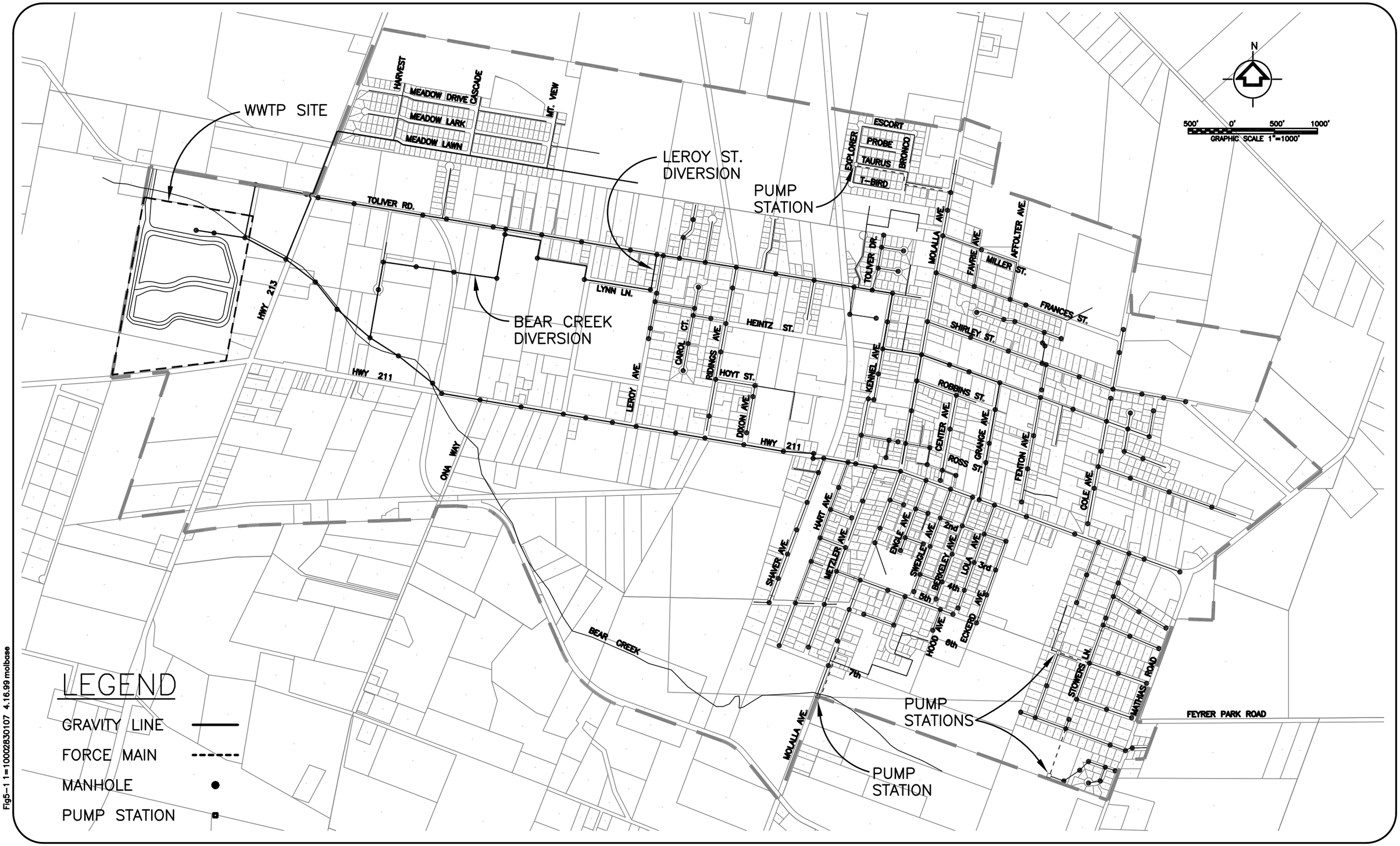
Evaluation of the existing collection system is complicated by two diversion points in the system. These diversions are shown on Figure 5-1.

The first diversion point is at Toliver Road and Leroy Avenue, where flows from the trunk along Toliver Road are split to the west and the south. This diversion manhole's two outlet pipes have the same flow-line elevation. The flow split to the west continues in the trunk along Toliver Road. The diversion to the south turns west approximately 500 feet south of Toliver Road, then turns north to just south of Toliver Road, then turns west and runs parallel to Toliver Road to a manhole approximately 1,800 feet west of Leroy Avenue. This manhole is the point of the second diversion.

The second diversion point splits flows to the north and to the south. This diversion manhole has the same flow-line elevation for its two outlet pipes and both pipes are 15-inch. The flow split to the north enters the trunk along Toliver Road. The flow split to the south flows along the Toliver/Bear Creek Sanitary Sewer Diversion constructed in 1991, which discharges into the Bear Creek interceptor.

5.1.2 Collection System Pump Stations and Force Mains

The locations of the four public collection system pump stations and their discharge force mains are shown on Figure 5-1. One pump Station is on Explorer west of Molalla Avenue in the north end of the City; another pump station is on Molalla Avenue south of 7th Street; the other two pump station are in the vicinity of Stowers Lane south of Highway 211. Each pump station serves a very small area and will not serve significantly more future users. As a result, these stations were not evaluated as part of this plan.



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City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 5-1

EXISTING WASTEWATER FACILITES LOCATION MAP

5.1.3 Infiltration and Inflow

Previous efforts to remove I/I from the collection system are reported to have reduced flows, but recent flow data for the overall collection system indicate that a significant amount of I/I remains in the system (see Chapter 3). Additionally, the plant operators report that significant amounts of grout have been collected at the plant headworks, indicating the previous grout work has not been entirely effective. Flow monitoring was performed early in 1999 in an attempt to identify areas of excessive I/I. The results of this monitoring are discussed in Chapter 6.

5.1.4 Summary of Collection System Deficiencies

The following are the major deficiencies identified in the existing collection system:

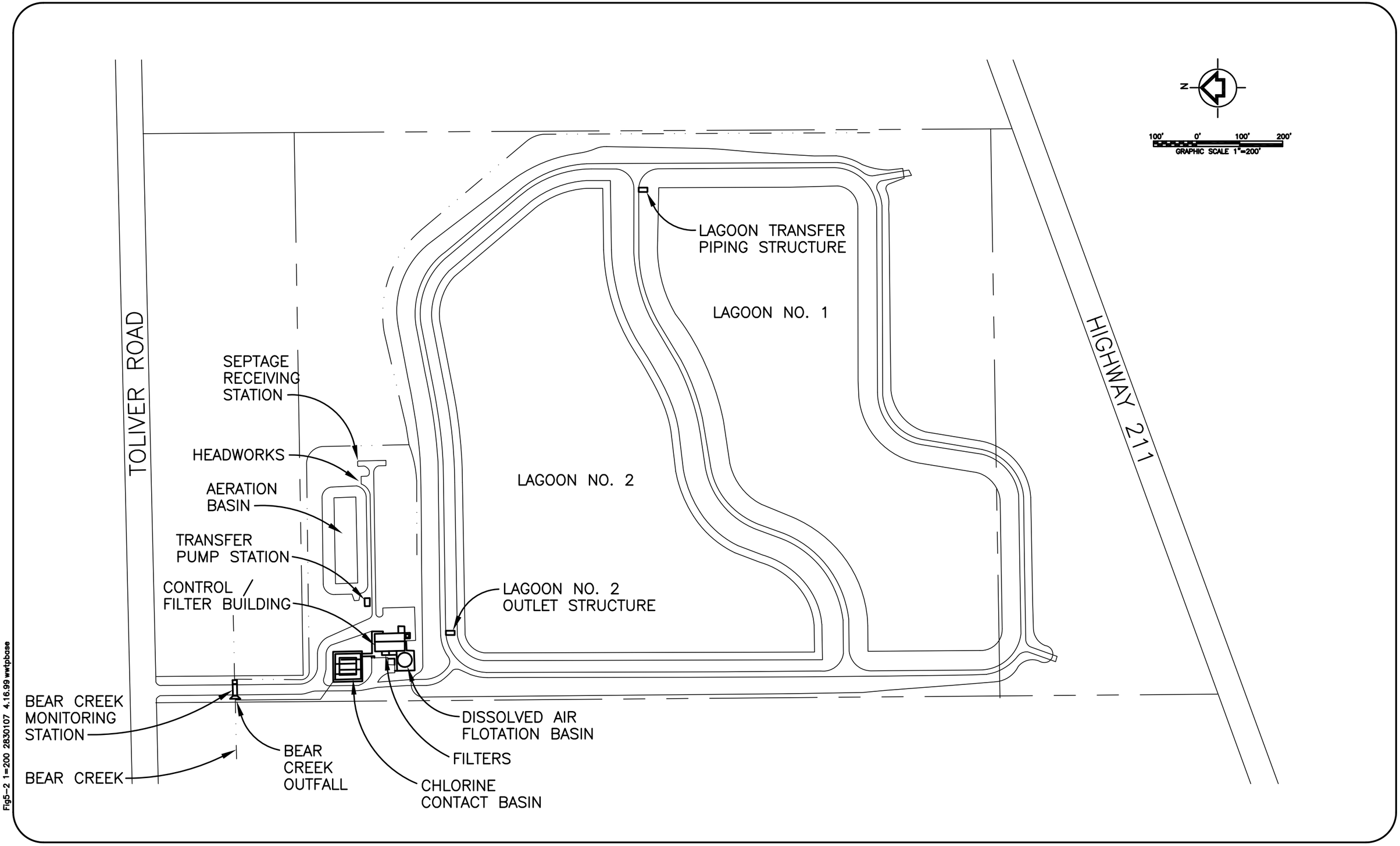
- High I/I indicates problems in the integrity of the collection system piping. Over half of the collection system is over 40 years old.
- There is approximately 3,700 feet of open joint pipe.
- A significant portion of the previous I/I reduction work (grouting) work has exceeded its useful life and has failed.
- Facilities are generally hydraulically overloaded during winter storms, which causes I/I related surcharging.

5.2 EXISTING TREATMENT FACILITIES

The City's wastewater treatment plant is in the west end of the City just south of Toliver Road, as shown on Figure 5-1. The plant was constructed in 1980, replacing the original treatment plant that had been built in 1955 approximately half a mile east of the site of the existing plant. A plan of the existing treatment plant is shown on Figure 5-2.

The plant has a headworks, which includes comminution (grinding) and flow measurement using a Parshall flume. Influent flows by gravity from the headworks to an aeration basin. A pump station is required to transfer the wastewater from the aeration basin to the first of two facultative lagoons, which provide both treatment and storage. Disinfection is accomplished using gaseous chlorine. Dry-weather effluent is disposed of by land application. Excess dry-weather effluent is stored in the lagoons. Wet-weather flows and stored effluent are further treated using dissolved air floatation (DAF) and gravity filters prior to a stage-based surface water discharge to Bear Creek.

Table 5-2 summarizes design data for the plant's components. A hydraulic profile and a flow schematic of the existing plant are shown on Figure 5-3.



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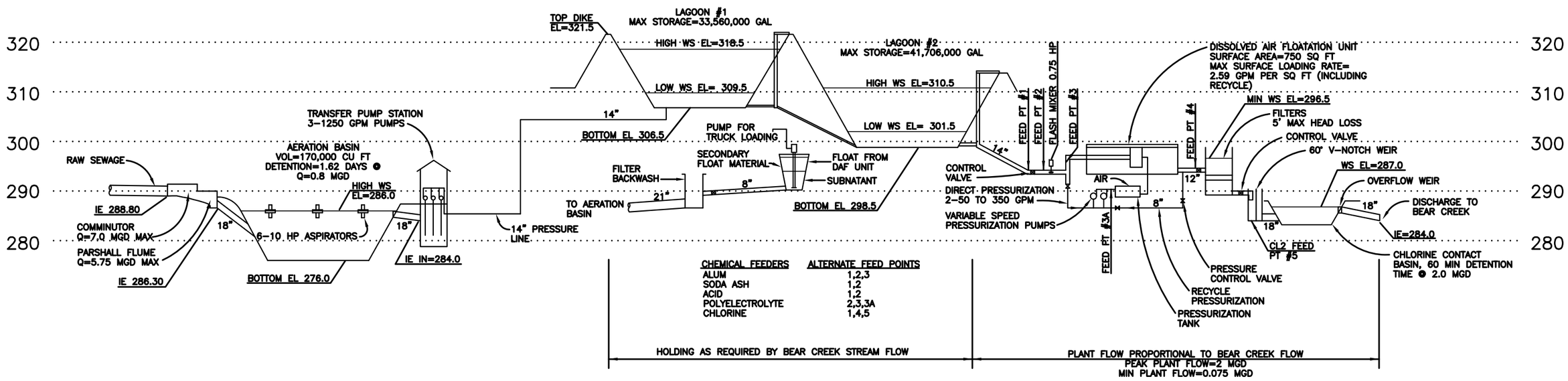
WASTEWATER FACILITIES PLAN

FIGURE 5-2
EXISITING WWTP
SITE PLAN



PROFILE AS PRESENTED ON SHEET 1 OF 43 OF MAY, 1977 SEWAGE TREATMENT PLANT DRAWINGS PREPARED BY WESTECH ENGINEERING, INC.

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WASTEWATER FACILITIES PLAN

FIGURE 5-3

EXISTING WWTP HYDRAULIC PROFILE AND FLOW SCHEMATIC

TABLE 5-1.
EXISTING TREATMENT PLANT FACILITIES DESIGN DATA

Item	Description / Design Data	
Headworks	Influent Line	
	Size	21-inch
	Full Flow Capacity	5.25 mgd
	Comminutor	
	Number	2
	Maximum Capacity	3.5 mgd each
	Bypass	Bar screen – manually cleaned
	Parshall Flume	
	Size	9-inch
	Maximum Capacity	5.75 mgd
	Influent Sampler	Time-composite (to be replaced with flow-paced)
Aeration Basin	Basin Dimensions	
	Size (bottom of basin)	200 feet by 54 feet
	Side Slopes (horiz:vert)	2:1
	Maximum Side Water Depth	10 feet with 2 feet freeboard
	Basin Volume, Maximum	175,000 cubic feet
	Basin Liner	Asphalt-concrete
	Aerators	
	Type	Aspirating
	Number	6
	Horsepower, each	10 horsepower (hp)
	Basin Outlet	Overflow weir to pump station
Pump Station	Pumps	Constant speed, vacuum priming centrifugal
	Type	
	Number	3
	Horsepower, each	25 horsepower
	Capacity, each	1,250 gallons per minute (gpm) at 56 feet total dynamic head (TDH)
	Maximum Firm Capacity	2,400 gpm using two pumps
	Discharge Force Main	
	Size	14-inch
	Length	1,615 feet
Lagoon No. 1	Lagoon Dimensions	
	Surface Area	11.4 acres (at 6-foot (average) depth)
	Maximum Depth	12 feet with 3 feet of freeboard
	Working Depth (Max. to Min.)	9 feet
	Lagoon Volume, Maximum	137 acre-feet
	Lagoon Liner	Native clay
	Aeration	None (three 10 hp aerators originally installed have been removed)
	Outlet	
	Size	10-inch
	Type	Surface weir and fixed pipe on bottom of lagoon

TABLE 5-1 (continued).
EXISTING TREATMENT PLANT FACILITIES DESIGN DATA

Item	Description / Design Data	
Lagoon No. 2	Lagoon Dimensions	
	Surface Area	13.6 acres (at 6-foot (average) depth)
	Maximum Depth	12 feet with 3 feet of freeboard
	Working Depth	9 feet
	Lagoon Volume, Maximum	163 acre-feet
	Lagoon Liner	Native Clay
	Outlet	
	Size	14-inch
	Type	Fixed pipes at two depths
Dissolved Air Flotation	Capacity	
	Surface Area	750 square feet (sf)
	Maximum Surface Loading Rate	2.59 gpm/sf, including recycle
	Hydraulic Capacity	2.80 mgd, including recycle
	Chemical Feed Rates	
	Alum	75 – 150 mg/l
	Soda	37 – 75 mg/l
	Polymer	0.5 – 10 mg/l
	Acid	0 - 10 mg/l
	Operating Parameters	
	Pressurized Recycle Flow	350 to 700 gpm
	Operating Pressure	45 to 80 pounds per square inch (psi)
	Solids to Air Ratio	0.03
	Maximum Horizontal Velocity	3.1 feet per second
	Maximum Daily Sludge	2290 pounds dry solids, 15,300 gallons
Gravity Filters	Capacity	
	Number of Filters	2
	Surface Area, Total	310 square feet
	Maximum Loading Rate	5 gpm/sf
	Hydraulic Capacity	2.23 mgd
	Media	
	Type	Gravel, sand, and anthracite coal
	Depth	22” gravel, 9” sand, 21” coal
	Backwash / Surface Wash	
	Type	Automatic on timer or pressure differential
	Backwash Rate	20 gpm/sf
	Surface Wash Rate	103 gpm
Disinfection	Type	150-pound gas cylinders
	Dosage	
	Chlorinator Capacity	100 ppd
	Feed Rate, Minimum	30 to 50 pounds / million gallons
	Residual, Minimum	2 mg/l
	Chlorine Contact	
	Sidewater Depth	4 feet with 1 foot of freeboard
	Volume	67,500 gallons
	Length to Width Ratio	24

TABLE 5-1 (continued).
EXISTING TREATMENT PLANT FACILITIES DESIGN DATA

Outfall	Size and Material	18-inch concrete
	Length	270 feet
	Diffuser	
	Number of Ports	1
	Type	Discharge parallel with horizontal plane and roughly perpendicular to streamflow
	Stream Gauge	
	Type	Box Culvert with 10' weir
	Measurement	Float Gage in stilling well adjacent to box culvert (currently not operable)
Irrigation System	Irrigation Pump (Adjacent Land)	
	Type of Pump	Portable Dewatering
	Number of Pumps	1
	Capacity	250 gpm at 300 feet TDH
	Horsepower	50 horsepower
	Irrigation Pump (Plant Site)	
	Type of Pump	Vertical Turbine
	Number of Pumps	1
	Capacity	530 gpm at 175 feet TDH
	Horsepower	30 horsepower
	Sprinkler Type	Fixed Impact (on site)
	Irrigation Area	
	Plant Site	~70 acres
	Adjacent Site	~15 acres
Standby Power	Generator Set	
	Type	Natural Gas
	Size	300 amps and 600 volts
	Transfer Switch	Automatic
	Facilities Served	Headworks, Aeration Basin, Transfer Pump Station
Alarm Telemetry	Alarm Conditions	High water level in filters, DAF arm out of alignment, air compressor stops operating, power outage

5.2.1 Headworks

The headworks consists of two comminutors, a manually cleaned bar screen, influent flow measurement (Parshall flume), and influent sampling.

Influent Sewer

The sewer line entering the plant has a full flow gravity capacity of 5.25 mgd, but under surcharged flow it is capable of delivering higher flows to the plant. In fact, during the storm of February 1996, the influent sewer flooded the entire headworks and aeration lagoon area. High influent flows exceeded the capacity of the Parshall flume, and the entire flume was completely topped by 3 or more inches of water for a 24-hour period. The pump

station was sand bagged to prevent up to 2 feet of water from damaging the controls and equipment in the pump station. The pump station operated at full capacity but could not prevent flooding. An opening was cut through the dike separating the aeration basin from Bear Creek to allow the ponding wastewater to bypass to the creek.

Comminution and Screening

The influent passes through one Muffin Monster comminutor, one standard comminutor, and one emergency manually cleaned bar screen (three parallel channels). The design capacity of each comminutor is 3.5 mgd.

Grit Removal

The headworks has no grit removal capability. Grit passes through the headworks into the aeration basin. It accumulates there and must be periodically dredged.

Influent Flow Measurement

Influent flow measurement is accomplished with a flume and a bubbler flow meter. The flume is a 9-inch standard Parshall flume with a capacity from 0.06 to 5.75 mgd. The influent flow typically exceeds the capacity of the flume each winter.

As discussed in Chapter 3, the configuration of the channel upstream of the flume is such that flow measurements made at the flume are not fully reliable. For accurate measurements with a Parshall flume, the approaching flow should be relatively free of turbulence. At the Molalla treatment plant, there is a steep slope in the channel from the comminutors and bar screen just upstream of the flume. This slope, in combination with the convergence of three channels just upstream of the slope, creates considerable turbulence at the upstream end of the flume. The City determined that influent flows less than 2.5 mgd are measured accurately at the flume, but influent flows greater than 2.5 mgd are measured approximately 10 to 15 percent high at the flume.

Influent Sampling

The influent sampler is a portable ISCO Model 3710FR. Composite samples are drawn downstream of the Parshall flume on a time-composite basis. The unit is only two years old and is in good condition.

At the end of 1999, the effluent sampler stopped working due primarily to long operational life. The plant operator purchased a new sampler similar to the influent sampler, but with flow-paced operation. The operator plans to replace the existing influent sampler with the new sampler and move the existing influent sampler to sample the effluent flows since effluent flows are more consistent than influent flows. As such, the influent flows will be sampled on a flow-paced basis.

Septage

The headworks has provisions to receive septage upstream of the flow meter. According to the operators, the plant has not accepted septage for about six years. The design septage load is unknown since there is no design data on the plans for the plant. The septage

receiving station consists of a manhole for the septage haulers to dump into. There are no provisions for measurement, sampling, holding or equalization. It is assumed for this plan that the plant will not accept septage.

5.2.2 Aeration Basin

The aeration basin is just downstream of the headworks. The basin is referred to on the plans for the plant as an aeration basin, but it is actually an aerated lagoon since activated sludge is not returned to the basin. The mixed liquor concentration is similar to the strength of the influent sewage. The influent is aerated prior to being pumped up to the two facultative lagoons.

A splitter box downstream of the flume diverts flows through an 18-inch line into the aeration basin and has provisions to split the flow between the existing basin and a future parallel basin. The second basin would require excavation at the toe of the north dike of Lagoon No. 2. Based on the type of vegetation on the exterior of the north dike of Lagoon No. 2, there may be seepage through the dike. This could affect the City's ability to construct a new aeration basin in this location.

Basin Sizing

The single-cell aeration basin is lined with asphalt-concrete. The side slopes are 2 to 1 (horizontal to vertical), and the bottom dimensions are 200 feet by 54 feet. The basin dikes are 12 feet high, for top dimensions of 248 feet by 102 feet. The basin is designed to operate at a 10-foot depth with 2 feet of freeboard. The resulting volume of the basin is 1.3 million gallons. The design hydraulic retention time in the basin is 1.6 days at the plant average design flow of 0.8 mgd.

Aeration

The basin was originally fitted with three 40-horsepower floating updraft centrifugal aerators. Due to high operation and maintenance costs, they were replaced in 1996 with six 10-horsepower aspirating aerators. Provisions were made for the addition of more aerators as needed. The new aerators have reduced the amount of accumulated sludge in the basin as a result of improved mixing and provide improved aeration. Sludge and grit deposits that had accumulated in the basin were suspended by the new aerators and pumped to Facultative Lagoon No. 1.

An aerator service derrick is provided to remove the aerators from the lagoon for cleaning. It has not been used because the guy wires of the other aerators block access for a service vehicle to reach the derrick and therefore the aerator. The operators use a backhoe to lift aerators from the lagoon for servicing.

Grit Removal

Grit was removed from the aeration basin in 1981, shortly after the plant went into service. With the installation of the new aspirating aerators, the majority of the grit has been suspended and pumped to Lagoon No. 1, returning most of the available volume of the basin for treatment.

The aeration basin was provided with a floating dredge to ease cleaning. The operators find it impractical to use because the single small suction pipe removes grit from only a small area at a time. It is also very difficult to move the dredge past the aerator mooring lines.

Because there is only one aeration basin, maintenance requires the basin to be removed from service and raw sewage to be diverted directly into the facultative lagoons. This upsets lagoon operation by decreasing dissolved oxygen (DO) in the lagoons. This reduces lagoon effluent quality for a period of several weeks.

5.2.3 Transfer Pump Station

The transfer pump station located at the southwest edge of the aeration basin lifts the aerated influent from the aeration basin to Lagoon No. 1.

Pumps

The station has three vacuum primed, constant-speed, centrifugal pumps. Each is a 25-horsepower Cornell pump with GE motor. The station is designed to be operated with one or two pumps matching influent flow and the third pump providing redundancy; however, it is possible to operate all three pumps when needed. The pumps are activated by mercury floats located in the wet well.

The pump capacity is not adequate for existing peak wet-weather influent flows. The current capacity of the pump station is approximately 2,400 gpm, or 3.5 mgd, with two pumps running. Winter flows typically exceed the capacity of the pumps several times a year. The operators use the dredge pump and an engine-driven trash pump to pump excess flows to Lagoon No. 1 through above-ground aluminum force mains during peak flow conditions.

The vacuum priming system fails regularly and is in constant need of repair.

Wet Well

A rectangular concrete wet well lies below the pumps. The wet well is 10 feet by 11.3 feet and has an operating depth range of 4 feet, for a wet well volume of 3,000 gallons.

Force Main

The pump station discharge force main is a 14-inch main. The force main transports the flows to the east end of Lagoon No. 1. At the current capacity of the pump station, the velocity in the force main is 5 fps.

Building

The pumps are in an at-grade concrete block building located above the wet well. The plant's standby generator is also in this building. The standby generator is described separately below.

5.2.4 Facultative / Storage Lagoons

The facultative/storage lagoons are south of the rest of the treatment plant facilities. The lagoons are operated in series. Lagoon No. 1 is the upstream lagoon and Lagoon No. 2 is the downstream lagoon and the furthest north. The lagoons provide additional treatment, flow equalization, sludge treatment/storage and effluent storage. The lagoons are lined with native clay.

Lagoon Sizing

Both lagoons are approximately 12 feet deep. At average water depth, Lagoon No. 1 is approximately 11.4 acres and Lagoon No. 2 is approximately 13.6 acres, for a total average lagoon area of 25.0 acres. This provides a total storage volume of approximately 300 acre-feet, or 98 million gallons.

Lagoon Operation

During the winter, flows are stored in the lagoons and released to the advanced treatment system at a rate within the capacity of the filters. During the summer, most of the flows are stored in the lagoons for eventual advanced treatment and discharge to the river in the winter. (Some summer flows are irrigated on the plant site and nearby private farmland.)

Appendix E includes a water balance evaluation of the lagoons and effluent disposal facilities for existing influent flows. The water balance shows that the existing storage and disposal system is barely adequate for existing flows.

Lagoon Aeration

Lagoon No. 1 was originally constructed with three 10-horsepower aerators at the upstream (east) end of the lagoon. After operating the aerators for a short period after plant construction, the operators found that use of the aerators was not needed to meet discharge permit requirements and seemed to have little if any effect on the lagoon effluent quality. The aerators were subsequently removed.

Sludge Accumulation

Lagoon No. 1 has a single pipe inlet near the bank, which resulted in a large pile of solids near the inlet. This lagoon had to be kept at a depth of over 6 feet to prevent the water level from dropping below the top of the sludge, exposing it to the air and resulting in both hardening and odors. This condition reduced the amount of available storage in Lagoon No. 1 by nearly one-third.

The City tried to dredge Lagoon No. 1 in 1991 using the small dredge in the aeration basin. The dredge proved ineffective due to its small size and poor intake design. The area that was dredged quickly refilled with sludge. In the fall of 1999, the City contracted out the dredging of the sludge accumulation. The dredging operation has caused some treatment difficulties likely due to the suspension of solids, but is expected to improve treatment performance in the long-term.

Lagoon Piping

Effluent is transferred from Lagoon No. 1 to Lagoon No. 2 through a level control structure. The normal mode of operation is to withdraw effluent from the surface of the lagoon using an overflow weir. The overflow elevation can be adjusted by adding or removing wooden stop logs. The maximum capacity of this facility is 5 mgd. A low level drain is also provided, which the operators occasionally use to discharge from Lagoon No. 1 to Lagoon No. 2. The operators would prefer a structure designed to allow them to draw effluent from multiple levels.

An outlet structure is located at the east end of Lagoon No. 2. Effluent may be withdrawn from either of two effluent pipes that extend horizontally from the tower through the sloped bank of the lagoon. The normal mode of operation is to withdraw effluent from Lagoon No. 2 at a point approximately 5 feet above the bottom of the lagoon. The second pipe is at the lagoon bottom elevation. The valving in the outlet structure is awkward and seldom used because the rate of withdrawal can also be controlled by manually adjusting the valving at the filter building. The operators would prefer that the structure be modified to allow them to draw from multiple levels in Lagoon No. 2 to gain the best lagoon effluent.

5.2.5 Dissolved Air Flotation

The gravity filters are preceded by a dissolved air flotation (DAF) tank. The primary purpose of the DAF unit is the removal of algae. Compressed air is supplied from one pressure vessel. Two pumps feed wastewater into the pressure vessel. The solids collected in the DAF tank are discharged by gravity to the aeration basin.

The system was originally designed to use alum, polymer, soda, and acid feeds, but currently only alum is used. Alum, used for coagulation of the effluent, is fed into the lagoon effluent flow by an in-pipe injector positioned upstream of the pressure vessel. The alum is fed at a range of rates from 35 to 70 mg/l.

5.2.6 Gravity Filters

The filters are used only during the winter when the plant discharges to Bear Creek. The operators report that the TSS discharged from the lagoons to the filters can reach as high as 200 mg/l, primarily as a result of algae, and can still be filtered, following the DAF unit, to meet the 10 mg/l requirement.

Filter Description

Effluent passes from the DAF unit into a steel splitter box inside the filter building, then by gravity into the two gravity multi-media filters. The filters are open top steel tanks set into the wall of the filter building to allow the splitter box to be inside. The majority of the filters are positioned outside the building. Automatic valves with mechanical time controls are used to operate the filters.

The filters were designed to operate for a range of flows between 100 gpm (0.15 mgd) and 1,550 gpm (2.23 mgd) with both filters in operation. The filters contain gravel, sand, and

charcoal. The charcoal was replaced in 1994. The filter media layer depths are marked on the outside of the filters.

Prechlorination

The operators tried prechlorination ahead of the filters but dropped the practice because no improvement in effluent quality was obtained. The operators chlorinate the filters once each year at annual start-up. They do not chlorinate them during the winter operation.

Filter Backwash

The filters have an automatic backwash cycle. The filter backwash uses water from the chlorine contact basin. The backwash pump is mounted in the chlorine contact basin. No air agitation of the filter media is used in backwash. The pump has a rated capacity of 2,925 gpm at 62 feet TDH and is equipped with a 60 hp motor.

5.2.7 Disinfection

The disinfection facilities use gaseous chlorine to reduce bacteria levels in the effluent. During the winter, the lagoons and filtration reduce bacteria levels to a point that chlorination is not needed to meet effluent discharge requirements. Dechlorination is not currently provided since chlorination is used only on effluent to be reused by irrigation.

Chlorine Feed System

The existing chlorine feed system uses gaseous chlorine supplied from 150-pound cylinders. Chlorine is withdrawn from the cylinders by vacuum. The system includes one 100-ppd chlorinator. The control of the chlorinator was recently modified to be flow-paced.

The chlorinated effluent passes over the effluent weir used for flow measurement. The turbulence provided by the weir allows for mixing of the chlorine in the effluent.

Chlorine Contact Tank

The chlorine contact tank was designed for one-hour detention at the average design flow of 0.8 mgd, which is less than the current annual average flow of approximately 1.2 mgd. The tank has baffles to allow for serpentine flow through the tank.

5.2.8 Outfall

The existing discharge permit allows limited discharge to Bear Creek during winter months, as described in Chapter 4. Flood conditions on Bear Creek have not been mapped. Bear Creek flows to Rock Creek, which then discharges to the Pudding River. The Pudding River flows north and joins the Molalla River about a mile from its confluence with the Willamette River between Wilsonville and Canby. With existing flows, the plant has exceeded its mass load limit to the creek during peak wet-weather conditions.

Outfall

The 18-inch plant outfall extends to Bear Creek and discharges near the plant access bridge. The pipe discharges at the edge of the streambank and has no diffuser. The pipe has an unsurcharged capacity of 2.2 mgd.

Stream Gauge

Since the current discharge permit has dilution limits, the stream flow in the creek must be measured continuously. The culvert under the access road was fitted with a 10-foot long rectangular weir without end contractions. A recording stream gauge was installed at the access road bridge in a stilling well. Soon after it was constructed, the gauge became inoperable and is not used. The operators use a staff gauge on the wall near the weir to determine the creek flow.

5.2.9 Irrigation System

Land application of effluent for irrigation is allowed by the current discharge permit and is even encouraged because it is a beneficial use of the effluent. Currently, irrigation takes place on approximately 15 acres at the plant site and on approximately 70 acres of private farmland adjacent to the plant. Current application of effluent has exceeded defined agronomic rates. Under the current arrangement, the owner of the farmland dictates the amount and time of irrigation, which means that the City cannot always irrigate when necessary for optimum operation of the plant.

An irrigation pump was mounted on an exterior pad above the chlorine contact basin. This 530-gpm vertical turbine pump is used for irrigating the plant site. The farm-owner installed piping and a pump to draw water from the chlorine contact basin as he needs it since he was unable to get the quantity and pressure needed to irrigate his land from the City's irrigation pump. His pump is a portable gas-driven pump positioned next to the west side of the chlorine contact basin. During the summer of 1999, the farmer's pump was unavailable, so the City used pumps from the emergency raw water pumping operation that had been required during the summer of 1996 at the City's water treatment plant.

5.2.10 Other Facilities

Other wastewater treatment facilities at the plant include compressed air, instrumentation and controls, standby power, and the office/laboratory.

Compressed Air

A large central air compressor in the filter building provides compressed air for valve operation and elsewhere as needed. The compressor works well and has had no maintenance problems.

Instrumentation and Controls

The instrumentation throughout the plant is nearly 20 years old; it is outdated and is becoming less reliable.

Standby Power

The transfer pump station building houses a natural gas generator for standby emergency power. The nameplate on the unit indicates that it operates at 300 amps and 600 volts (150 kw). The generator is exercised once a week and apparently works well. The unit is in good operating condition and is serviced twice per year.

Office/Laboratory

The existing office/laboratory building is undersized and the equipment is nearly 20 years old.

5.2.11 Summary of Treatment Facility Deficiencies

The following are the major deficiencies associated with the condition of existing treatment facilities:

- The existing headworks is hydraulically undersized for existing flows.
- The headworks configuration is such that the Parshall flume does not provide accurate readings.
- Because there are no grit removal facilities, grit accumulates in the aeration basin and facultative lagoons.
- The transfer pump station and associated force main are hydraulically undersized.
- The chlorine contact tank is hydraulically undersized for existing flows.
- Mass load limits to Bear Creek exceed permitted limits during peak wet-weather flows.
- Application of effluent on land currently exceeds defined agronomic rates.

5.3 EXISTING TREATMENT PERFORMANCE

Historical treatment performance was evaluated based on plant records from January 1996 through December 1998.

5.3.1 Historical Treatment Performance—Liquid Stream

The historical average effluent concentrations for 5-day biochemical oxygen demand (BOD) and total suspended solids (TSS) are summarized in Table 5-2. Both summer and winter averages are shown because the effluent requirements vary with the seasons. During the summer, effluent is land-applied and stored for discharge during the winter. During the winter, effluent is discharged to Bear Creek.

Tables 5-3 and 5-4 show the treatment plant's historical performance for BOD and TSS, respectively. The tables show violations of the existing NPDES permit's BOD and TSS discharge limits. The violations occurred during periods of extreme rainfall in February 1996 and December 1998.

TABLE 5-2.
HISTORICAL AVERAGE EFFLUENT CONCENTRATIONS

Year	BOD (mg/l)		TSS (mg/l)	
	Summer	Winter	Summer	Winter
1996	20.7	4.0	14.7	2.2
1997	10.3	3.3	9.0	2.4
1998	9.3	3.4	9.5	2.3
Overall Average	13.4	3.6	11.1	2.3

TABLE 5-3.
HISTORICAL TREATMENT PERFORMANCE—EFFLUENT BOD

Year	Maximum Monthly Average					Daily Maximum Loading (ppd)	
	Concentration (mg/l)		Loading (ppd)		Winter Minimum Removal		
	Summer	Winter	Summer	Winter		Summer	Winter
1996	37.9	5.1	127	82 [a]	83% [a]	276	521 [a]
1997	23.0	4.7	102	40	92%	189	70
1998	24.0	4.8	89	53	95%	203	154 [a]
Permit Limits		10 mg/l Maximum		66 ppd Maximum	85% Minimum		132 ppd Maximum

a. Recorded value violates NPDES permit limit.

TABLE 5-4.
HISTORICAL TREATMENT PERFORMANCE—EFFLUENT TSS

Year	Maximum Monthly Average					Daily Maximum Loading (ppd)	
	Concentration (mg/l)		Loading (ppd)		Winter Minimum Removal		
	Summer	Winter	Summer	Winter		Summer	Winter
1996	28.0	5.6	103	89 [a]	89%	182	542 [a]
1997	15.0	3.7	71	46	91%	186	71
1998	25.9	2.8	99	41	87%	259	133 [a]
Permit Limits		10 mg/l Maximum		66 ppd Maximum	85% Minimum		132 ppd Maximum

a. Recorded value violates NPDES permit limit.

Table 5-5 summarizes historical disinfection performance. There were no permit violations regarding fecal coliform during the period of record summarized. Note that fecal coliform data and permit limits shown represent the geometric mean.

TABLE 5-5.
HISTORICAL TREATMENT PERFORMANCE—DISINFECTION

	Chlorine Residual (mg/l)			Fecal Coliform (colonies/ml)	
Year	Monthly Average	Monthly Maximum	Monthly Minimum	Monthly Maximum	Weekly Maximum
1996	0.6	2.5	0.0	13	65
1997	0.5	3.0	0.0	10	33
1998	0.5	3.5	0.0	42	130
Permit Limits				200	400

The NPDES and MAO discharge requirements include limits on the pH range of the effluent. Effluent temperature data are used in outfall mixing zone analyses. Table 5-6 shows the historical records for pH and temperature. There were no permit violations regarding pH during the period of record summarized.

TABLE 5-6.
HISTORICAL TREATMENT PERFORMANCE—pH AND TEMPERATURE

	Effluent pH			Effluent Temperature (°C)		
Year	Avg.	Min.	Max.	Avg.	Min.	Max.
1996	7.0	6.5	7.6	15	10	21
1997	7.0	6.4	7.6	16	10	21
1998	6.8	6.4	7.4	16	9	22
Permit Limits		6.0	9.0			

As shown by the water balance in Appendix E, effluent is being applied at the land application site at greater than agronomic rates and at maximum hydraulic loading rates.

5.3.2 Historical Treatment Performance—Solids Stream

Since the City built the existing treatment plant, sludge disposal has not been an issue because the lagoons have provided ample capacity to digest and store the sludge. Since the plant is nearing 20 years of operation, sludge disposal was addressed in 1999. The City developed a biosolids management plan and contracted out the dredging of the sludge accumulation in facultative/storage Lagoon No. 1. The sludge was applied to the adjacent farmland where the City currently reuses effluent.

5.3.3 Summary of Treatment Performance Deficiencies

The following are the major deficiencies associated with historical treatment performance:

- The treatment plant has isolated periods of noncompliance with permit limits for BOD and TSS during periods of extreme rainfall. The violations are related to mass load limitations in Bear Creek rather than the plant's ability to produce high quality effluent.
- The effluent land application site is undersized.

5.4 EXISTING SYSTEM AND OPERATOR CLASSIFICATION

The classifications for the existing system and the three operators are shown in Table 5-7.

TABLE 5-7.
SYSTEM AND OPERATOR CLASSIFICATIONS

	Collection System	Treatment Plant
System	Level 3	Level 3
Operator 1	Level 3	Level 2
Operator 2	Level 2	Level 1
Operator 3	Level 2	Level 1

CHAPTER 6.
COLLECTION SYSTEM

This chapter describes the collection system evaluation. An infiltration and inflow (I/I) flow monitoring analysis was performed for this study, and an I/I removal recommendation was developed. The potential for system expansion was determined. The capacity of the existing collection system was evaluated based on the recommendations of the I/I analysis and the determination of future collection system expansion needs.

Detailed cost estimates are included in Appendix C. All cost estimates are in 2000 dollars and are total project costs, which include construction costs, contingencies and allied costs (engineering, legal, and administrative costs).

6.1 INFILTRATION/INFLOW

As described in Chapter 4, the Molalla collection system experiences high levels of I/I. Current wastewater flows are significantly higher than the per capita guidelines developed by the EPA. Flow monitoring was undertaken to provide a basis for identifying sources of I/I, estimating the quantity of I/I from significant sources and the cost of removing identified I/I sources, and making a recommendation for I/I removal.

6.1.1 Flow Monitoring

I/I flow monitoring for this study was performed in January and February 1999. This monitoring, as well as general observations by City and KCM staff, were used to determine areas of high I/I.

Flow Monitoring Program

KCM coordinated with the City to establish a flow monitoring program to be performed by City and KCM staff. The monitoring involved entering manholes and measuring the size of line, depth of flow, and velocity of flow for the sewer lines draining into each manhole. Velocity was measured using a propeller meter positioned at two-thirds the depth of flow. At manholes more than 8 feet deep, a tripod and harness were used as a safety precaution.

An initial observation of system flows by City and KCM staff on December 28, 1998 was used to determine the most appropriate areas for monitoring. Flow monitoring was performed on January 14, 1999 after 0.4 inches of rain the previous night and on January 22, 1999 after an entire week of rain and showers. The results of the monitoring on these two days were used to determine areas of potentially significant I/I. These areas were further investigated during a night monitoring session to isolate I/I from domestic flows. The night monitoring occurred on the night of February 18, 1999 after a week of rain and showers.

The following constraints on the flow monitoring program must be considered when evaluating the data:

- During heavy rains, much of the trunk line along Toliver Road is surcharged, which surcharges many of the main lines that feed into it. Not only does surcharging in the system make it difficult to perform the flow measurements, but the results of the monitoring are skewed due to the restriction in capacity. In general, measurements were not taken at surcharged manholes.
- The split manhole along the Toliver trunk line at Leroy Avenue diverts the flows parallel to the Toliver trunk until another split manhole that diverts flows back into the Toliver trunk and to the Bear Creek trunk. This configuration makes it difficult to assess unit flow contributions for the portions of the Toliver trunk downstream of Leroy Avenue and the Bear Creek trunk downstream of the influent diversion.
- The trunk lines along Bear Creek and Highway 211 have very difficult access. The line along Bear Creek between Highway 213 and Highway 211 is primarily in an undeveloped area with no road access to the manholes, and ponding occurs around the manholes during heavy rains. The line along Highway 211 from Bear Creek to about Ridings Avenue is difficult to access due to heavy traffic on Highway 211 with very little highway shoulder. Also along this line, a guardrail was placed over the manhole at Bear Creek and most of the other manholes are within a deep ditch on the side of the highway, so that a tripod could not be used. Few measurements could be taken along these trunk lines.
- The four collection system pump stations may skew the flow measurements. These pump stations, however, have very small contributing areas and are not likely to significantly affect the measurements in the main and trunk sewer lines.

Flow Monitoring Results

Results of the flow monitoring program are summarized in Table 6-1. A map in Appendix B shows the indexing of the manholes. Additional flow monitoring details are included in Appendix D.

To allow better comparison of flows at the manholes, a unit flow value was calculated that accounts for the amount of upstream pipe contributing flow to each manhole (flows further downstream in the system are generally higher because of the larger drainage area). For each pipe segment upstream of the manhole, the length of the segment in feet was multiplied by the pipe diameter in inches. Flow measured at the manhole in gallons per day (gpd) was then divided by the sum of this value for all upstream segments. The dimensions of this unit flow are gallons per day per inch-diameter-foot (gpd/in.dia-ft).

TABLE 6-1.
FLOW DATA FROM FLOW MONITORING

Manhole Location	Manhole Number [a]	Direction of Inflow [b]	Unit Flows [c] (gpd/in.dia.-ft.) January 14	January 22	February 18
Toliver east of Highway 213	10	east			4.7 [d]
Toliver east of Highway 213	16	east		surcharged	
Toliver near Bear Creek diversion	22	east southeast	3.8 [d] low [e]	7.6 [d] -	
Toliver and Leroy	28	east south	2.9 1.5	9.0 surcharged	
Leroy south of Toliver	32	east south		surcharged surcharged	
Carol south of Toliver	38	north south		low [e] low [e]	
Toliver and Ridings	56	east south	3.9 low [e]	8.3 11.1	4.9 low [e]
Ridings and Heintz	60	south		17.2	
Toliver and Revilot	82	east north	2.2 low [e]	9.2 surcharged	
Heintz and Kennel	98	east south	3.4 low [e]	8.6 surcharged	5.3 low [e]
Kennel south of Heintz	100	south		low [e]	
Molalla and Heintz	114	east south north	3.2 3.0 3.5	surcharged surcharged surcharged	
Heintz east of Molalla	172	east		6.0	
Heintz and Grange	176	east south		6.7 6.1	
Highway 211 and Lola	226	east south	1.8 drop [f]	surcharged drop [f]	
Lola and 2nd	302	east south		17.3 5.6	
Highway 211 and Fenton	228	east		6.1	2.0
Highway 211 west of Cole	230	east south		2.5 low [e]	
Highway 211 east of Cole	232	east south		7.9 6.6	2.3 low [e]
Heintz and Fenton	178	east south	3.8 3.7	6.6 6.4	2.6 low [e]

TABLE 6-1 (continued).
FLOW DATA FROM FLOW MONITORING

Manhole Location	Manhole Number [a]	Direction of Inflow [b]	Unit Flows [c] (gpd/in.dia.-ft.) January 14	January 22	February 18
Heintz and Cole	182	south		20.7	
		east		low [e]	
		north		low [e]	
Cole and Patrol	200	east		low [e]	
		south		low [e]	
Molalla and Shirley	116	east	5.6	surcharged	6.5
		north	0.9	surcharged	2.7
Shirley east of Molalla	134	east		3.9	
Shirley and Fenton	154	east		7.4	
		south		low [e]	
Shirley and Cole	160	east		low [e]	
		north		low [e]	
Francis east of Molalla	122	east		low [e]	
Highway 213 and Bear Creek	500	southeast		10.3 [d]	5.0 [d]
		north		2.3	1.7
Hwy 213 north of Bear Creek		north		2.0	
Industrial Way	508	east		-	-
Highway 211 and Ona	526	east	1.1		
Highway 211 and Shaver	546	south	1.5	5.7	3.7
Highway 211 and Hart	562	east			5.4
		south			5.8
Hart and 3rd	566	east		low [e]	
		south		6.9	
Hart and 4th	570	east	4.0	15.8	6.9
		south	low [e]	low [e]	low [e]
Hart and 5th	572	east		low [e]	
		south		low [e]	
Metzler and 5th	580	east		9.6	4.4
		south		10.1	2.1
Molalla and 5th	586	east		8.6	
		south		8.3	

a. Manhole numbers are indexed to the collection system inventory in Appendix B.
b. Direction of inflow is the direction from which the flow enters the manhole.
c. Unit flows are calculated based on the measured flow and the length and size of upstream piping.
d. Unit flows at these manholes are skewed by the diversion from the Toliver trunk to the Bear Creek system.
e. Low indicates that flows were not deep enough to be measured (less than about 2 inches).
f. Drop indicates a drop manhole and/or large difference between inflow invert and outflow invert.

6.1.2 I/I Evaluation

The flow monitoring results were used to evaluate I/I removal projects. Generally, I/I removal in areas with the highest unit flows is most likely to be effective. Potential I/I removal projects were evaluated from the January 22 and February 18 data. I/I removal projects assume full sanitary sewer replacement, including piping, manholes, and service laterals to ensure that all sources of I/I are addressed. Additional details of this evaluation are included in Appendix D.

The January 22 data show four areas with unit flows of more than 15 gpd/in.dia.-ft. These are the areas that drain to Ridings and Heintz, Lola and 2nd from the east, Heintz and Cole from the south, and Hart and 4th from the east (see Figure 6-1). I/I removal projects in these areas would involve over 11,000 feet of 8-inch piping, approximately 36 manholes, and approximately 208 laterals, for a total project cost of $1,740,000. With the implementation of these I/I removal projects, the most I/I removal that can be expected on a peak daily average flow (PDAF) basis is 0.65 million gallons per day (mgd), or 10 percent of the existing PDAF and 6 percent of the 20-year PDAF projection.

The February 18 data show two areas with unit flows of more than 6 gpd/in.dia.-ft. These are the areas that drain to Molalla and Shirley from the east and to Hart and 4th from the east (see Figure 6-1). I/I removal projects in these areas would involve almost 11,000 feet of 8-inch piping, approximately 36 manholes, and approximately 196 laterals, for a total project cost of $1,650,000. With implementation of these I/I removal projects, the most I/I removal that can be expected on a PDAF basis is 0.29 mgd, or 5 percent of the existing PDAF and 3 percent of the 20-year PDAF projection.

These findings indicate that significant costs are required to remove a relatively small amount of flow. For example, the I/I removal projects for $1,740,000 that will remove 0.65 mgd on a peak flow basis will cost more than would be saved by reduced downstream facility sizing. Removing 0.65 mgd of I/I flows of 0.65 mgd would allow the size of downstream sewers to be reduced by only one pipe size, if at all. Additionally, the I/I removal would not significantly reduce summer flows, and most of the treatment facility costs are to accommodate the maximum month dry-weather flow (MMDWF). If the estimated I/I flows of 0.65 mgd were removed, cost savings at the treatment plant would come primarily from reduced capacity requirements for the headworks and pump stations. The estimated cost savings at the treatment plant are likely to be less than 20 percent of the estimated cost of the I/I removal project of this example.

Moreover, I/I removal projects often do not provide permanent I/I removal. When I/I cannot enter the system in one area, it is likely to enter through the adjacent or downstream system. This may be especially true in Molalla, where the ground elevation and water table are gently sloping and the potential I/I removal projects are not isolated from other portions of the system.

In conclusion, the flow monitoring data indicates that existing I/I is fairly widespread. The potential I/I removal projects that were evaluated would cost more than could be saved in the sizing of downstream facilities. As a result, no I/I removal projects are recommended. It is important, however, that the City develop a program for ensuring that I/I does not become worse than current levels.

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City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 6-1
POTENTIAL AREAS FOR
I/I REMOVAL PROJECTS

6.1.3 I/I Recommendation

Since I/I cannot be attributed to a specific basin or basins within the Molalla system, no location-specific I/I reduction projects are recommended. It is recommended, however, that the City implement an ongoing I/I reduction program aimed at keeping I/I from becoming worse. This program would target repairs that are needed to minimize the risk of further collection system degradation and subsequent I/I increases. The recommended I/I reduction program includes the following:

- **Smoke testing, cleaning and video survey of the sanitary sewer collection system.** Smoke testing would be used as an initial investigation to identify significant inflow sources. Cleaning the sanitary sewer is often required before a video survey can be made. A video survey of areas of suspected infiltration would allow problems to be identified and ranked in order of severity.
- **Repair/replacements of problem areas identified by the smoke testing and video survey.** Based on the smoke testing, inflow sources should be removed. Based on the video survey, portions of the sanitary sewer that appear to be allowing infiltration into the system should be upgraded or replaced.
- **Cleaning the storm sewer collection system.** Often storm sewers have reduced capacity due to grit and debris buildup. Cleaning the storm sewers and thereby increasing the storm sewer capacity can allow for better routing of runoff and, therefore, less chance inflow will enter the sanitary sewer.

Estimated Costs

Table 6-2 shows estimated unit costs and annual costs for the recommended I/I reduction program. The unit costs assume that smoke testing and cleaning and video survey of the sanitary system will be performed by contractors. The unit costs for pipe replacements assume 8-inch pipes at 5-foot depth or less, with manholes 400 feet apart. If larger or deeper piping or more manholes are replaced, less length of piping can be replaced for the annual cost shown. Annual cleaning and surveying of 5,000 feet of sanitary sewer will allow the entire existing collection system to be surveyed within 20 years. It was assumed that 10 percent of the system surveyed each year would have to be replaced.

TABLE 6-2.
I/I REDUCTION PROGRAM ESTIMATED ANNUAL COSTS

Program Element	Estimated Unit Cost	Annual Piping Coverage	Estimated Annual Cost
Smoke testing, cleaning and video survey	$ 1.50/foot	5,000 feet	$7,500
Replacement of problem areas	$ 64.00/foot	500 feet	$32,000
Cleaning storm sewer	$ 1.00/foot	500 feet	$500
Total Annual Cost			**$40,000**

Preliminary Schedule

The flow monitoring program performed with this study and discussed above was used to determine a preliminary schedule for the I/I reduction program. Areas that the program found to have higher than average I/I were given the highest priority for cleaning and video surveying. Suggested priorities (with 1 being the highest priority) for the first few years of the I/I reduction program are as follows:

1) The subbasin that drains to Hart and Highway 211 (Manhole 562) from the south. This basin includes systems on Hart from Highway 211 to 7th, Metzler from 3rd to 7th, Molalla from 5th to 7th, and 5th from Hart to Lola, for an approximate total of 8,000 feet of piping.
2) The subbasin that drains to Lola and Highway 211 (Manhole 226) from the south. This basin includes systems on Lola from Highway 211 to 5th and Eckerd from 2nd to 5th, for an approximate total of 2,500 feet of piping.
3) The subbasin that drains to Shirley and Molalla (Manhole 116) from the east. This basin includes systems on Shirley from Molalla to east of Cole, on Cole from Shirley to Frances, and north of Shirley, for an approximate total of 5,500 feet of piping.
4) The subbasin that drains to Toliver and Ridings (Manhole 56) from the south. This basin includes systems on Ridings from Toliver to Highway 211 and Dixon from Hoyt to Highway 211, for an approximate total of 3,000 feet of piping.
5) The subbasin that drains to Hart and Highway 211 (Manhole 562) from the east. This basin includes systems on Highway 211 from Hart to Berkley, Molalla from Highway 211 to 4th, Engle from Highway 211 to 4th, Sweigle from Highway 211 to 5th, and Berkley from Highway 211 to 5th, for an approximate total of 7,500 feet of piping.

Conclusion

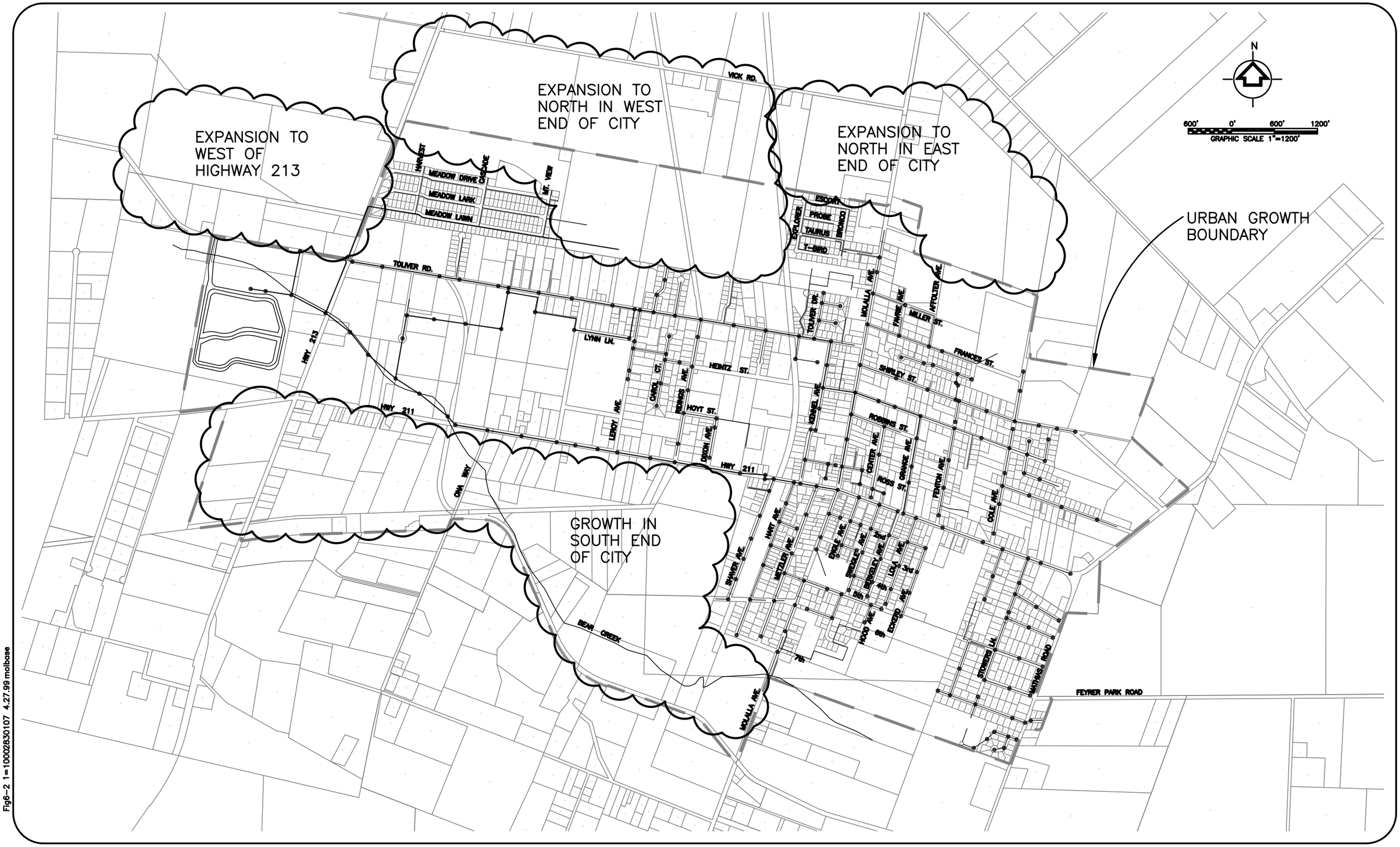
It is recommended that the City implement the I/I reduction program described above. The purpose of the I/I reduction program is to control I/I in the existing system so that it does not exceed current levels. As part of the program, the City must be prepared to require property owners to remove any illegal connections or make repairs to obviously leaking laterals that are identified during the video survey of the system.

6.2 FUTURE EXPANSION OF THE COLLECTION SYSTEM

An analysis of future expansion requirements for the collection system involved identifying likely areas of expansion and determining the probable impacts of expansion on the existing system.

6.2.1 Areas of Expansion

As discussed in Chapter 2, the City expects that the UGB will be expanded in the near future to accommodate a continued demand for growth. A likely area of expansion for additional residential area is to the north, to Vick Road, in the west end of the City. Other



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City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 6-2

AREAS OF POTENTIAL SYSTEM EXPANSION

areas for potential expansion for residential land might be to the west of Highway 213 and to the north in the east end of the City. There are also tracts of vacant commercial and industrial land available within the UGB in the southeast area of the City. These general areas of potential system expansion are shown in Figure 6-2.

6.2.2 Expansion Facilities and Impacts on the Existing System

Required facility expansions are described below for each general area of system expansion. Also described are the existing facilities that are likely to be affected by each area of system expansion.

Potential Residential Expansion to the North in the West End of the City

The existing subdivision north of Toliver Road is served by a 12-inch trunk line running south on Highway 213 to the Bear Creek trunk. The capacity of this 12-inch trunk should be adequate for the addition of flows from future residential expansion to the north of this subdivision. It is assumed that all future development in this expansion area will provide conveyance to this 12-inch trunk.

Potential Residential Expansion to the West of Highway 213

Expansion west of Highway 213 would be near the treatment plant and the existing trunk along Toliver Road. It is assumed that all future development in this expansion area will provide conveyance to the plant or to the trunk along Toliver.

Potential Residential Expansion to the North in the East End of the City

Expansion to the north in the east end of the City would be near the existing trunk along Toliver Road and the trunk along Molalla Avenue. It is assumed that all future development in this expansion area will provide conveyance to one of these trunk lines.

Potential Commercial and Industrial Growth in the South End of the City

The only existing system near this expansion area is the trunk along Highway 211. Some of this expansion area can be served by this trunk, but much of this area will require additional trunk lines. It is recommended that a new trunk line be installed along Highway 213 from the Bear Creek trunk to the south. Also recommended is a new trunk line from Highway 211 at Bear Creek to the south and east. It is assumed that all future development in this expansion area will provide conveyance to the existing trunk along Highway 211 or either of the two recommended trunk lines.

6.2.3 Recommendations for System Expansion

In general, system expansion will be served by the existing system and by facilities installed by the developers of future expansion areas. In some areas, the existing system will have to be upgraded to provide capacity to serve the expansion areas; these upgrades are discussed later in this chapter. Two system extensions are recommended into the south end of the City. These recommended trunk line extensions are shown in Figure 6-3 and are keyed to the capital improvement program (CIP) project numbering.

General System Extensions

System extensions should be completed as necessary to reach new development as it occurs. All system extensions should use 8-inch lines as a minimum size. The need for specific extensions will be determined by the location and phasing of future development.

Industrial (South) Trunk Extension

Future development in the south end of the City is not likely to occur until there is adequate water and sewer service in this area. The Industrial (South) Trunk extension is recommended to serve the east end of this area. This trunk should be a 12-inch line and should connect to the upstream end of the existing Bear Creek trunk. The estimated total project cost is $860,000.

Highway 213 (South) Trunk Extension

The Highway 213 (South) Trunk extension is recommended to serve future development in the south end of the City in the vicinity of Highway 213, or the west end of the expansion area. This trunk should be a 12-inch line connecting to the proposed 12-inch sewer line to be installed as part of the commercial development in the northeast corner of the intersection of Highway 211 and Highway 213 and as shown on Figure 6-3. The estimated total project cost is $310,000.

6.3 COLLECTION SYSTEM EVALUATION

Existing and future conditions were considered in the evaluation of the collection system and the recommended improvements. The collection system must be capable of conveying peak wastewater flows to the wastewater treatment plant. It was assumed for the collection system evaluation that the PDAF must be accommodated by the full flow gravity capacity of the system and that surcharging will allow for conveyance of the peak instantaneous flow (PIF). The evaluation assumes that existing diversions will remain.

6.3.1 Existing Conditions

Table 6-3 shows the estimated existing PDAF through trunks and main lines in the existing system. These estimates are based on approximate drainage areas. The approximate full flow gravity capacities of the collection system trunks and main lines are shown, as well as any deficit in capacity as compared to the corresponding PDAF. The data show that some existing collection system trunks and mains are under-capacity, based on the assumptions of the analysis. These lines have apparently been able to convey peak flows to the plant by surcharging.

TABLE 6-3.
COLLECTION SYSTEM CAPACITY ANALYSIS – EXISTING CONDITIONS

Trunk Line	Percent of Total Service Area Served[a]	PDAF (mgd)	Approx. Capacity (mgd)	Capacity Deficit (mgd)
Treatment Plant Trunk (Total)	100%	6.2	5.6	0.6
Toliver Basin				
Toliver from 2,200' east of Hwy 213 to Plant Trunk	55%	3.4	2.5	0.9
Toliver from Leroy to 2,200' east of Hwy 213	40%	2.5	1.6	0.9
Toliver from Wittenberg to Leroy	70%	4.3	3.8	0.5
Toliver from Kennel to Wittenberg	60%	3.7	2.4	1.3
Heintz and Molalla to Toliver and Kennel	55%	3.4	2.1	1.3
Molalla from Hwy 211 to Heintz	5%	0.3	1.4	
Grange and Hwy 211 to Heintz and Molalla	20%	1.2	1.6	
Bear Creek Basin				
Bear Creek from Diversion Addition to Plant Trunk	45%	2.8	3.0	
Bear Creek from Hwy 211 to Diversion Addition	25%	1.6	3.8	
Hwy 211 from Dixon to Bear Creek	25%	1.6	2.2	
Hwy 211 from Molalla to Dixon	20%	1.2	1.4	
Hwy 213 (North) Trunk	5%	0.3	1.5	

a. This value represents the portion of flow from the City of Molalla's total service area that drains through the trunk line.

6.3.2 Future Conditions

The sizing of collection system upgrades must consider future system flow conditions. Table 6-4 shows the estimated future PDAF through trunks and main lines in the existing system and in system extensions. The approximate full-flow gravity capacities of the collection system trunks and main lines are shown, as well as any deficit in capacity compared to the corresponding PDAF. The data show that under future flow conditions, some existing collection system trunks and mains will be under-capacity, based on the assumptions of the analysis.

TABLE 6-4.
COLLECTION SYSTEM CAPACITY ANALYSIS—FUTURE CONDITIONS

	Exist. Service Area		New Service Areas		Total	Approx.	Capacity
	Percent of Total[a]	PDAF (mgd)	Percent of Total[b]	PDAF (mgd)	PDAF (mgd)	Capacity (mgd)	Deficit (mgd)
Treatment Plant Trunk (Total)	100%	6.2	100%	4.7	10.9	5.6	5.3
Toliver Basin							
Toliver from Highway 213 to Plant Trunk	65%	4.0	45%	2.1	6.1	2.5	3.6
Toliver from 2,200' east of Highway 213 to Highway 213	60%	3.7	20%	0.9	4.7	2.5	2.2
Toliver from Leroy to 2,200' east of Highway 213	40%	2.5	10%	0.5	3.0	1.6	1.4
Toliver from Wittenberg to Leroy	80%	5.0	20%	0.9	5.9	3.8	2.1
Toliver from Kennel to Wittenberg	65%	4.0	20%	0.9	5.0	2.4	2.6
Heintz and Molalla to Toliver and Kennel	60%	3.7	20%	0.9	4.7	2.1	2.6
Molalla from Highway 211 to Heintz	15%	0.9	0%	0.0	0.9	1.4	
Grange and Highway 211 to Heintz and Molalla	20%	1.2	5%	0.2	1.5	1.6	
Bear Creek Basin							
Bear Creek from Diversion Addition to Plant Trunk	35%	2.2	55%	2.6	4.8	3.0	1.8
Bear Creek from Highway 211 to Diversion Addition	15%	0.9	35%	1.6	2.6	3.8	
Highway 211 from Dixon to Bear Creek	15%	0.9	20%	0.9	1.9	2.2	
Highway 211 from Molalla to Dixon	10%	0.6	5%	0.2	0.9	1.4	
Extension – Industrial (South) Trunk	0%	0.0	15%	0.7	0.7	N/A	
Extension – Highway 213 (South) Trunk	0%	0.0	10%	0.5	0.5	N/A	

a. This value represents the portion of flow from the total existing service area that will drain through the trunk line after proposed changes to the collection system.
b. This value represents the portion of flow from new service areas added in the future that will drain through the trunk line.

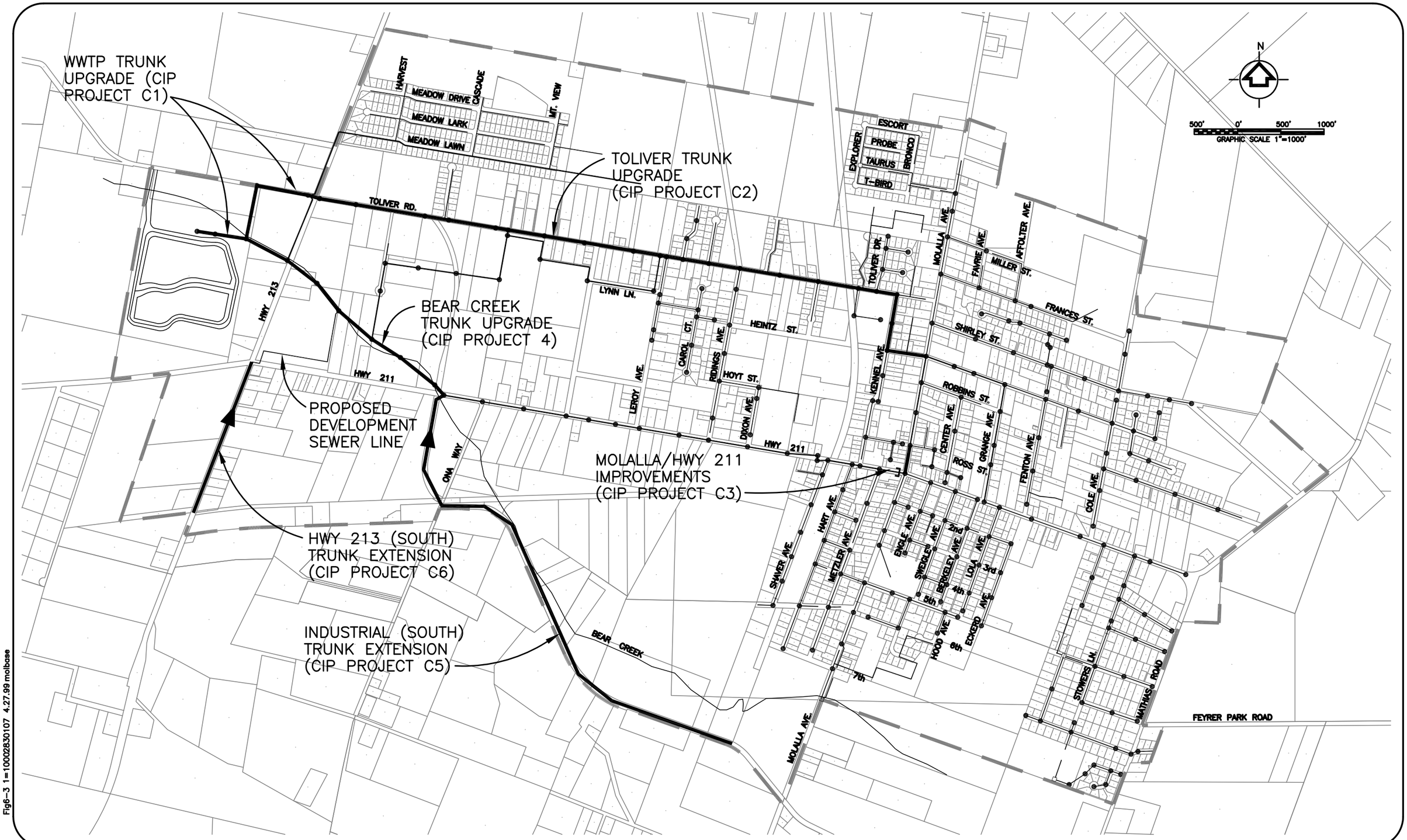
Two assumptions were made for future conditions that change the drainage configuration of the collection system. These assumptions are as follows:

- The trunk sewer along Highway 211 will not be upsized. This is a very deep sewer that was installed in 1995. To ensure that flows to this trunk do not exceed its capacity, the area south of Highway 211 and east of Molalla Avenue that currently drains to this trunk must be diverted north along Molalla Avenue to the Toliver Road trunk. This configuration is similar to the one that existed before the trunk sewer along Highway 211 was constructed. The Toliver Road upgrade will then be sized to accommodate the additional flows.
- The trunk sewer on Highway 213 from the north would be directed into the Toliver Road trunk at its crossing of Highway 213. The Highway 213 line currently runs literally through a manhole in the Toliver line. The City would like to break the line at the manhole, thereby draining all flows from the north into the Toliver line. The existing 12-inch line from Toliver road to Bear Creek would remain to pick up any future services along its length.

6.3.3 Recommendations for Improvements

Based on the evaluation of the collection system under future conditions, several upgrade and improvement projects are recommended. The recommended upgrades and improvements are shown on Figure 6-3 and are keyed to the CIP projects. The upgrades and improvements are also described below and summarized in Table 6-5 with estimated total project costs. Detailed cost estimates are included in Appendix C.

- **Treatment Plant Trunk Upgrade (CIP Project T1).** This project would upgrade the main trunk lines from the intersection of Toliver Road and Highway 213 to the plant. These upgrades would generally following the horizontal and vertical alignment of the existing lines. The existing 15-inch lines from Highway 213 to the confluence of the Toliver trunk and the Bear Creek trunk would be upgraded to 24-inch lines. The existing 21-inch lines from the confluence of the Toliver trunk and the Bear Creek trunk would be upgraded to 30-inch lines. As part of this project, the line from the north that literally runs through the manhole along the Toliver trunk at Highway 213 would be broken at the manhole as discussed above.
- **Toliver Road Trunk Upgrade (CIP Project T2).** This project would upgrade the trunk lines along Toliver Road from the intersection of Molalla Avenue and Heintz Street to the intersection of Toliver Road and Highway 213. If the existing lines are to be upgraded, they would be replaced with 24-inch lines. Because the City is currently improving Toliver Road, however, further disruption of the paved area in the near-term should be avoided and alternative alignments should be investigated in the predesign of this project. It is possible that a parallel trunk line could be installed for much of the distance of the project by following Heintz Street from Molalla Avenue to Leroy Avenue to Lynn Lane and possibly even further west through the school property and along the existing bypass alignment.



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WASTEWATER FACILITIES PLAN

FIGURE 6-3
RECOMMENDED COLLECTION SYSTEM UPGRADES AND SYSTEM EXTENSIONS

- **Molalla Avenue / Highway 211 Improvements (CIP Project T3).** This project would connect the manhole at Molalla Avenue and Highway 211 to the manhole to the north, thereby routing all upstream flows to the Toliver Road trunk. The line to the west of the manhole at Molalla Avenue and Highway 211 would be plugged to remove the flows from the Highway 211 trunk line as discussed above. This project will free up capacity in the Highway 211 trunk line for future development in the south end of the City. The Toliver Road Trunk Upgrade (CIP Project T2) was sized to accommodate the additional flows that will result from this project.
- **Bear Creek Trunk Upgrade (CIP Project T4).** This project would upgrade the main trunk lines from Highway 211 to the confluence of the Bear Creek trunk and the Toliver Road trunk. These upgrades would generally following the horizontal and vertical alignment of the existing lines. The existing 15-inch lines from Highway 211 to the confluence of the Toliver trunk and the Bear Creek trunk would be upgraded to 21-inch lines.

TABLE 6-5.
COLLECTION SYSTEM UPGRADE REQUIREMENTS

	Upgrade Pipe Size (inches)	Upgrade Pipe Length (feet)	Construc-tion Cost per Foot	Total Construc-tion Cost	Total Project Cost
Treatment Plant Trunk Upgrade From the intersection of Toliver Road and Highway 213 to the Plant	30	780	$240	$190,000	$270,000
	24	1,180	$192	$230,000	$330,000
			Total	$420,000	$600,000
Toliver Road Trunk Upgrade From Molalla Avenue and Heintz Street to the end of the Treatment Plant Trunk Upgrade	24	7,900	$192	$1,520,000	$2,200,000
Molalla/Hwy 211 Improvements Connect manhole at Molalla Avenue and Hwy 211 to manhole to the north; plug existing line to the west	12	330	$96	$30,000	$50,000
Bear Creek Trunk Upgrade From Highway 211 and Bear Creek to the Plant Trunk	21	1,830	$168	$310,000	$450,000
Total					**$3,300,000**

6.4 SUMMARY OF COLLECTION SYSTEM RECOMMENDATIONS

Collection system recommendations are as follows:

- **I/I Reduction Program**—The recommended I/I reduction program includes the following:

  - Cleaning and video survey of the sanitary sewer collection system.
  - Correction of problems identified by the video survey.
  - Cleaning the storm sewer system.
  - An annual budget of $40,000 per year.
- **System Extensions**—Recommendations for service to future expansion areas include the following (see Figure 6-3):
  - General system extensions to new development provided by developers.
  - Industrial (South) Trunk Extension: $860,000.
  - Highway 213 (South) Trunk Extension: $310,000.
- **Existing System Upgrades**—Recommendations for upgrades to the existing system are as follows (see Figure 6-3):
  - Upgrade the Treatment Plant Trunk: $600,000.
  - Upgrade the Toliver Trunk from the Treatment Plant Trunk to Molalla and Heintz: $2,200,000.
  - Improvements at Molalla and Highway 211: $50,000.
  - Upgrade the Bear Creek Trunk from the Treatment Plant Trunk to Highway 211: $450,000.

CHAPTER 7.
WASTEWATER TREATMENT PLANT

The City of Molalla wastewater treatment plant will require improvements to address existing deficiencies (see Chapter 5), as well as the demands of expected growth over the next 20 years (see Chapter 3). Evaluation of improvement alternatives involved the following steps:

- Design conditions were established for two phases of the 20-year planning period. Design conditions are the expected flow and load conditions that a wastewater treatment plant must be designed to accommodate. For analysis of the Molalla treatment plant, 10-year flow and load projections were used for Phase 1 and 20-year projections were used for Phase 2.
- Six general treatment alternatives were reviewed for their suitability in the City of Molalla's wastewater treatment system. A preliminary analysis eliminated all but three of the alternatives.
- The liquid-stream treatment aspects of the remaining three alternatives were evaluated in detail. This evaluation included the development of cost estimates. A preferred approach was selected as the recommended liquid-stream treatment process.
- Three solids management alternatives were developed and evaluated in detail. This evaluation included the development of cost estimates. The three alternatives were compared and one was selected as the recommended solids management program.
- Planning level design criteria were developed for all elements of the overall wastewater treatment improvement recommendation.

Detailed cost estimates are included in Appendix C. All cost estimates are in 2000 dollars and are total project costs, which include construction costs, contingencies and allied costs (engineering, legal, and administrative costs).

7.1 PRELIMINARY ANALYSIS

The preliminary analysis included the establishment of treatment plant design conditions and preliminary screening of six general improvement alternatives.

7.1.1 Wastewater Treatment Plant Design Conditions

The growth assumptions used to project flows and loads for the 20-year planning period include the population more than doubling, with similar increases in commercial and industrial development. This growth rate is likely to require that some treatment plant improvements be carried out in phases; therefore, design conditions were developed for flows and loads in 10 and 20 years. Flow and load projections were determined in Chapter 3. The I/I reduction program discussed in Chapter 6 does not recommend significant I/I reduction projects. Table 7-1 shows the resulting design conditions.

TABLE 7-1.
DESIGN CONDITIONS

Design Parameter	Phase 1	Phase 2
Design Year	2009	2019
Design Population	9,950	13,370
Flows, million gallons per day (mgd)		
Average Dry Weather Flow (ADWF)	1.6	2.1
Maximum Month Dry Weather Flow (MMDWF)	2.9	4.0
Average Wet Weather Flow (AWWF)	2.6	3.2
Maximum Month Wet Weather Flow	4.4	5.4
(MMWWF)	8.8	10.9
Peak Day Average Flow (PDAF)	14.9	18.5
Peak Instantaneous Flow (PIF)		
Loads, pounds per day (ppd)		
Average Biochemical Oxygen Demand (BOD)	2,300	3,090
Maximum BOD	5,160	6,940
Average Total Suspended Solids (TSS)	1,610	2,170
Maximum TSS	3,570	4,790

For evaluation of treatment alternatives, the required effluent standards must be established. The regulatory requirements are discussed in Chapter 4. The subsequent minimum effluent standards that the wastewater treatment plant improvements must achieve are summarized in Table 7-2.

TABLE 7-2.
MINIMUM EFFLUENT STANDARDS

Parameter	Effluent Limit
BOD, Monthly Average	10 mg/l
BOD, Weekly Average	15 mg/l
TSS, Monthly Average	10 mg/l
TSS, Weekly Average	15 mg/l
E. Coli, 30-day log mean	126 colonies/ 100 ml
E. Coli, maximum	406 colonies/ 100 ml
Chlorine Residual	0 mg/l

7.1.2 Initial Treatment Alternatives

The following six general treatment improvement concepts were reviewed as initial alternatives:

- No action

- Regional treatment
- I/I reduction only (no improvement of existing treatment plant)
- Upgrade the existing plant treatment processes
- Convert the plant to an oxidation ditch plant
- Convert the plant to a sequencing batch reactor (SBR) plant

No Action

Under the no-action alternative, no changes would be made to the existing collection or treatment facilities. This alternative is not feasible for two reasons:

- The treatment plant currently operates above design capacity during winter, occasionally resulting in sewage overflows from the plant to Bear Creek.
- Continued growth within the City is expected, which would further strain the capacity of the existing facilities.

Regional Treatment

In the regional-treatment alternative, the City's wastewater would be conveyed to a regional treatment plant to be treated along with wastewater from other communities. Regional treatment has the potential benefits of reducing capital and operation and maintenance (O&M) costs per unit of treatment capacity.

The cities closest to Molalla that are large enough to be considered for a regional treatment partnership are Canby to the northwest, Woodburn and Hubbard to the west, and Silverton and Scotts Mills to the southwest. All of these cities, however, are too far away (Scotts Mills is the closest at 8 miles away) for regional treatment to be economically beneficial; any capital investment saved by regional treatment would be spent on the additional conveyance facilities required. Therefore, the regional treatment option is not feasible.

I/I Reduction Only

Under the I/I reduction only alternative, only I/I reduction projects would be performed and no improvements would be made to the existing treatment facilities. This alternative is not feasible for many reasons:

- The I/I in the system is widespread, as determined by the I/I analysis discussed in Chapter 6. Most or all of the existing collection system would have to be replaced to significantly reduce I/I, so this alternative would not be cost-effective.
- The summer effluent disposal facilities are not adequate for the existing dry weather flows, as shown in the water balance analysis of the existing plant facilities with existing influent flows. (See Appendix E for the water balance spreadsheets). Since the dry weather flows would not be affected by I/I reduction, the effluent disposal facilities would still be inadequate.

- As discussed in Chapter 5, the existing plant has other operational deficiencies that are not attributable to excessive I/I.

Upgrade the Existing Plant Treatment Processes

Improvements in this alternative would generally include upgrading each of the existing treatment processes to meet the design requirements. Specific improvements in this alternative are the upgrade of the headworks and transfer pump station, construction of new aeration basins, installation of secondary clarification in 10 years to accommodate Phase 2 design conditions, and upgrades to and expansion of the existing advanced treatment facilities.

Convert the Plant to an Oxidation Ditch Plant

For this alternative, the plant improvements would include upgrading some of the existing treatment processes, including the headworks and the transfer pump station, and the construction of oxidation ditches and secondary clarifiers upstream of the existing lagoons.

Convert the Plant to an SBR Plant

For this alternative, the plant improvements would include upgrading some of the existing treatment processes, including the headworks and the transfer pump station, and the construction of SBRs of the existing lagoons.

Summary of Initial Treatment Alternatives

Through the screening of initial alternatives above, all but three alternatives were deemed not feasible. The three remaining alternatives are upgrading the existing plant treatment processes, converting to an oxidation ditch plant, and converting to an SBR plant. The liquid stream treatment processes of these three alternatives are evaluated in more detail below and compared to determine an improvement recommendation. The solids management facilities recommendations are evaluated later in this chapter. The effluent disposal evaluation is included in Chapter 8.

7.2 LIQUID-STREAM TREATMENT PROCESSES

Each of the alternatives still considered feasible after the initial screening of alternatives are evaluated in more detail and compared to determine the most cost-effective alternative below.

7.2.1 Upgrade the Existing Plant Treatment Processes

For the alternative to upgrade the existing plant treatment processes, the following improvements would be required:

- New preliminary treatment facilities and influent flow metering (headworks), located in the vicinity of the existing headworks.
- New transfer pumping facilities located in the vicinity of the existing transfer pump station.

- New aeration basins located in Lagoon No. 1.
- Future installation of secondary clarifiers and return activated sludge (RAS) and waste activated sludge (WAS) pumping facilities.
- Expansion of the advanced treatment system, the air flotation (DAF) unit with associated flocculation equipment and filter units located in the vicinity of the existing control building.
- Upgraded disinfection equipment located in the vicinity of the existing control building.

Preliminary Treatment and Flow Metering (Headworks)

Preliminary treatment removes solids, debris, and/or grit to minimize the deposit of these materials in the treatment plant basins. Preliminary treatment is primarily for the purpose of reducing plant maintenance costs, so preliminary treatment processes often are selected based on operator preference. Three main options for preliminary treatment can be provided individually or in combination: comminution (grinding), grit removal, and screening.

Comminution

Comminution is the preliminary treatment process at the existing plant. Comminutors are relatively common at plants of this size. If a comminutor is used, then a bypass channel with a manual bar screen, as provided at the existing plant, is required for when the comminutor is not operational. Comminutors reduce the size of material in the influent flow, but do not remove it from the flow stream. Comminutors have recently been installed upstream of fine screens to break up fecal material prior to screening and subsequently reduce the amount of fecal material in the screenings.

Grit Removal

Grit removal systems include aerated grit chambers and vortex separators. For many small treatment plants with parallel basins, grit can be removed manually from downstream basins every five to 10 years more economically than grit removal processes, which require regular maintenance. Grit removal can also reduce wear on sludge pumps, and some unit processes, such as jet aeration, require grit removal.

Screening

Screening can be accomplished with coarse, fine, or micro screens. Course-screen bar spacing is usually 3/8-inch or more; fine screen spacing is 1/4-inch to 1/8-inch, and micro screen spacing is less than 1/8-inch, down to as small as 0.06 inches. Screenings are stored in a dumpster prior to their ultimate disposal in a landfill. Many garbage haulers and landfills require that screenings be washed to remove fecal material and compacted to reduce water content.

Coarse screening can be provided by manually cleaned bar screens, usually with spacing of about 1 inch, or with mechanically cleaned screens, usually with smaller spacing. Coarse screening usually results in less head loss than fine screening.

Fine screening equipment typically provides screening, washing, and compacting in one unit. The advantage to washed screenings is that fecal matter is returned to the waste stream for treatment in the secondary process. Compacted and washed screenings have significantly less weight, reducing the cost of screenings disposal.

Micro screening can be accomplished using a side hill screen, a static screen, or a mechanical rotating drum screen. Micro screening removes some grit. Micro screens typically have significantly more head loss than coarse and fine screens.

Preliminary Treatment and Flow Metering (Headworks) Recommendation

The following headworks improvements are recommended as part of this alternative:

- ***Screening.*** Two in-channel fine screens with washing and compaction are recommended, based on their lower capital cost and ease of operation. The units would be operated in parallel and each would be sized for half of the Phase 2 PIF. The screens would be located outside with a canopy cover. An emergency manually cleaned bar screen bypass would be located parallel to the screens, sized for half of the Phase 2 PIF.
- ***Flow Measurement.*** A larger Parshall flume would be installed and sized for the Phase 2 PIF.

The new headworks would likely be located south of the existing headworks; the new headworks could be built while the existing headworks remains in service. The increased capacity of the proposed facilities would require a significantly larger footprint than the existing headworks. The estimated total project cost for the proposed headworks is $630,000, which includes an allowance for removing the existing headworks.

Grit removal was originally recommended with the new headworks construction, but was removed from the recommendation due to cost considerations. For this unit process, a grit basin with bypass channel would be installed and sized for the Phase 2 PIF. Two dry pit grit pumps would be provided in a below-grade dry well. Each pump would discharge to a dedicated grit cyclone. Both cyclones would discharge to a common grit classifier. The grit cyclones and grit classifier would be located in a building. The grit room would be ventilated to meet OSHA requirements, but odor control facilities would not be provided. The required grit building would be approximately 300 square feet. The estimated total project cost for the proposed headworks including grit removal is $1,250,000. During the headworks design, the City must compare the additional cost of grit removal with the advantages of including this unit process. It was assumed for this report that grit removal will not be provided.

Transfer Pumping

The existing transfer pump station downstream of the aeration basin has insufficient capacity for existing wet weather flows. It is recommended that the existing pump station be replaced with a new station.

Wastewater influent pump stations must be sized for the PIF with redundancy. Redundancy means that if the largest pump is out of operation, the remaining pumps are

able to pass the peak flow. This prevents bypasses or overflows should a pump be out of service. For this application, however, the existing aeration basin tankage will no longer be used as an aeration basin and could be used for flow equalization. The aeration basin volume is 175,000 cubic feet (cf), or 1.3 million gallons (MG). It is assumed for planning purposes that use of the existing aeration basin for flow equalization would allow for sizing the pump station for the PDAF. This assumption must be confirmed during the design of the transfer pump facilities.

Forcemain

The existing forcemain is approximately 1,615 feet of 14-inch piping. At the Phase 2 PDAF, the resulting velocity in the forcemain is 16 feet per second (fps) while recommended velocities are between 4 and 6 fps. Not only is the existing forcemain severely undersized for design conditions, but it may have to be relocated in some areas to accommodate the locations of the new transfer pump station and new secondary treatment facilities. It is therefore recommended that two parallel 18-inch forcemains replace the existing forcemain.

Design of a new forcemain must address the potential for production of hydrogen sulfide. Hydrogen sulfide is produced when the biochemical oxygen demand exceeds the available oxygen. The critical flow condition for hydrogen sulfide production is at low flows when the pumps run infrequently and the detention time in the forcemain is at its longest. Hydrogen sulfide causes odors and, more critically, can corrode concrete.

The potential for hydrogen sulfide was evaluated for existing and proposed conditions. For existing conditions with the single 14-inch forcemain, it appears that there is a marginal potential for hydrogen sulfide production, but the City currently does not report any odors. This could be because the discharge point of the forcemain is always submerged and may dampen any odor problems. For the proposed conditions with the two parallel 18-inch forcemains, the likelihood of hydrogen sulfide production increases with the larger forcemain volume.

For this alternative, no hydrogen sulfide control is initially recommended, for the following reasons:

- Currently, there is no problem associated with hydrogen sulfide.
- The forcemains will discharge into aerated lagoons, which will immediately add oxygen to the influent flows.
- The potential for hydrogen sulfide production will decrease as flows to the plant increase.
- If odor problems arise, hydrogen sulfide control can be retrofitted into the system.

Pumps

It is recommended that three pumps be installed, each sized for half the Phase 2 PDAF, with the third pump providing redundancy. The pumps would each have a capacity of 5.4 mgd, or 3,800 gpm. When only one pump is running, as during summer conditions, only one of the 18-inch parallel forcemains would be used, for a resultant velocity of 4.8 fps when the

pump is running at full speed. When two pumps are running, as during winter conditions, both forcemains would be used.

It is recommended that the existing pumps be replaced with vertical turbine solids handling pumps. The new pumps would have variable frequency drives (VFDs). VFDs allow pump output to closely match system flow, providing more continuous flow to the secondary process than is possible with single-speed pumps and a normal fill/draw cycle. Continuous flow reduces the capacity requirements of the wet well and allows for more consistent plant operation.

Wet Well

The existing wet well is undersized for the Phase 2 PDAF and the proposed pumps discussed above. It is recommended that a new wet well be built. The new pump station would likely be located south of the existing pump station; the new pump station could be built while the existing pump station remains in service.

Electrical and Controls

Space open to raw sewage is classified as a National Electric Code (NEC) Class I space. Based on the ventilation provided, electrical equipment and controls must meet the applicable Class I division or be located outside the Class I space. It is assumed that the VFDs and controls would be located in a room adjacent to the classified space to avoid the requirement for explosion-proof rated equipment.

Transfer Pumping Recommendation

In addition to the pumping improvements described above, a new 600-square-foot room would be required. The estimated total project cost for the pumping improvements is $1,380,000.

Secondary Treatment

For this alternative, the existing treatment process would initially remain. To treat future Phase 2 flows, the equivalent of three times the existing aeration basin capacity is required.

There is only space for one additional aeration basin in the vicinity of the existing aeration basin: at the toe of Lagoon No. 2. Also, construction of an aeration basin in this location would be problematic due to the proximity of Lagoon No. 2. Since adequate aeration basin volume for Phase 2 flows and loads cannot be constructed in the vicinity of the existing aeration basin, it was assumed that the necessary aeration basin volume would be obtained by partitioning off a portion of Lagoon No. 1.

Secondary clarification and RAS and WAS pumping would be added as part of the Phase 2 improvements (in approximately 10 years) to convert the plant to an extended aeration activated sludge plant. With secondary clarification, flows from the aeration basin are diverted to clarifiers that separate the liquids and solids. A portion of the microorganisms that settle out in the clarifiers are returned to the aeration basins to be mixed with the incoming wastewater and are called RAS. The remaining solids are wasted to the sludge handling facilities and are called WAS.

An advantage of adding the secondary clarification and RAS/WAS pumping equipment is that the secondary treatment process would produce effluent close to 10 mg/l BOD and 10 mg/l TSS (10/10) year-round. Secondary effluent that bypasses the remaining facultative portion of the lagoons, thereby minimizing algae growth, may not need filtration prior to discharge to surface waters assuming 10/10 permit limits.

Secondary Treatment Recommendation

Secondary treatment facilities for this alternative would involve installing two 2.0 million gallon aeration basins as part of the Phase 1 improvements. At an operating depth of 12 feet, the plan area for each basin at the average water depth would be approximately 0.52 acres. For Phase 1 operation, aeration would be provided by a total of 120 horsepower of aerators. The existing 60 horsepower of aerators in the existing aeration basin could be moved to the new basins. The aerator motor starters and controls would be housed in the new pump station building (discussed above) or the existing operations building.

Phase 2 improvements would include secondary clarification in two 85-foot-diameter clarifiers. RAS/WAS pumping, blowers, and process controls would be housed in a building near the clarifiers. Another 60 horsepower of aerators would be added to the aeration basins.

The estimated Phase 1 total project cost, for the aeration basins and associated facilities, is $1,490,000. In ten years, the recommended secondary clarifiers and RAS/WAS facilities should be added at an estimated total project cost of $2,860,000 in 2000 dollars.

Advanced Treatment

The existing advanced treatment system consists of flocculation, a DAF unit, and gravity sand filters. This advanced treatment system is used to remove algae and other suspended solids from the facultative lagoon effluent when discharging to Bear Creek in the winter. As discussed in Chapter 5, the capacity of the advanced treatment system is approximately 2.2 mgd, which is undersized for the existing MMWWF of 3.1 mgd as well as the Phase 2 MMWWF of 5.4 mgd.

For the Phase 1 design condition, with the aerated basins in place but not the secondary clarifiers, winter flows would be routed through the aeration basins, the remaining facultative portions of the existing lagoons, and the advanced treatment facilities. The facultative lagoons would provide flow equalization such that the advanced treatment facilities could be sized for the Phase 1 MMWWF of 4.4 mgd. This would require doubling the capacity of the existing facilities, or adding one DAF unit, with associated flocculation equipment, and two filters. A different advanced treatment process with an equivalent capacity could also be used if proven to be more cost-effective.

For the Phase 2 design condition, the secondary clarifiers would be in place. Winter flows would be routed through the aeration basins and the clarifiers, but would bypass the facultative lagoons. This would ensure that the suspended solids in the winter flows do not increase with algae from the facultative lagoons, allowing the advanced treatment facilities to be bypassed. Since the extended aeration activated sludge process (aeration basin and secondary clarifiers) is likely to produce close to 10/10 effluent, only a portion of the winter

flows need to be filtered to ensure that 10/10 effluent is produced. As such, the 4.4 mgd filter capacity for the Phase 1 facilities will be adequate for Phase 2 operation.

Advanced Treatment Recommendation

The existing facilities are over 20 years old and in need of various upgrades to extend their service life. In addition to refurbishing the existing DAF unit and gravity sand filters, one new equal-size DAF unit with associated flocculation equipment and two new equal-size sand filters are required to meet the Phase 2 design condition. The estimated total project cost for these advanced treatment improvements is $2,140,000. As stated above, the advanced treatment facilities could also be replaced with 4.4 mgd capacity of another advanced treatment process if it is proven to be more cost-effective.

Effluent Disinfection

Effluent disinfection prevents the spread of waterborne diseases. To protect public health, effluent must be disinfected before it can be discharged to surface waters. There are four main options for effluent disinfection: gaseous chlorine, hypochlorite, ozone, and ultraviolet radiation.

Gaseous Chlorine

Municipalities often use chlorine gas for disinfection. The gaseous system consists of feeding chlorine gas by vacuum through a chlorinator that controls the gas flow rate at the injector. Chlorine gas and water are combined in the injector to form a solution that is diffused into the effluent. The chlorinated effluent flows into a chlorine contact chamber where disinfection occurs. Generally, contact times are 60 minutes at average flow and 20 minutes at peak flow.

Chlorine gas is classified as a hazardous material. Changes in safety regulations and insurance costs have significantly increased the cost of gaseous systems due to their hazardous nature. It is now generally required that containment and scrubbers for chlorine gas be provided according to the standards of the current Uniform Fire Code (UFC). These facilities contain and neutralize any chlorine gas leaks and prevent their release to the atmosphere. A chlorine scrubber uses another hazardous material—liquid caustic.

Chlorine and chlorine residuals are toxic to aquatic life; therefore, dechlorination is often required before discharging to surface waters. Dechlorination is a process to neutralize residual chlorine in the effluent. Dechlorination can be accomplished using sulfur dioxide gas, sodium bisulfite solution, or dry bagged salts. Solid salts are rarely used anymore because safety issues require extreme caution on the part of the operator when handling and mixing the salts into liquid. A concern with dechlorination when using sulfur dioxide or sodium bisulfite is the subsequent reduction of dissolved oxygen in the effluent since the dechlorinating agent often must be overdosed to ensure adequate dechlorination. Carefully controlling dosages of the dechlorinating agent based on chlorine residual and flow is very important to prevent exerting an oxygen demand on the receiving water.

The existing treatment plant uses gaseous chlorine for disinfection, but disinfection is not needed prior to surface water discharge due to the advanced treatment facilities and the

long residence time in the lagoons. With the recommended improvements to the plant and the future increase in wastewater flows, disinfection will be required year-round. Dechlorination prior to surface water discharge will be required if the use of chlorine is continued. Adding dechlorination would qualify as a change to the existing system and would require that containment and scrubbers for chlorine gas be provided. Evaluation would be required to determine if additional chlorine contact volume would be needed.

Hypochlorite

Hypochlorite (bleach) can be fed into effluent using a pumped system or an eductor system that operates like the injector system used for gaseous chlorine. As with gaseous chlorine disinfection, the chlorinated effluent flows into a chlorine contact chamber where disinfection occurs. Dechlorination, as discussed above, is also typically required when effluent is discharged to a surface water body.

Equipment suppliers and recent studies by consultants have shown that disinfection with hypochlorite is now more economical and safer (on a catastrophic basis) than using chlorine gas in small communities. Exposure to hypochlorite, however, is a concern with regard to burns to skin. Hypochlorite is available as a liquid solution or as a solid in tablet form that must be dissolved prior to use. Solid salts are rarely used anymore since safety issues require extreme caution on the part of the operator when handling and mixing the hypochlorite (for chlorination) and sulfite salts (if required for dechlorination). There is also equipment available for on-site hypochlorite generation on a scale suitable for small communities. Hypochlorite loses strength with age and has a useful shelf life of approximately 45 days.

Disinfection will be required year-round due to the recommended improvements to the plant and the future increase in wastewater flows. As with the gaseous chlorine option, dechlorination prior to surface water discharge will be required if hypochlorite is used. Evaluation would be required to determine if additional chlorine contact volume would be needed.

Ozone

Ozone is an unstable gas that must be generated on-site from air or oxygen. Ozone injection into the effluent flow can be accomplished by a variety of methods, including mechanical mixing and jet injection. Ozone reacts quickly and requires a relatively short contact time.

In the past, ozone has been associated with disinfection in water systems. When adequate levels of ozone are used, disinfection is much more complete than with chlorine disinfection. While ozone disinfection can be feasible for large wastewater plants, construction and operating costs are relatively high for smaller systems. Ozone disinfection of wastewater is almost nonexistent in the United States, but is more common in Europe.

Ultraviolet Radiation

A typical ultraviolet (UV) disinfection system consists of modular UV lights along an open channel or in a closed conduit. The disinfection process of a UV system occurs in a very short time; therefore, a separate contact chamber is not necessary.

Disinfection with UV light has been feasible for many years, but it has only become accepted and economically competitive in recent years. UV systems are much safer than other disinfection options. Operation and maintenance generally consists of periodically cleaning and replacing UV bulbs. UV disinfection systems are relatively safe and do not expose the operator to chemicals.

UV disinfection requires consistently high effluent quality to guarantee effectiveness. The Phase 1 use of the advanced treatment facilities and the future activated sludge secondary process of this alternative would provide adequate and consistent effluent quality for UV disinfection. Any continued use of the existing lagoons, however, will provide a source of algae, thereby requiring the operation of the advanced treatment. A UV system cannot function with the amount of algae typical from lagoons after an algae bloom.

Disinfection Recommendation

Ozone disinfection was excluded from further consideration because of its high construction and operation costs. UV disinfection was excluded because the effluent out of the existing lagoons could not be adequately disinfected if the advanced treatment facilities are bypassed, as they are likely to be during the summer. Of the remaining two disinfection alternatives, gaseous chlorine and hypochlorite, operator preference is for gaseous chlorine. The main reasons for recommending gaseous chlorine are that the existing system is gaseous chlorine and that there are limitations on hypochlorite storage.

The gaseous chlorine system would use the existing chlorine feed system to continue feeding an average chlorine dosage of approximately 6 mg/l. A chlorine residual analyzer and feed controls would be added for dechlorination. Two liquid sodium bisulfite pumps for dechlorination would be installed. A scrubber would be located near the chlorine room.

Sufficient chlorine contact volume would be required to provide 20 minutes of contact time at the peak flow and 60 minutes of contact time at average flows. Discharge flows to Bear Creek are dependent on the BOD concentration in the effluent; as the BOD concentration drops, the allowable discharge increases. Assuming a minimum 2 mg/l BOD in the effluent and the permitted maximum monthly average mass load limit of 66 ppd BOD, the resulting peak flow to Bear Creek is 3.95 mgd. This would require 54,900 gallons of chlorine contact volume at a 20-minute detention time. The average flow would be based on a higher BOD concentration in the effluent, which would likely be around 5 mg/l, for a resulting average flow to Bear Creek of 1.58 mgd. This would require 65,900 gallons of chlorine contact volume. The existing chlorine contact basin volume is adequate to provide these required contact times.

Flows discharged to the Molalla River (an option discussed in Chapter 8) would use approximately three miles of the transmission main from the plant to the river for contact time. As a result, no additional dedicated chlorine contact facilities would be required. Use of the transmission pipeline for chlorine contact would require a remote dechlorination and effluent sampling station at the end of the chlorine contact volume and reaeration facilities downstream of the dechlorination. These facilities are included in the cost estimates for the effluent disposal evaluation in Chapter 8.

The estimated total project cost for the gaseous chlorine facility upgrades is $380,000.

Other Facilities

The overall treatment process improvements will require other appurtenant new facilities including the following:

- ***Standby Power.*** Standby power generation must be provided at the plant. DEQ has indicated that Class I reliability is required; therefore, under emergency conditions, the standby power source must be able to operate the entire plant. It is estimated that a 750-kilowatt generator would be required to run the entire plant. The existing standby generator has a 150-kilowatt capacity and will not be adequate to operate the entire plant and is also at the end of its service life. A standby generator is required with the effluent/irrigation pumping facilities discussed in Chapter 8. The generator provided with the effluent/irrigation pumping facilities will be sized to accommodate the entire plant. As a result, no additional costs for standby power need be included with this treatment alternative.
- ***Office/Laboratory.*** The existing office/laboratory, a corner of the operations building, is over twenty years old, as is the laboratory equipment. It is recommended that an allowance be made to renovate the office/laboratory and purchase new equipment during the other treatment facilities upgrade. The total budget allowance for the office/laboratory improvements is $50,000.

Summary of Alternative to Upgrade the Plant Treatment Processes

The process recommendations for the alternative to upgrade the existing plant treatment processes are listed in Table 7-3 with associated cost estimates.

TABLE 7-3.
COST ESTIMATE FOR ALTERNATIVE TO EXPAND THE EXISTING PLANT

Treatment Facility Component	Cost Estimate
PHASE 1 FACILITIES	
Preliminary Treatment—Fine Screen	$630,000
Transfer Pumping—Vertical Turbine Solids Handling Pump Station	$1,380,000
Secondary Treatment—Aeration Basins	$1,490,000
Advanced Treatment—Expand Capacity	$2,140,000
Effluent Disinfection—Gaseous Chlorination and Dechlorination	$380,000
Other Facilities—Miscellaneous Improvements	$50,000
Total	**$6,070,000**
ADDITIONAL PHASE 2 FACILITIES	
Secondary Treatment—Secondary Clarifiers and RAS/WAS (2000 Dollars)	$2,860,000

7.2.2 Convert to an Oxidation Ditch Plant

For the alternative to convert to an oxidation ditch secondary treatment process, the following improvements would be required:

- New preliminary treatment facilities and influent flow metering (headworks) located in the vicinity of the existing headworks.
- New transfer pumping facilities located in the vicinity of the existing transfer pump station.
- New secondary treatment facilities (oxidation ditches and secondary clarifiers) located south of Lagoon No. 1.
- Upgraded disinfection equipment located in the vicinity of the existing control building.

This alternative's recommendations for preliminary treatment, transfer pumping, and miscellaneous improvements are the same as those developed for the alternative to upgrade the existing plant treatment processes. Details and cost estimates for this alternative with regard to secondary treatment, advanced treatment, and effluent disinfection are included below.

Secondary Treatment

For this alternative, the plant's secondary treatment process would be converted to an oxidation ditch—a continuous flow, extended aeration activated sludge treatment process. An oxidation ditch uses a long, continuous channel that is oval or circular. Oxidation ditches provide an oxic (oxygen-rich) environment where oxidation and nitrification occur. The oxic environment can be created by low-speed surface aerators that also serve as mixers or by aeration diffusers and mixers. Secondary clarification is separate from the oxidation ditch. Oxidation ditch systems provide excellent treatment. One drawback is that the system typically requires more land than an SBR process, which is discussed below.

Secondary Treatment Recommendation

The oxidation ditch secondary treatment alternative involves installing two 2.0-million gallon basins for a Phase 2 MMDWF capacity of 4.0 mgd. The footprint for the oxidation ditch concrete tanks would be approximately 220 feet by 200 feet. Aeration rotors would be used in the ditches. There would also be two 85-foot-diameter secondary clarifiers. RAS/WAS pumping, blowers, and process controls would be housed in a building of approximately 3,000 square feet. The estimated total project cost for the oxidation ditch secondary treatment is $7,370,000. This includes an allowance for site work.

Advanced Treatment

With the treatment improvements of this alternative, advanced treatment is not required to meet current discharge limits. The oxidation ditch plant will reliably meet the 10/10 monthly average effluent concentrations of the current permit, which are standard for summer discharge in the Willamette River basin. Winter discharge requirements are not likely to be more stringent than summer basin standards.

To maintain 10/10 quality, the effluent cannot be detained in the existing lagoons, where algae blooms would cause violation of the TSS discharge requirements. All effluent produced during the summer will have to be land applied rather than stored for winter discharge, and all effluent produced during the winter will have to be discharged without storage. No effluent stored in lagoons can be discharged to surface waters under this alternative because of the lack of advanced treatment to remove algae.

Advanced Treatment Recommendation

It is recommended that use of the existing advanced treatment system be discontinued for this alternative. The oxidation ditch process of this alternative will provide adequate treatment to meet effluent disposal requirements assuming that the effluent disposal facilities recommendations in Chapter 8 are implemented.

Effluent Disinfection

Effluent disinfection alternatives were evaluated for the alternative to upgrade the plant treatment processes. The recommendation for continued use of gaseous chlorination facilities is also appropriate for this alternative.

The chlorine contact requirements for this alternative are different from the requirements for the alternative to upgrade the plant treatment processes because the advanced treatment facilities will be eliminated for this alternative, so stored summer flows will no longer be discharged during the winter months. As such, the peak discharge to Bear Creek will be less for this alternative, and less chlorine contact time will be required. This does not change the required improvement, however, because in both alternatives the existing contact tank is adequate for the flows that will be discharged to Bear Creek. The transmission line will provide contact time for flows discharged to the Molalla River.

Effluent Disinfection Recommendation

As described for the alternative to upgrade the plant treatment processes, the estimated total project cost for the gaseous chlorine facility upgrades is $380,000. This cost includes dechlorination at the plant for Bear Creek discharges.

Summary of Alternative to Convert to an Oxidation Ditch Plant

The process recommendations for the alternative to convert to an oxidation ditch secondary process are listed in Table 7-4 with associated cost estimates.

TABLE 7-4.
COST ESTIMATE FOR ALTERNATIVE TO CONVERT TO AN OXIDATION DITCH PLANT

Treatment Facility Component	Cost Estimate
Preliminary Treatment—Fine Screen	$630,000
Transfer Pumping—Vertical Turbine Solids Handling Pump Station	$1,380,000
Secondary Treatment—Oxidation Ditch and Secondary Clarifiers	$7,370,000

Advanced Treatment—Abandon Existing Facilities		$0
Effluent Disinfection—Gaseous Chlorination and Dechlorination		$380,000
Other Facilities—Miscellaneous Improvements		$50,000
	Total	**$9,810,000**

7.2.3 Convert to an SBR Plant

For the alternative to convert to an SBR secondary treatment process, the following improvements would be required:

- New preliminary treatment facilities and influent flow metering (headworks) located in the vicinity of the existing headworks.
- New transfer pumping facilities located in the vicinity of the existing transfer pump station.
- New secondary facilities (SBRs) located south of Lagoon No. 1.
- Upgraded disinfection equipment located in the vicinity of the existing control building.

This alternative's recommendations for preliminary treatment, transfer pumping, advanced treatment, effluent disinfection, and miscellaneous improvements are the same as those developed for the alternative to convert to an oxidation ditch process. Details and cost estimates for this alternative with regard to secondary treatment are included below.

Secondary Treatment

An SBR is an extended aeration activated sludge system that uses two or more parallel basins, often operated in a batch treatment mode. All phases of an SBR cycle take place in a single basin. There are several SBR process manufacturers and each process is somewhat different from the rest. In general, the phases of the SBR cycle are as follows: anoxic fill, aerated fill for some applications, react, settle, decant, and idle/waste sludge. During the anoxic fill phase, the basin is filled with wastewater. Aeration begins during or after fill and continues through the react phase. The aerators are then turned off and the biomass is allowed to settle. During the decant phase, treated effluent is removed from the basin by a decanter. Finally, settled sludge is pumped from the basin for sludge treatment while the basin waits to receive the next batch of wastewater flow and repeat the cycle. Most SBR systems require an equalization basin downstream of the SBR tanks to equalize the flow from the SBR to the disinfection and disposal facilities.

The SBR process requires only one basin for both aeration and clarification. SBR systems are computer-controlled and some systems are very complex and mechanically-intensive. Other systems are more straightforward with the exception of the computer control system. Effluent quality from an SBR is typically very high.

Secondary Treatment Recommendation

SBR secondary treatment would involve installing two 2.0-million-gallon basins for a Phase 2 MMDWF capacity of 4.0 mgd. The footprint of the SBR concrete tanks would be approximately 315 feet by 105 feet. There would also be a 315-by-24-foot equalization basin. The blowers and process controls would be housed in a building of approximately 3,000 square feet. The estimated total project cost for the SBR treatment is $7,440,000. This includes an allowance for site work.

Summary of Alternative to Convert to an SBR Plant

The process recommendations for the alternative of converting to an SBR plant are listed in Table 7-5 with associated cost estimates.

TABLE 7-5.
COST ESTIMATE FOR ALTERNATIVE TO CONVERT TO AN SBR PLANT

Treatment Facility Component	Cost Estimate
Preliminary Treatment—Fine Screen	$630,000
Transfer Pumping— Vertical Turbine Solids Handling Pump Station	$1,380,000
Secondary Treatment—SBR	$7,440,000
Advanced Treatment—Abandon Existing Facilities	$0
Effluent Disinfection—Gaseous Chlorination and Dechlorination	$380,000
Other Facilities—Miscellaneous Improvements	$50,000
Total	**$9,880,000**

7.2.4 Treatment Process Recommendation

The primary consideration for selecting between the improvement alternatives is typically cost. A present worth analysis prepared to compare the alternatives is shown in Table 7-6. The analysis is based on a 20-year period and an annual 6 percent interest rate. Future project costs are inflated at an annual rate of 4 percent. The three alternatives have essentially equal present worth values.

TABLE 7-6.
PRESENT WORTH ANALYSIS OF TREATMENT PROCESS ALTERNATIVES

	Upgrade Plant Treatment Processes	Convert to Oxidation Ditch Plant	Convert to SBR Plant
TOTAL PROJECT COSTS			
Initial Construction (2000 Dollars)	$6,070,000	$9,810,000	$9,880,000
Add'l Phase 2 Construction (2000 Dollars)	$2,860,000	$0	$0
Add'l Phase 2 Construction (2010 Dollars)	$4,230,000	$0	$0
Add'l Phase 2 Present Worth (2000 Dol.)	$2,360,000	$0	$0
Total Project Costs (2000 Dollars)	$8,430,000	$9,810,000	$9,880,000
ANNUAL COSTS			
Startup Annual Costs (2000 Dollars)	$304,000	$214,000	$248,000
Phase 2 Annual Costs (2000 Dollars)	$389,000	$292,000	$328,000
Present Worth of Annual Costs (2000 Dol.)	$6,000,000	$4,410,000	$5,000,000
Total Present Worth Value (2000 Dol.)	**$14,430,000**	**$14,220,000**	**$14,880,000**

Selection criteria other than cost are as follows:

- Ease of Operation—The ease of operation for each alternative is roughly reflected in the annual costs of each; the lower the annual costs, generally the easier the operation. The oxidation ditch plant would have the least equipment and controls and would be the easiest to operate with ditch aeration rotors, clarifiers, and RAS/WAS pumping equipment. The SBR plant has aeration blowers, flow equalization equipment, and complex controls for the treatment stages in the SBR basins. The upgrade of the plant treatment processes would have the most equipment, including the aerators, the DAF units with auxiliary equipment and chemical feed requirements, the gravity filters, and eventually the clarifiers and RAS/WAS equipment.
- Winter Water Quality—The alternatives are all sized to reliably achieve 10/10 effluent during the winter for the 2019 design condition. The alternative to upgrade the plant treatment processes also includes the advanced treatment facilities that would allow for reliably achieving 5/5 effluent for up to 4.4 mgd. The SBR would also likely reliably meet better than 10/10 effluent.
- Summer Water Quality—The alternative to upgrade the plant treatment processes is the only alternative that will allow for the production of Level IV reclaimed water (effluent) during the summer because it is the only one that provides coagulation and filtration with the advanced treatment

facilities. The higher level of treatment will provide more flexibility in where the City disposes of the effluent.

- Phasing—The only alternative that readily lends itself to facility phasing over the 20-year planning period is the upgrade of the plant treatment facilities. All other alternatives could not be cost-effectively phased toward the 2019 design condition. The phasing of facilities allows flexibility so that future expenditures are made based upon actual growth.
- Future Expansion Potential After 20-Year Period—All alternatives use treatment processes that are modular and can be expanded by adding similar facilities. The alternative to upgrade the plant treatment processes would be the most difficult and expensive to expand after 2019 with regard to the advanced treatment processes. The 2019 design for this alternative includes two DAF units and four filters. As more units are added, the piping and controls between all the units becomes complex. It is likely that any future expansion of the facilities would involve replacing these units with larger facilities more suited to the higher future flows.

In conclusion, the alternative to expand the plant using existing treatment processes has the significant advantage of higher summer water quality over the other alternatives. The 20-year cost-effective phasing of facilities for this alternative is another advantage. Since this alternative also has approximately the same present worth as the other alternatives, it is recommended that the City upgrade the existing plant treatment processes.

7.3 SOLIDS MANAGEMENT

Management of solids from the treatment plant encompasses several components: sludge treatment, volume reduction by thickening or dewatering, and sludge disposal. All components must work together and adhere to the standards set forth in the *Code of Federal Regulations* (40 CFR Part 503 and 40 CFR Part 257; See Chapter 4.)

The evaluation of the solids management alternatives was based on the recommendation for liquid stream treatment facilities discussed above. The recommended upgrade of the existing plant treatment processes includes continued use of aeration basins and the facultative lagoons for Phase 1. During this period, no solids management facilities are needed and solids could be dredged prior to the Phase 2 construction. For Phase 2, the addition of the secondary clarifiers will require facilities for management of the WAS. For this study, three solids management alternatives were evaluated:

- Facultative Sludge Lagoon
- Aerobic Digestion and Facultative Sludge Lagoon
- Aerobic Digestion and Lime Stabilization.

7.3.1 Facultative Sludge Lagoon

For the facultative sludge lagoon (FSL) alternative, a portion of Lagoon No. 1 would be converted to a FSL. Treatment, thickening, and short-term storage of the solids would be accomplished within the FSL.

Treatment

An FSL is an open-air retention pond or lagoon that stores the solids for an extended period. Treatment occurs through a combination of anaerobic and aerobic processes. Solids decay anaerobically on the bottom of the lagoon, while gases and other byproducts from the decaying solids are scrubbed and stabilized aerobically in a water cap over the solids. The digestion of the solids in an FSL can provide reductions in volatile solids of up to 50 percent. Thickening also takes place by gravity settling of the solids and draining the lagoon supernatant back to the treatment facilities.

FSLs typically have a depth of 10 to 15 feet. A 3- to 4-foot aerobic water cap is required on the surface to prevent odors. FSLs are often designed with a solids retention time (SRT) of two years or more and multiple feed points to provide flexibility in solids disposal. FSLs are usually lined with clay or a flexible membrane to prevent leaching and subsequent contamination of groundwater.

Volume Reduction

Thickening takes place by gravity in an FSL. Due to the facilities' extended storage capacity, no other volume reduction measures are required.

Disposal

The solids in the FSL would be periodically dredged and land applied. Disposal options depend on the type of solids treatment provided. Solids must be treated to meet either Class A or Class B standards (see Chapter 4). Class A solids have very few restrictions on disposal because they are sufficiently treated for distribution to the public for use on lawns or home gardens. Class B solids must be applied to sites with restricted access and use. For the FSL alternative, the treatment provided will produce Class B solids.

It is assumed that the City would contract out the solids dredging and disposal operations since it would not be cost-effective to purchase, maintain, and operate equipment that would be used as infrequently as several weeks per year. It may be possible to land-apply the solids on an effluent irrigation site, depending on the type of crop that is grown.

Summary

The FSL would be sized for a minimum two-year SRT at the Phase 2 design loads and for a limit of 20 pounds volatile suspended solids (VSS) per day per 1,000 square feet of lagoon surface area. Assuming 85 percent sludge yield from the secondary treatment facilities, the required FSL surface area is 132,000 square feet, or 3.0 acres. Assuming an 8-foot sludge depth, 10 percent solids content, and 38 percent VSS destruction in the FSL, the SRT at the Phase 2 design loads, based on the required FSL surface area, is 8.3 years. This means that VSS surface loading is the limiting factor for FSL sizing.

The estimated total project cost for this sludge management alternative is $770,000. This includes associated sludge pumps, piping, and electrical/controls.

7.3.2 Aerobic Digestion with Facultative Sludge Lagoon

For the aerobic digestion with FSL alternative, aerobic digesters would be added and a portion of Lagoon No. 1 would be converted to an FSL. Solids would be treated in the digesters and the FSL. The addition of aerobic digesters upstream of the FSL would reduce the loading on the FSL, thereby reducing the required FSL area.

Treatment

In the aerobic digestion process, volatile solids are reduced in a tank that is supplied with oxygen, normally by means of blowers and diffusers or floating aerators. Federal pathogen reduction regulations for municipal sludge require a minimum SRT of 40 days at 20ºC or 60 days at 15ºC. Further stabilization would be achieved in the FSL.

Volume Reduction

Mechanical thickening upstream of the digesters could reduce the digester volume; however, this is not likely to be cost-effective on either a project cost or annual cost basis.

Thickening can take place in the FSL. Some thickening can also be accomplished in the digesters by draining off the supernatant. Due to the extended storage capacity of the FSL, no other volume reduction measures are required.

Disposal

Like the FSL alternative, this alternative will produce Class B solids. It is assumed that the City would contract out the solids dredging and disposal operations, at least in the short-term. It may be possible to land-apply the solids on an effluent irrigation site, depending on the type of crop that is grown.

Summary

Assuming a solids depth of 12 feet in the aerobic digester, 2 percent solids content, and a 85 percent sludge yield from the secondary treatment facilities, this alternative would include two 82-foot-diameter aerobic digesters with a combined SRT of 60 days. Aeration would be provided by surface aerators.

The FSL for this alternative would be sized for a minimum of two-year SRT at the Phase 2 design loads or for a limit of 20 pounds VSS per day per 1,000 square feet of lagoon surface, whichever results in the larger lagoon. Assuming 38 percent VSS destruction in the aerobic digesters, the required FSL surface area is 82,000 square feet, or 1.9 acres. Assuming an 8-foot sludge depth, 10 percent solids content, and an additional 10 percent VSS destruction in the FSL, the SRT at the Phase 2 design loads, based on the required FSL surface area, is 5.2 years. This means that the VSS surface loading is the limiting factor with regard to FSL sizing.

The estimated total project cost for this sludge management alternative is $2,260,000. This includes associated sludge pumps, piping, and electrical/controls.

7.3.3 Aerobic Digestion with Lime Stabilization

The aerobic digestion with lime stabilization alternative requires aerobic digesters, dewatering equipment, and lime feed equipment. Solids would be treated by lime stabilization.

Treatment

Solids treatment can be achieved by adding lime in sufficient quantity to raise the pH of the solids to 12 or higher, creating an environment in which microorganisms cannot survive. Prior to lime stabilization, solids are typically stored in an aerated basin until enough is accumulated to warrant operating the dewatering and lime stabilization equipment.

Lime stabilization requires several pieces of equipment and has relatively high operation and maintenance requirements. Lime storage must be near the point of lime application. Lime can be stored in a silo and delivered in bulk quantities of 30 tons per delivery. Also available are 600-pound and 1,200-pound lime hoppers. Bags of lime, typically 35 pounds, must be added to the hopper manually. Delivery of a palette of lime is 70 bags at 35 pounds per bag for a total of 1.25 tons per delivery.

Volume Reduction

The solids are typically dewatered prior to adding the lime. With dewatering, suspended solids in the sludge can be increased from 1 or 2 percent to 15 or 20 percent. The solids can be dewatered with or without prior thickening; thickening prior to dewatering increases the dewatered cake concentration up to 20 percent solids and, in some cases, the size of dewatering equipment can be reduced. Dewatering can be accomplished using a belt filter press or centrifuge.

For this size of plant, a belt filter press is recommended. Equipment required in addition to the press would be polymer storage, sludge pumps, polymer pumps, a flocculation tank, and sludge cake conveyance facilities. The press and equipment would be housed in a small building. Piping would be required to drain the pressate to the plant headworks. Dewatering equipment is typically operated only during working hours and requires significant operator attention during operation.

Disposal

Unlike the other solids-management alternatives, this alternative can produce Class A solids. Also, the solids would be dewatered sludge cake rather than liquid solids. The sludge cake can be loaded into a truck, hauled to an application site, and flung onto the site as with a manure spreader. It is assumed that the City would contract out the solids hauling and disposal operations, at least in the short-term. It may be possible to land-apply the solids on an effluent irrigation site.

Summary

For this alternative, one aerobic digester with accessible aeration diffusers would be installed, sized for an SRT of 10 days. This results in a 74-foot diameter digester, assuming

a solids depth of 10 feet, 1 percent solids content, and an 85 percent sludge yield from the secondary treatment facilities.

A belt filter press would be used for dewatering prior to adding lime. Using the press would increase the solids concentration to approximately 15 percent solids. Lime requirements are typically 0.2 to 0.3 pounds of lime for every dry pound of solids. Lime would be added to the sludge cake and mixed into the solids and then conveyed to the storage area. The press and the lime feed equipment would be housed in a building adjacent to the solids storage. The belt filter press would be sized to accommodate the Phase 2 design loads, since phasing the press equipment would not be cost-effective. A silo would be required to store the lime.

Solids storage capacity should be on the order of one year to provide maximum flexibility for solids disposal. The limed solids would not produce odors. At an 8-foot storage depth and 15 percent solids content, the footprint of a one-year SRT storage area for Phase 2 loading would be approximately 200 feet by 78 feet. The solids storage area would be covered with a canopy to protect the stored solids from rain and snow.

The estimated total project cost for this alternative is $3,360,000. This includes the digesters, the belt filter press, and lime feed system with associated equipment including a building, dewatered solids conveyance facilities and the dewatered solids storage area, and all piping and electrical/controls.

7.3.4 Solids Management Recommendation

The three solids management alternatives were compared to determine a recommendation. The main criterion for selection of a solids management option is cost. Table 7-7 shows the present worth estimates for each alternative through Phase 2. The FSL is by far the least expensive alternative to construct and it would have significantly less annual cost than the other alternatives. Based on the cost information alone, the FSL is the recommended alternative.

TABLE 7-7.
COST COMPARISON OF SOLIDS MANAGEMENT ALTERNATIVES

	Facultative Sludge Lagoon	Aerobic Digestion with FSL	Aerobic Digestion with Lime Stab.
TOTAL PROJECT COSTS			
Phase 2 Capital Costs (2000 Dollars)	$770,000	$2,260,000	$3,360,000

The solids management facilities are not required until Phase 2. It is recommended, however, that when Lagoon No. 1 is drained for the Phase 1 construction of the aeration basin diking, the diking for the FSL should also be installed. Piping would be installed to allow this cell to be operated as a small facultative lagoon. The total project cost estimate for this Phase 1 work is $350,000. During Phase 2 construction, the cell would be drained and the liner for the FSL and piping to the FSL from the secondary clarifiers would be installed. The total project cost estimate for this Phase 2 work is $420,000.

Biosolids Management Plan

The City prepared a Biosolids Management Plan in 1999. Once the facilities recommended above are in place, the Biosolids Management Plan must be updated. As influent loads to the plant increase, a regular program of solids disposal must be established.

Application sites can include both forest and pasture or other non-forest lands. Solids application can be by spraying liquid or thickened solids from a fixed or truck-mounted cannon, injecting solids below the surface, or spreading solids on the surface and incorporating them into the soil. Access trails must be provided on the site appropriate to the type of application. If a truck-mounted system is used, access trails are needed at intervals of no more than 250 feet.

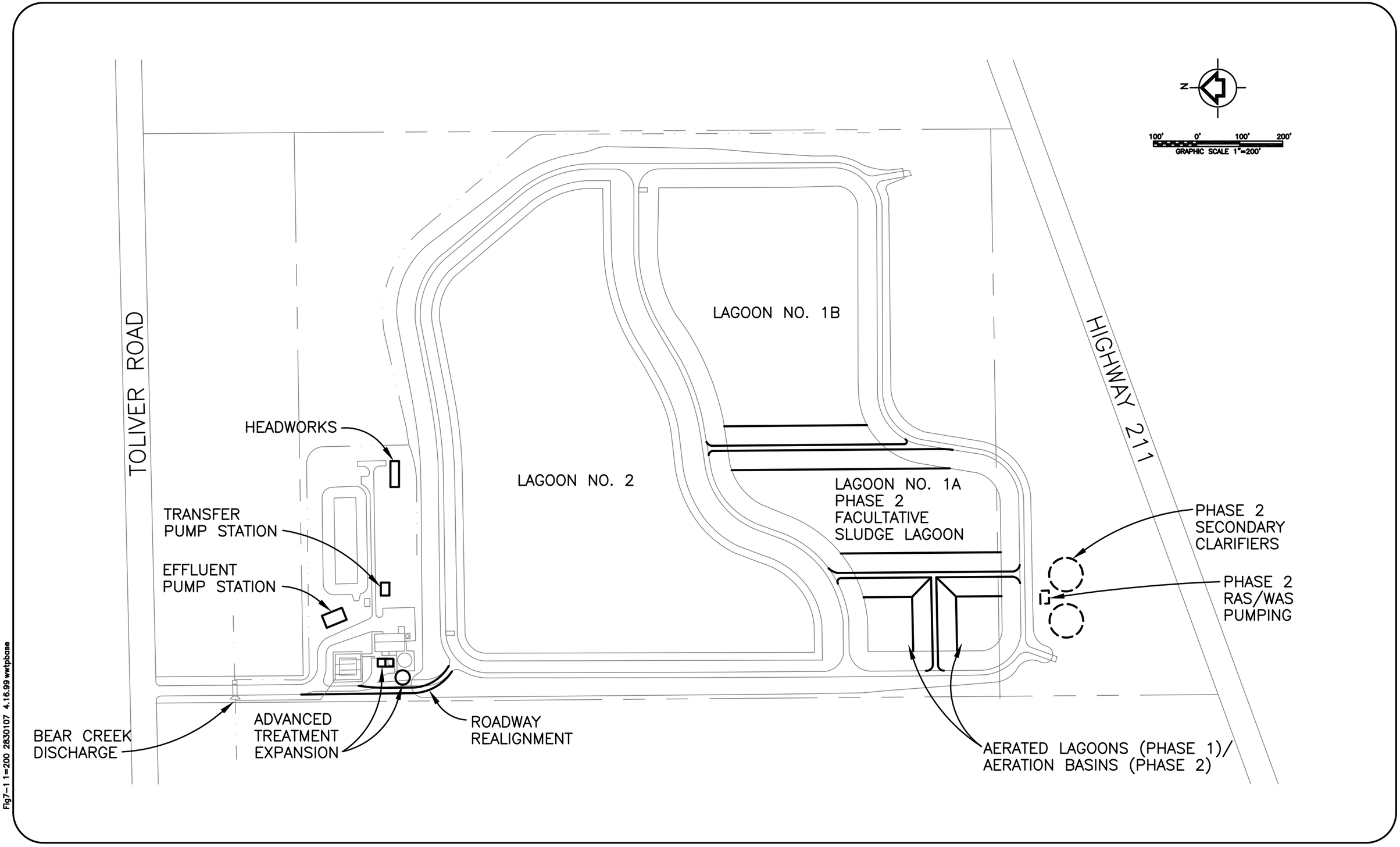
When evaluating application sites, physical conditions to consider include topography, transportation options and forest access, soils and geology, vegetation, water resources, and climate. Regulations pertaining to application sites from the Oregon Administrative Rules Chapter 340 Division 50 include the following:

- Slope—In general, liquid biosolids should not be surface-applied on bare soils where the ground slope exceeds 12 percent. Well vegetated sites with slopes up to 30 percent may be used for cake biosolids.
- Soil Depth—Soil should have a minimum rooting depth of 24 inches.
- Soil Type—Sites with saline and/or sodic soils should be avoided.
- Depth to Groundwater—For application of liquid biosolids, the minimum depth to permanent groundwater should be 4 feet and the minimum depth to temporary groundwater should be 1 foot.
- Buffers—Recommended buffers are as follows:
  - Surface water or ditch: 50 feet
  - Domestic water source or Well: 200 feet
  - Road: Varies depending on type and amount of traffic.
  - Residential Area: Buffer strip large enough to prevent nuisance odors or, for application of liquid biosolids, wind drift.

7.4 WASTEWATER TREATMENT PLANT RECOMMENDATION

The overall wastewater treatment plant recommendation is the sum of the recommendations in this chapter. The improvements include upgrading the existing plant treatment processes and the addition of a facultative sludge lagoon for solids management. Planning-level design criteria for the recommendations are included in Table 7-8 and the recommended site layout, flow schematic, and hydraulic profile are shown in Figures 7-1, 7-2, and 7-3, respectively. (See Chapter 8 for the effluent disposal facilities shown in Figures 7-1, 7-2, and 7-3.)

A summary of the Phase 1 treatment plant improvements is as follows:



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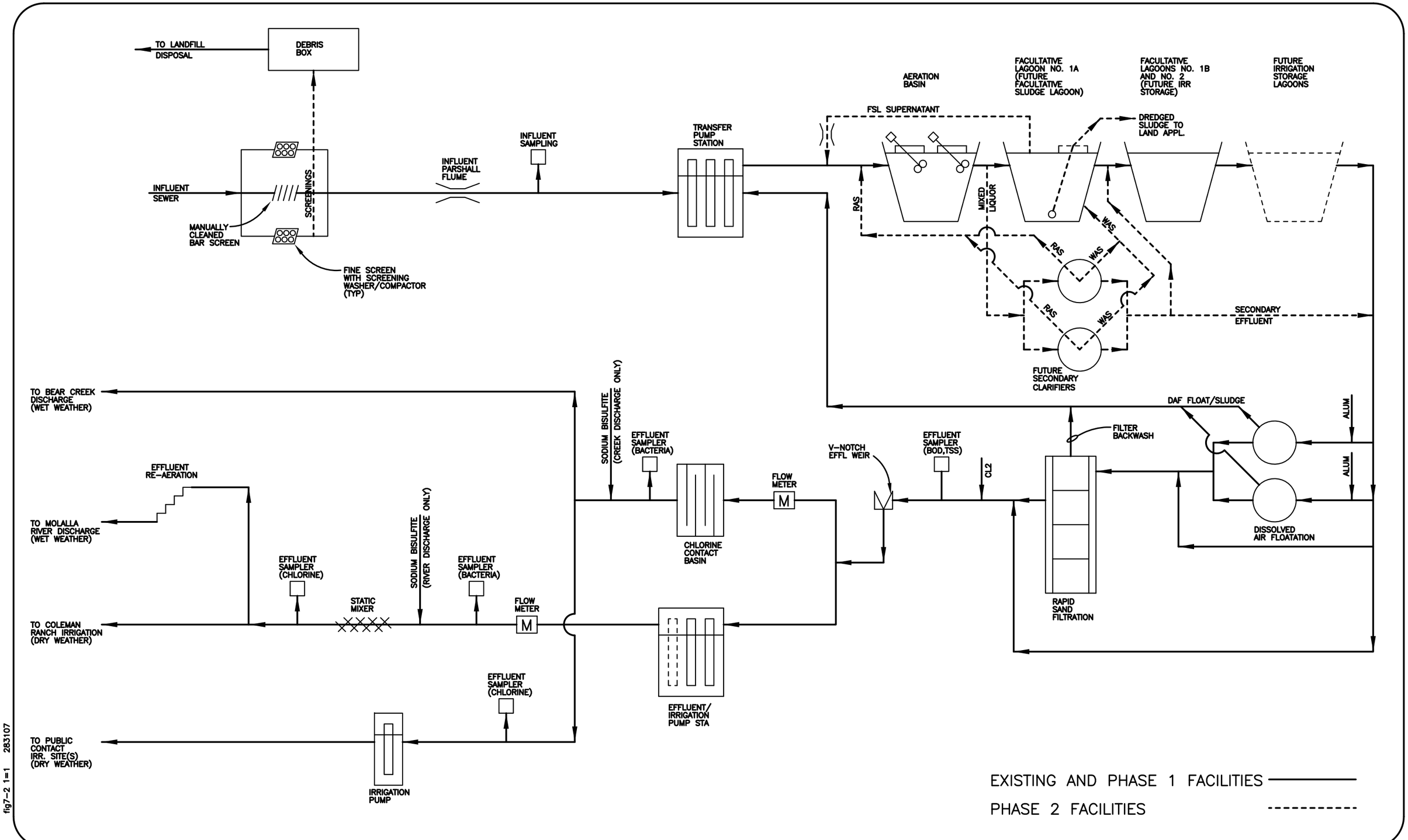
City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 7-1
SITE LAYOUT FOR THE
RECOMMENDED TREATMENT FACILITIES



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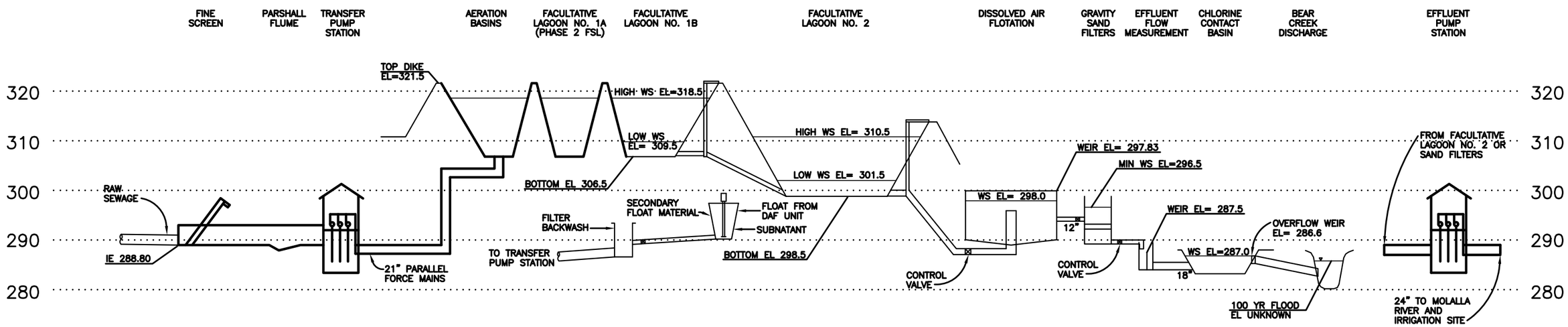
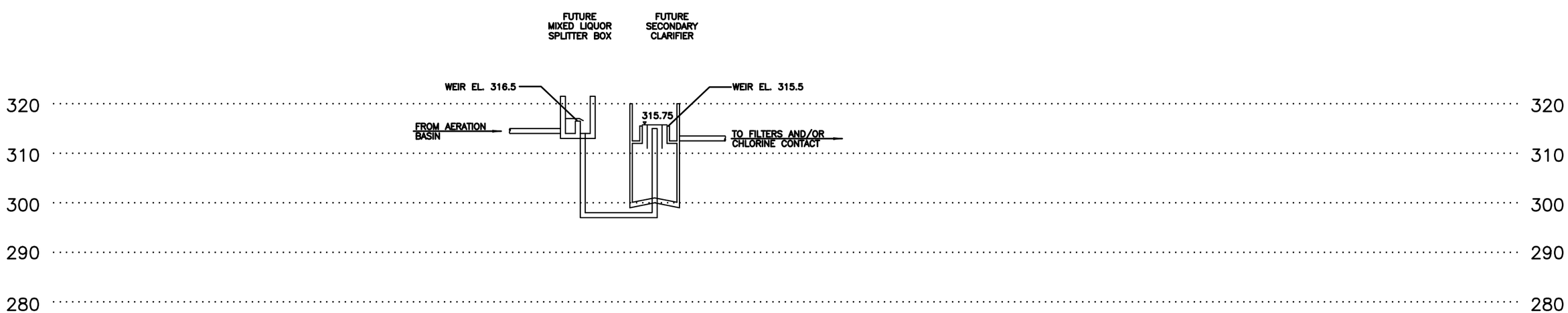
City of Molalla, Oregon WASTERWATER FACILITIES PLAN	FIGURE 7-2 PROCESS SCHEMATIC

PHASE 1 RECOMMENDATIONS



PHASE 2 RECOMMENDATIONS



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City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 7-3

HYDRAULIC PROFILE FOR THE RECOMMENDED TREATMENT FACILITIES

- Preliminary Treatment—Installation of facilities for fine screening with washing and compaction and flow measurement. (Cost estimate is $630,000.)
- Transfer Pumping—New pump station with vertical turbine solids handling pumps. (Cost estimate is $1,380,000.)
- Secondary Treatment—Construction of two aeration basins within the existing Lagoon No. 1 and installation of aeration equipment. (Cost estimate is $1,490,000.)
- Advanced Treatment—Upgrade and expansion of the dissolved air flotation and gravity sand filters. (Cost estimate is $2,140,000.)
- Effluent Disinfection—Upgrade of chemical equipment and addition of a chlorine scrubber at the existing plant site. (Cost estimate is $380,000.)
- Other—Miscellaneous improvements to the office/laboratory. (Cost estimate is $50,000.)
- Solids Management—Construction of diking in the existing Lagoon No. 1 for the Phase 2 facultative sludge lagoon to be completed as part of the Phase 2 treatment plant improvements. (Cost estimate is $350,000.)
- Total—The estimated total project cost is $6,420,000.

A summary of the Phase 2 treatment plant improvements is as follows:

- Secondary Treatment—Add secondary clarification and return activated sludge (RAS) and waste activated sludge (WAS) facilities. Add additional aeration to the aeration basins. (Cost estimate is $2,860,000.)
- Solids Management—Install a liner and associated piping in the facultative sludge lagoon cell. (Cost estimate is $420,000.)
- Total—The estimated total project cost is $3,280,000.

The following issues must be addressed in addition to the design of the recommended treatment plant upgrade:

- The Biosolids Management Plan will have to be updated for the recommended solids management facilities.

TABLE 7-8.
WASTEWATER FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Design Data		
Design Year	2009	2019
Design Population	9,950	13,370
Design Flows		
Average Dry Weather Flow (ADWF)	1.6 mgd	2.1 mgd
Maximum Month Dry Weather Flow (MMDWF)	2.9 mgd	4.0 mgd
Average Wet Weather Flow (AWWF)	2.6 mgd	3.2 mgd
Maximum Month Wet Weather Flow (MMWWF)	4.4 mgd	5.4 mgd
Peak Day Average Flow (PDAF)	8.8 mgd	10.9 mgd
Peak Instantaneous Flow (PIF)	14.9 mgd	18.5 mgd
Design Loads		
Average Biochemical Oxygen Demand (BOD)	2,300 ppd	3,090 ppd
Maximum BOD	5,160 ppd	6,940 ppd
Average Total Suspended Solids (TSS)	1,610 ppd	2,170 ppd
Maximum TSS	3,570 ppd	4,790 ppd
Headworks, Preliminary Treatment		
Screening		
Type of Screens	Fine, Rotary	
Number of Screens	2	
Peak Flow Capacity, each	9.3 mgd	
Bypass Screen	Manually-cleaned Bar Screen	
Screenings Washing	Yes	
Screenings Compaction	Yes	
Septage	Excluded	
Headworks, Influent Flow Measurement		
Number of Flumes	1	
Flume Size		
Throat Width	24 inches	
Peak Flow Capacity	21.4 mgd	
Minimum Flow Capability	0.27 mgd	

TABLE 7-8 (continued).
WASTEWATER FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Transfer Pump Station		
Type of Pumps	Vertical Turbine Solids Handling	
Number of Pumps		
Active	1	1
Standby		1
Pump Size		
Flow Capacity, each pump		3,800 gpm
Total Dynamic Head with two running		45 feet
Horsepower, assuming 50% efficiency		100 hp
Pump Control		VFD
Wet Well Operating Volume		10,000 gallons
Discharge Forcemain		
Number of Mains		2
Size		18-inch
Length		1,750 feet
Volume, Total		46,000 gallons
Detention Time, at one-quarter of 1999 ADWF		2.8 hours
Secondary Treatment		
Type of Treatment	Aerated Lagoon	Extended Aeration
Number of Basins		2
Basin Sizing		
Water Depth		12 feet
Basin Area at Half Water Depth, each		22,300 square feet
Basin Volume, each		2.0 million gallons
Hydraulic Retention Time, at Design MMDWF		24 hours
Hydraulic Retention Time, at Design PDAF		9 hours
Aeration		
Type of Aeration		Aspirating Aerators
Horsepower, total	120 hp	180 hp
Secondary Clarification		
Number of Clarifiers	—	2
Side Water Depth	—	15 feet
Diameter, each	—	85 feet
Surface Overflow Rate, at Design PDAF	—	960 gpd/ sf
RAS/WAS Pumping		
Number of RAS Pumps, Total	—	4
Number of RAS Pumps, Standby	—	2
RAS Pump Capacity, each	—	2,400 gpm
Number of WAS Pumps, Total	—	2
WAS Pump Capacity, each	—	50 gpm

TABLE 7-8 (continued).
WASTEWATER FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Secondary Treatment, cont.		
Use of Remaining Portions of Existing Lagoons	Facultative Lagoon and Holding	Holding
Lagoon No. 1, Remaining Portion		
Operating Depth, Maximum	12 feet	
Surface Area, at Half of Maximum Operating Depth	261,000 sf	
Volume	23 million gallons, or 72 acre-feet	
Lagoon No. 2		
Operating Depth, Maximum	12 feet	
Surface Area, at Half of Maximum Operating Depth	623,000 sf	
Volume	56 million gallons, or 172 acre-feet	
DAF Treatment		
Number of DAF Units		
Existing Units	1	
Additional Units	1	
DAF Tank Size		
Diameter	31 feet	
Surface Area	750 square feet	
Surface Loading Rate	2.0 gpm/sf	
Flow Capacity	2.2 mgd	
Filters		
Type of Filters	Gravity Sand	
Number of Filters		
Existing Units	2	
Additional Units	2	
Filter Size		
Diameter	20 feet	
Surface Area	310 square feet	
Surface Loading Rate	5.0 gpm/sf	
Flow Capacity	2.2 mgd	

TABLE 7-8 (continued).
WASTEWATER FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Effluent Disinfection		
Type of Effluent Disinfection		Gaseous Chlorination
Chlorine Contact, Bear Creek Discharge		
Type		Existing Chlorine Contact Tank
Volume		67,500 gallons
Contact Time, at allowable flow at 5 mg/l, 1.58 mgd		62 minutes
Contact Time, at allowable flow at 2 mg/l, 3.95 mgd		25 minutes
Chlorine Contact, Molalla River Discharge *		
Type		Transmission Line to River
Diameter		24-inch
Length		16,600 feet
Volume		390,000 gallons
Contact Time, at Design AWWF minus 0.8 mgd		234 minutes
Contact Time, at Design PDAF minus 0.8 mgd		56 minutes
Dechlorination		
Type		Liquid Sodium Bisulfite Feed
Feed Rate		1 mg/l per mg/l of Chlorine Residual
Location, Bear Creek		Downstream of Chlorine Contact Tank
Location, Molalla River *		Remote Station downstream of length of Chlorine Contact Facilities
Reaeration on Molalla River Discharge *		
Location		Downstream of Dechlorination
Type		Gravity
Solids Management		
Type of Treatment	—	Facultative Sludge Lagoon
Design Criteria		
Surface Loading, Maximum	—	20 VSS ppd per 1,000 sf
Yield from Secondary Treatment Facilities	—	0.85 lbs VSS per lb influent BOD
VSS Destruction in FSL	—	38%

TABLE 7-8 (continued).
WASTEWATER FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Lagoon Sizing		
Total Depth	—	12 feet
Solids Depth	—	8 feet
Water Cap Depth	—	4 feet
Area at Top of Solids Depth	—	3.0 ac
Solids Volume	—	6.0 million gallons
SRT, at 2019 Average Load	—	8.3 years

* See Chapter 8 for further description of these facilities.

Estimated annual operation, maintenance, and replacement costs for the recommended treatment plant upgrade are shown in Table 7-9 for startup and Phase 2 (20-year) conditions. The annual costs are shown in 2000 dollars. Detailed cost estimates are presented in Appendix C.

TABLE 7-9.
ESTIMATED ANNUAL OPERATION, MAINTENANCE, AND REPLACEMENT COSTS FOR THE WASTEWATER TREATMENT RECOMMENDATIONS

	Estimated Annual Cost	
Treatment Facility Component	Startup	Phase 2
Preliminary Treatment	$13,000	$17,000
Transfer Pumping	$25,000	$40,000
Secondary Treatment	$252,000	$312,000
Effluent Disinfection	$14,000	$20,000
Solids Management	$65,000	$98,000
Effluent Disposal (see Chapter 8)	$107,000	$143,000
Total Annual Cost	**$476,000**	**$630,000**

CHAPTER 8.
EFFLUENT DISPOSAL

In addition to the wastewater treatment plant improvement recommendations discussed in Chapter 7, the City must address the capacity deficiencies of the existing effluent disposal facilities.

Detailed cost estimates are included in Appendix C. All cost estimates are in 2000 dollars and are total project costs, which include construction costs, contingencies and allied costs (engineering, legal, and administrative costs).

8.1 EFFLUENT DISPOSAL ALTERNATIVES

Current effluent disposal methods include water reuse by irrigation during the summer and surface water discharge to Bear Creek during the winter. A water balance evaluation on existing conditions indicates that existing flows exceed the existing capacity of the effluent disposal facilities. (The water balance spreadsheets are included in Appendix E.) To meet existing condition requirements and to accommodate future flows, both the summer and winter effluent disposal facilities must be expanded.

Alternatives for expanding the effluent disposal facilities are as follows:

- Summer reuse
- Summer groundwater infiltration
- Winter surface water discharge – Bear Creek
- Winter surface water discharge – Molalla River
- Summer surface water discharge – Molalla River
- Winter holding and reuse during summer of winter flows

8.1.1 Summer Reuse

The Oregon Department of Environmental Quality (DEQ) encourages reuse of effluent by land application in the summer. The City currently irrigates effluent on the treatment plant site and on an adjacent field owned by a farmer. With agricultural property surrounding Molalla, the City is well situated for additional effluent reuse by land application.

Knowing the need for additional irrigation area, during 1999 the City approached landowners of agricultural property throughout the vicinity of the City of Molalla. Agricultural land in the area ranges from golf courses and nurseries to feed corn and pastureland. Since the City needs a significant amount of irrigation area, as shown in Table 8-2, the City's goal was to not only identify landowners interested in effluent for irrigation, but to make agreements with only a few landowners with large parcels of property to minimize the facilities requirements and land lost to buffering. Much of the land in proximity to the treatment plant is made up of relatively smaller parcels of less than 100

acres. An exception to this is a large ranch east of the City that includes a total of approximately 700 acres of pastureland.

In addition to the large land available, there are two other benefits to irrigating effluent on this large ranch. Currently, the owner of the ranch, Mr. Steve Coleman, irrigates his pastureland using Molalla River water as part of the water district that owns the water rights on the canal east of the river. Irrigating effluent will offset this use of the river, which runs low during the summer and is water quality limited as discussed in Chapter 4. Also, the north end of the ranch is located on the Molalla River where it is recommended that the City pursue an effluent discharge permit, as discussed below. New facilities can be used for the dual purpose of summer effluent reuse and winter effluent disposal and thereby save on facilities costs.

In the fall of 1999, the City established an agreement with Mr. Coleman to provide effluent for irrigation on the southern 220 acres of his property. This agreement includes a 25-year lease of this land and an option by both parties to extend the lease to other areas of his ranch. The pastureland that will be irrigated on the Coleman ranch requires, on average, a net irrigation of 16 inches per season.

Summary of Summer Reuse

In summary, it is recommended that the City continue efforts to provide effluent to the Coleman Ranch for summer reuse. The recommended facilities are discussed in more detail below. The recommended liquid stream treatment facilities will allow the production of Level IV reclaimed water. The pastureland will only require Level II or Level III treatment; the lower quality Level II treatment would require wider buffers and therefore higher land areas. It is recommended that the City provide a minimum of Level III treatment to ensure low bacteria levels in the effluent and to use more of the land for irrigation with the lesser buffer requirements of the higher quality effluent.

DEQ requires that a Reuse Plan be developed before effluent is irrigated. This plan must include a sample Site Authorization Agreement between the City and the property owners, as well as guidelines that the property owner must follow regarding the application of the reclaimed water to be included in the agreement.

It is also recommended that the City continue summer effluent reuse at the plant site and on the adjacent field.

8.1.2 Summer Groundwater Infiltration

Groundwater infiltration was considered for summer effluent disposal. DEQ has indicated that there are no such facilities in Oregon west of the Cascades. KCM's experience with a community east of the Cascades indicates that pursuing this type of effluent disposal requires a great deal of time and cost to study the applicable site.

Even if the City could identify an appropriate site and receive approval from DEQ, the costs for this alternative are high. The treatment facilities must include an additional unit process for denitrification of the effluent to ensure that total nitrogen and especially

nitrates in the effluent are low. The recommended liquid stream facilities improvements are not well-suited to the addition of denitrification.

The disposal facilities would likely require significant area. The City of Vernonia has an exfiltration pond (effluent is allowed to seep through the bottom of the pond) with an approximate exfiltration rate on the order of 0.5 inches per day. The Soil Conservation Service (SCS) soil surveys indicate similar underlying soil types in Vernonia and Molalla. Applying the exfiltration rate experienced in Vernonia to the Molalla flows results in approximately 136 acres and 172 acres of lagoons for Phase 1 and Phase 2 design conditions, respectively. The costs associated with these area requirements and the diking that would be needed are prohibitively high. Also, the land in the vicinity of the City is prime agricultural land and its use as infiltration basins areas would preclude its intended agricultural use.

As a result, groundwater infiltration is not considered a cost-effective alternative for summer effluent disposal and was not considered further.

8.1.3 Winter Surface Water Discharge – Bear Creek

The current method of effluent disposal in the winter is surface water discharge to Bear Creek. Bear Creek is a tributary of the Pudding River and is a relatively small creek in the vicinity of the existing treatment plant discharge. Due to future increases in wastewater flows, plant effluent will soon exceed the capacity of this small creek.

Two requirements on the City's current discharge permit severely limit the volume of discharge to Bear Creek. One is a dilution requirement that the effluent BOD concentration divided by the ratio of stream flow to effluent flow not exceed 1 mg/l. When stream flows are low during the early winter months on this seasonal creek, allowable discharges are minimal based on this requirement. The other requirement is the monthly average mass load limit of 66 ppd of BOD. Even when stream flows are higher in the winter, allowable discharges are limited as a result of this mass load limitation. Due to the small capacity of Bear Creek and the water quality limited status of the Pudding River, neither of these requirements are likely to be relaxed in future permits. The Department of Environmental Quality (DEQ) has indicated to the City that increased discharges to Bear Creek are not likely to be allowed, but that it is reasonable to assume that discharges meeting the current permit can continue.

In order to achieve the current mass load limit at the Phase 2 MMWWF of 5.4 mgd, the plant would have to produce effluent with BOD concentrations of 1.5 mg/l or less. This assumes that the discharge is not limited by the dilution requirement. Obtaining this high quality of effluent on a continuous basis is not possible and, as such, the City must develop another means of winter effluent disposal.

Summary of Winter Surface Water Discharge – Bear Creek

It is recommended that the City continue to discharge to Bear Creek to the maximum extent allowed by the existing permit limits. This existing outfall is adequately sized since discharge flows to Bear Creek will not increase. The existing Bear Creek flow monitoring station should be upgraded.

8.1.4 Winter Surface Water Discharge – Molalla River

As discussed above, the City must develop another means of winter effluent disposal to meet the requirements of the plant. There are no other economically feasible options for winter effluent disposal except for surface water discharge to the Molalla River.

The Molalla River is the closest water body of adequate size to accommodate a new surface water discharge. The river is approximately 5 miles (along existing roadways) east of the treatment plant. The river is also approximately 5 miles (along existing roadways) north of the treatment plant. The easterly route has the significant advantage of City-owned right-of-way for the majority of the route, while the northerly route follows primarily along state highway. The easterly route would be the more cost-effective alternative; therefore, all further analysis assumes the easterly route. Additionally, as discussed above, the Coleman Ranch on which future summer effluent reuse will occur is along the easterly route alignment, thereby allowing for dual purpose facilities.

Flow Analysis for Winter Surface Water Discharge – Molalla River

The first step in evaluating the Molalla River for a new surface water discharge was to determine available flows. There is a U.S. Geological Survey (USGS) gauging station at Goods Bridge at River Mile 6.0. This gauging station is downstream of the potential outfall at Molalla, which is near River Mile 20. The drainage area upstream of Molalla is approximately 64 percent of the total drainage area at the USGS gauging station.

The design flow used to represent the worst case for surface water discharge is typically the 7-day duration, 10-year frequency (7Q10) low flow. According to the USGS statistical summary of records from 1930 to 1978, the 7Q10 at the USGS gauging station is 43 cubic feet per second (cfs). The 7Q10 low flow invariably occurs during late summer, but approximations can be made for every month of the year. According to the USGS statistical summary, the 7Q10 low flow is approximately equal to the 95 percent exceedence flow for the month of August in the Molalla River at the gauging station. For this study, therefore, the USGS statistical summary of the 95 percent exceedence flow for each month was assumed to represent the 7Q10 low flow for each month.

Table 8-1 shows the USGS 95 percent exceedence flows at the gauging station, the estimated portion of these flows at the potential outfall at Molalla, and the resulting allowable discharge to the Molalla River assuming an effluent biochemical oxygen demand (BOD) concentration of 10 mg/l and dilution requirements similar to those on Bear Creek. Flows are shown in cubic feet per second (cfs) and million gallons per day (mgd).

Based on this analysis, there is adequate capacity in the Molalla River from December through April, when allowable discharge exceeds the projected peak daily average flow (PDAF) through 2019 (10.9 mgd). In November, the allowable 7Q10 discharge is only 3.3 mgd, but it is unlikely that flows to the treatment plant would exceed the average wet weather flow (AWWF) during 7Q10 river flow conditions, and the projected 2019 AWWF is 3.2 mgd. Hence, capacity is adequate for discharge to the river in November as well. Any November plant flows that do exceed the discharge capacity could be stored in the plant lagoons for the short time until river flows increase.

TABLE 8-1.
MOLALLA RIVER DILUTION ANALYSIS

Month	95% Exceedence Flow [a] (cfs)	95% Exceedence Upstream of Molalla [b] (cfs)	(mgd)	Allowable Discharge at 10 mg/l BOD [c] (mgd)
October	60	38	25	2.5
November	80	51	33	3.3
December	274	175	113	11.3
January	443	277	179	17.9
February	441	282	182	18.2
March	542	347	224	22.4
April	503	322	208	20.8
May	326	209	135	13.5
June	171	109	70	7.0
July	79	51	33	3.3
August	46	29	19	1.9
September	50	32	21	2.1

a. Statistical information from USGS gauging station at Goods Bridge, River Mile 6.0.
b. Flow upstream of Molalla on 64 percent of the drainage area to Goods Bridge.
c. Allowable plant discharge based on BOD concentration in effluent shown and dilution ratio requirement of 1.

Additionally, the allowable discharge in May and June exceeds the projected maximum month dry weather flow (MMDWF) through 2019 (4.0 mgd). It may therefore be possible for the City to extend its discharge into the early summer months. This would likely only occur during wet seasons when land application would not be possible. (See the discussion on summer surface water discharge below.)

Water Quality Analysis for Winter Surface Water Discharge – Molalla River

The water quality parameters associated with an effluent discharge to the Molalla River were evaluated in detail. The resulting analysis is included in Appendix G. This analysis indicates that the winter surface water discharge will not significantly impact the water quality of the Molalla River. As discussed in Appendix G, however, effluent discharge would be limited in November during periods of extremely low river flow.

Summary of Winter Surface Water Discharge – Molalla River

It is recommended that the City pursue a surface water discharge to the Molalla River. New facilities are required for piping and pumping the effluent the approximately five miles from the treatment plant to the Molalla River. These facility requirements are

discussed in more detail below. In addition to new facilities, the City must negotiate with DEQ for a permit to allow discharge to the Molalla River.

8.1.5 Summer Surface Water Discharge

The alternative of summer surface water discharge was considered for summer effluent disposal. Currently, the City cannot discharge to Bear Creek during the summer because of extreme low stream flows in the creek and it is not likely that the City will ever be allowed summer discharge to Bear Creek, primarily because the Pudding River is classified as being water quality limited.

The recommendation to provide winter surface water discharge to the Molalla River, however, provides the potential for using these facilities during the summer. As discussed in Chapter 4, the Molalla River is listed by DEQ as water quality limited in the summer for temperature. For a summer surface water discharge to the Molalla River to be approved, evaluation of the impact of the discharge on stream temperature would be carefully scrutinized. Table 8-1 shows, however, that the 7Q10 low flows in May and June are higher than that for November. The water quality analysis in Appendix G indicates that the impacts of effluent discharge on the river during a June 7Q10 low flow condition would meet regulatory requirements.

For this study, it was assumed that the City would not be able to obtain a permit that includes surface water discharge throughout the entire summer. As indicated in the water quality analysis in Appendix G, however, discharge in May and June would not cause unacceptable impacts on the quality of the river. The City should request a permit that would allow discharge in May and June when the weather is wet and does not allow for irrigation.

8.1.6 Winter Holding and Reuse during the Summer of Winter Flows

Another alternative was evaluated for disposal of winter flows, holding these flows through the winter and irrigating during the summer. This reuse would be in addition to the summer reuse discussed above that addresses the reuse of summer flows only.

A water balance evaluation was performed for this alternative assuming continued winter discharge to Bear Creek, no Molalla River discharge, and irrigation on pastureland. The water balance indicates that this alternative would require 1,200 of irrigation area and almost 200 acres of storage lagoons in addition to the storage lagoons and irrigation area required when the Molalla River discharge is assumed, as shown in Table 8-2. The costs associated with this alternative would be prohibitive and it was therefore not considered further.

8.2 EFFLUENT DISPOSAL RECOMMENDATIONS

The recommendations for effluent disposal include continued Bear Creek discharge, winter effluent discharge to the Molalla River, summer effluent reuse on local pastureland, and continued summer effluent reuse at the plant and adjacent site. The overall effluent disposal recommendations are represented in the water balance evaluation for planning conditions included in Appendix E. This water balance evaluation is summarized in Table

8-2. Projected flows are shown as average dry weather flows (ADWF) and average wet weather flows (AWWF).

TABLE 8-2.
WATER BALANCE SUMMARY

Year	Flows (mgd)		Average Discharge		Facility Requirements	
	ADWF	AWWF	Bear (mgd) (1)	Molalla (mgd) (2)	Irrigation (3)	Lagoon (4)
2000	1.07	1.93	0.79	1.44	85 ac (ex) + 230 ac	28 ac (ex)
2004	1.25	2.23	0.79	1.86	55 ac (ex) + 290 ac	22 ac (ex) + 2 ac
2009	1.58	2.61	0.79	2.35	55 ac (ex) + 380 ac	22 ac (ex) + 8 ac
2019	2.12	3.23	0.79	3.10	55 ac (ex) + 550 ac	22 ac (ex) + 16 ac

1) Bear Creek discharge assuming 10 mg/l BOD in effluent and the current 66 pounds per day (ppd) mass load permit limit.
2) Molalla River discharge assuming all flows not discharged to Bear Creek, but not in excess of available advanced treatment capacity.
3) Irrigation areas do not include buffer areas. Currently irrigated areas indicated by (ex).
4) Lagoon areas represent total area at average water depth. Existing lagoon area indicated by (ex).

Bear Creek Facilities Upgrade

It is recommended that winter surface water discharge to Bear Creek continue to the currently permitted limits. Assuming effluent with 10 milligrams per liter (mg/l) BOD, the average discharge to Bear Creek will be 0.79 mgd as shown in Table 8-2. If effluent quality is higher (lower BOD), the discharge to the creek can be increased accordingly. The existing Bear Creek flow monitoring station should be upgraded since the existing stage gaging station is not currently working.

Effluent Disposal to Molalla River

It is recommended that the City pursue winter surface water discharge to the Molalla River. New facilities would include an effluent pump station, approximately five miles of transmission piping, and an outfall to the river. These facilities are shown on Figure 8-1. In addition to these facilities, the City must negotiate with DEQ for a permit for a surface water discharge to the Molalla River.

Table 8-2 shows the average discharge to the Molalla River assuming 0.79 mgd average discharge to Bear Creek. Prior to the advanced treatment upgrades recommended in Chapter 7, the maximum total winter discharge equals the existing capacity of the advanced treatment facilities of 2.23 mgd to ensure that all flows have less than 10 mg/l BOD.

The water balance evaluation presented in Appendix E represents a year of average conditions, average influent flows, average precipitation and evaporation, and average

irrigation requirements. During a generally drier year, influent flows and precipitation will be lower and evaporation and irrigation requirements will be higher. Under these conditions, the storage lagoons may need to be drawn down below the 3-foot depth assumed in the water balance evaluation. During a generally wetter year, influent flows and precipitation will be higher and evaporation and irrigation requirements will be lower. Under these conditions, the City will have to discharge into the early summer months (May and June). It is therefore important that the City obtain a permit that will allow discharge during May and June to accommodate a wet weather year, otherwise, facilities will have to be oversized and will cost more than those presented in this plan.

Summer Reuse

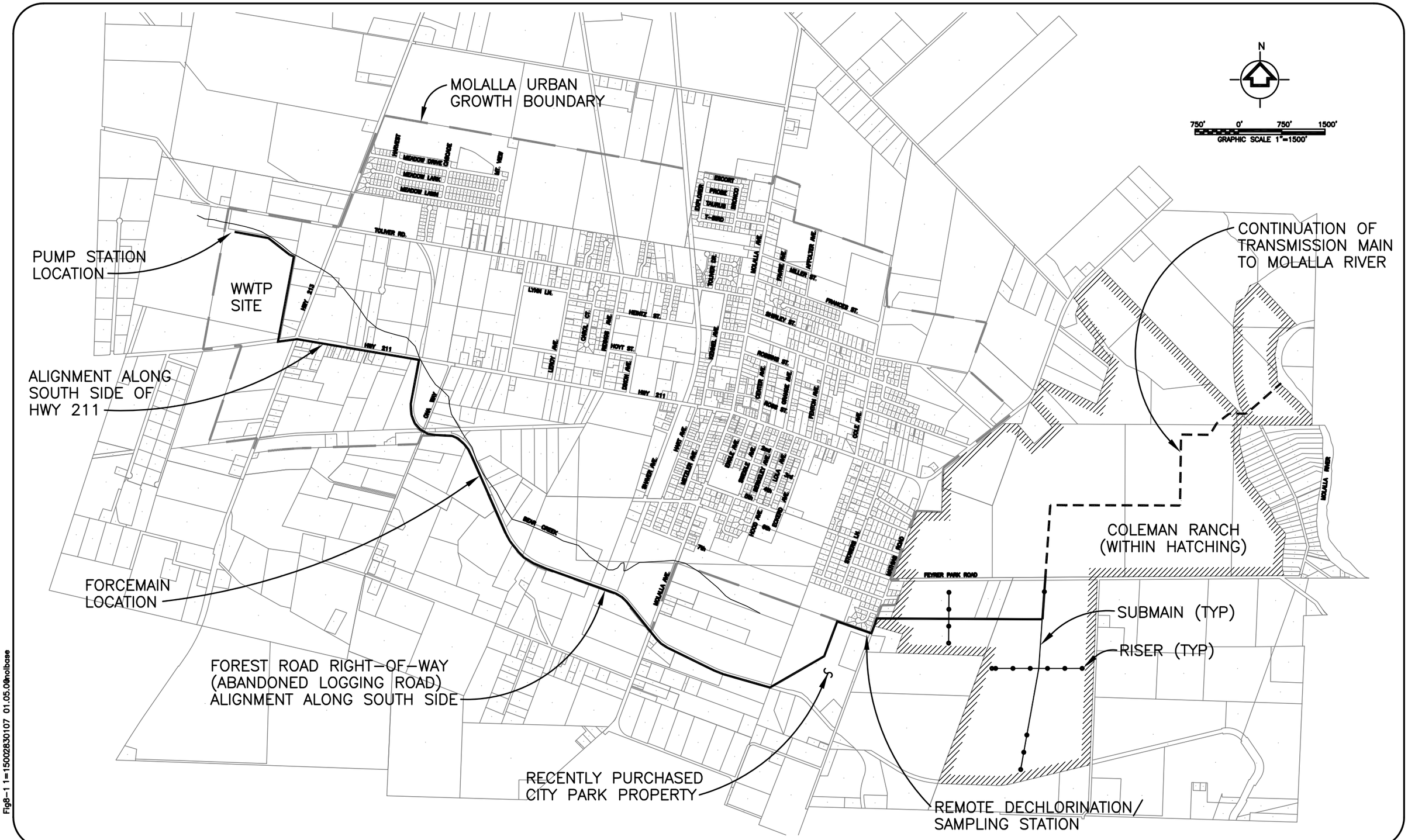
The City must expand the summer reuse facilities and has already entered into an agreement with a landowner of pastureland located to the east of the City. The proposed irrigation area is shown on Figure 8-1. Initially, only the area south of Feyrer Park Road will be irrigated, but there is also the potential for irrigating most of the approximately 700 acres of the Coleman Ranch.

Facilities Recommendations

The facilities required to implement the disposal recommendations include an effluent/irrigation pump station located at the treatment plant, an effluent/irrigation forcemain from the pump station to irrigation at the Coleman ranch, an extension of the effluent/irrigation forcemain to a Molalla River outfall, and storage lagoons located in the vicinity of the existing storage lagoons. The design of the effluent/irrigation pump station and forcemain was developed concurrently with this plan and is detailed in the *City of Molalla Preliminary Design Report; Effluent/Irrigation Pump Station, Forcemain, and Outfall, TetraTech/KCM, October 1999*. Sizing for the storage lagoons is presented in the water balance evaluation included in Appendix E. Planning-level design criteria for the recommendations are also included in Table 8-3. The flow schematic and hydraulic profile for the overall recommended wastewater treatment and disposal facilities are shown in Figures 7-2 and 7-3, respectively.

A summary of the recommended effluent disposal facilities, with estimated total project costs unless otherwise noted are as follows:

- Effluent/Irrigation Pump Station—Installation of effluent/irrigation pump station and flow control facilities at the wastewater treatment plant. This project also includes a standby generator sized to run the plant during a power outage. (Construction bid plus an allowance for change orders and design/administration fees are approximately $1410,000.)
- Effluent/Irrigation Forcemain—Installation of effluent/irrigation forcemain to Feyrer Park Road. (Construction bid plus an allowance for change orders and design/administration fees are approximately $1,900,000.)
- Effluent Outfall—Installation of effluent/irrigation forcemain extension from Feyrer Park Road to outfall to the Molalla River. (Cost estimate is $1,270,000.)



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KCM

City of Molalla, Oregon

WASTEWATER FACILITIES PLAN

FIGURE 8-1

EFFLUENT/IRRIGATION PUMP STATION AND FORCEMAIN LOCATION MAP

- Phase 1 Effluent Storage Facilities—Purchase of property adjacent to plant, installation of half of the required effluent storage volume, and installation of additional irrigation facilities. (Cost estimate is $3,030,000.)
- Phase 2 Facilities—Installation of the second half of the required effluent storage volume, installation of additional irrigation facilities, and addition of effluent/irrigation pump. (Cost estimate is $2,630,000.)
- Total—The estimated total project cost is $10,240,000.

The following issues must be addressed in addition to the design of the recommended effluent disposal facilities:

- An Effluent Reuse Plan will be required prior to irrigation at the Coleman Ranch.
- The City must apply for an effluent discharge permit to the Molalla River from the DEQ.

TABLE 8-3.
EFFLUENT DISPOSAL FACILITIES PLANNING LEVEL DESIGN CRITERIA

Design Parameter	Phase 1	Phase 2
Effluent/Irrigation Pump Station		
Pumps		
Number of Active Pumps	1	1
Number of Standby Pumps		1
Capacity, each pump		3,500 gpm
Total Dynamic Head		277 feet
Horsepower		300 hp
Operation		VFD
Wet Well		
Type		12-ft dia. manhole
Operating Volume, Pump No. 1 On to Pump Off		3,800 gallons
Level Control		Pressure Transducer
Standby Power		
Type		Diesel Engine
Capacity		750 kW
Transfer Switch		Automatic
Fuel Tank Capacity		500 gallons
Effluent/Irrigation Forcemain		
Forcemain		
Diameter, Inner		24-inch
Length (to Feyrer Park Road)		20,400 feet
Detention Time		
Design Average Summer		8.7 hours
Design Average Winter		4.9 hours
Effluent/Irrigation Forcemain Extension, Outfall		
Forcemain		
Diameter, Inner		24-inch
Length (from Feyrer Park Road to Molalla River)		7,000 feet
Outfall (see Appendix G)		
Number of Ports		3
Diameter of Ports		8-inch
Effluent Storage Lagoons		
Dimensions		
Water Depth	12 feet	12 feet
Dike Height	15 feet	15 feet
Average Area	7.5 acres	7.5 acres
Side Slopes	3:1 (horiz.: vert.)	3:1 (horiz.: vert.)
Volume	90 acre-feet	90 acre-feet
Liner	HDPE	HDPE

CHAPTER 9.
IMMEDIATE RECOMMENDATIONS

The evaluations and recommendations presented in Chapters 6, 7, and 8 for the collection system, the treatment plant, and the effluent disposal facilities, respectively, focus on the requirements for the 20-year planning period. These recommendations generally provide for facilities with over two times the capacity of the existing facilities and cannot be implemented in the very short term due to significant funding issues. This chapter presents recommendations for immediate improvements prior to the implementation of the recommendations from Chapters 6, 7, and 8.

The existing facilities are essentially at capacity now and, based on recent growth and the short-term projected increases in flows and loads, are expected to have inadequate capacity within the next few years. This was demonstrated by the Notice of Noncompliance issued to the City by the DEQ for exceeding permit limits during the first three months of 1999. The immediate recommendations presented in this chapter provide a basis for short-term planning prior to implementation of the long-term recommendations. All cost estimates are in 2000 dollars and are total project costs, which include construction costs, contingencies and allied costs.

9.1 COLLECTION SYSTEM

The collection system trunk lines do not have adequate capacity to convey the existing peak hour flow (PHF) without surcharging. It is anticipated that the collection system will have adequate capacity during the interim by surcharging during peak flow events. As a result, no immediate improvements are recommended for the collection system.

9.2 WASTEWATER TREATMENT PLANT

Several phasing alternatives are possible for the wastewater treatment improvements, but none would be cost-effective with respect to long-term requirements. It is recommended that the City pursue funding for the recommended facilities as soon as possible and address short-term capacity deficiencies with limited immediate improvements.

The City can implement several measures immediately to improve the level of treatment and extend the capacity of the plant:

- Remove the sludge accumulation in Lagoon No. 1.
- Add aeration at the inlet end of Lagoon No. 1.
- Improve the piping out of Lagoon No. 2.
- Add dechlorination facilities.

9.2.1 Remove Sludge in Lagoon No. 1

The City dredged and disposed of the sludge accumulation at the inlet of Lagoon No. 1 in the fall of 1999. The dredging process took place while the lagoon was still in service. Most

of the sludge was accumulated in the bottom 6 feet of the lagoon from the west edge of the lagoon to about 400 feet to the east. The remaining east end of the lagoon has only 1 foot of solids accumulation and was not be dredged. The dredging operations removed approximately 700 dry tons of solids, which were applied to the field immediately west of the plant site (where the City currently irrigates effluent during the summer).

9.2.2 Add Aeration in Lagoon No. 1

Since the sludge accumulations in Lagoon No. 1 was removed, the City can add aeration at the inlet end of the lagoon. This will effectively increase aeration basin volume without incurring the cost of a new basin. Lagoon No. 1 was originally constructed and equipped with three 10-horsepower aerators. These aerators were removed because they weren't necessary at the time.

The primary purposes for adding aeration is to provide oxygen to reduce the loading on the remaining facultative portions of the lagoons, thus preventing odors caused by overloading the lagoons and preventing future mounding of sludge at the lagoon inlet. This project would reuse as much of the existing facilities as possible (mooring posts, electrical conduits, etc.) from the original aerators in Lagoon No. 1. The total project cost estimate for this work is $50,000. This estimate is based on providing three 10-horsepower aspirating aerators, conduits, conductors, and motor starters.

The sludge dredging was performed in the fall of 1999 and the aerators can be installed any time after that. It is recommended that the City add the aerators during the fall of 2000.

9.2.3 Improve Piping out of Lagoon No. 2

As discussed in Chapter 5, the normal mode of operation is to withdraw effluent from Lagoon No. 2 at a point about 5 feet above the bottom of the lagoon. Effluent may be withdrawn from either of two pipes that extend horizontally from the tower through the sloped bank of the lagoon. The lower pipe is at the lagoon bottom elevation. The valving in the outlet structure is awkward and seldom used since the rate of withdrawal can also be controlled by manually adjusting the valving at the filter building. The operators would prefer that the structure be modified to allow them to draw from multiple levels in Lagoon No. 2. This improvement is also one that can be used with the long-term facilities recommendations when there will be continued use of Lagoon No. 2 for effluent holding.

The structure could be modified to include a flexible pipe outlet to allow for varying the level of withdrawal. A winch mounted to the top of the structure would then be used to control the level of the pipe. The piping should be sized to minimize the approach velocities into the piping. The purpose of the structure is to vary the level of the pipe based on the level in the lagoon to minimize the amount of algae in the effluent. A reduction in algae levels will lessen the load on the DAF unit and will likely increase its capacity.

At the design condition the lagoons will be used for effluent holding and not necessarily for treatment, so water level variations are likely to be even wider than current variations. The structure must take future operating conditions into account. The total project cost estimate for this work is $60,000.

Lagoon No. 2 must be drained for the structure to be modified. The best time to drain the lagoon is in early spring while the plant can still discharge to the river. It is recommended that the improvements to the structure be constructed in April 2001.

9.2.4 Add Dechlorination Facilities

The City currently disinfects using chlorine during the summer, but does not have to disinfect to meet permitted bacteria limits for winter discharge. Dechlorination has therefore not been needed. There are times during the year, however, when the City could be irrigating and discharging at the same time (early spring and late fall). During this time, the effluent would be chlorinated for reuse, but would need to be dechlorinated prior to discharge.

Dechlorination is most often achieved by chemical addition. The City has chemical feed pumps and mixing tanks from when additional chemicals were added as part of the operation of the DAF unit. This equipment can be used for liquid sodium bisulfite feed for dechlorination. A new chlorine residual analyzer and control equipment is required to appropriately feed the dechlorination chemical. The City has had the feasibility of this approach investigated by a local supplier of this type of equipment.

This project would use as much as possible of the existing facilities (chemical pumps, mixing tanks, etc.) from when the DAF unit was originally constructed. The City has already received from a local supplier a cost estimate of $14,000 for the modification of existing facilities and addition of the remaining facilities needed. It is not expected that significant engineering and administration costs would need to be added to the estimate, though an allowance for construction contingencies should be added. The resulting budget amount for this work is $20,000.

Dechlorination facilities should be implemented prior to the coming winter to allow for maximum discharge to Bear Creek.

9.3 SUMMARY OF IMMEDIATE RECOMMENDATIONS

Table 9-1 summarizes the immediate recommendations, listing them in the order in which they should be implemented. These improvements should extend the capacity of the plant until construction of the improvements recommendations in Chapter 7, assuming that the improvement recommendations are implemented as shown in the capital improvement program (CIP) schedule in Chapter 10.

TABLE 9-1.
COST ESTIMATES FOR IMMEDIATE RECOMMENDATIONS

Immediate Measure	Estimated Cost	Schedule		
		Design	Bid	Construct
Remove Sludge in Lagoon No. 1	already finished			Fall 1999
Add Dechlorination Facilities	$20,000	May 2000	July 2000	Sept. 2000
Add Aeration in Lagoon No. 1	$50,000	May 2000	July 2000	Sept. 2000
Improve Piping out of Lagoon No. 2	$60,000	Oct. 2000	Jan. 2001	April 2001
	$120,000			

CHAPTER 10.
CAPITAL IMPROVEMENT PROGRAM

This capital improvement program (CIP) lists all recommended improvements for the Molalla wastewater system and provides a plan for implementing them over the next 20 years. The projects correct system deficiencies identified in Chapter 5, accommodate expected growth over the planning period as described in Chapter 3, and provide for regulatory compliance as discussed in Chapter 4. The recommendations are based on the evaluations included in Chapters 6, 7, 8, and 9. The CIP is separated into two general categories, projects for the collection system and projects for the wastewater treatment plant. The projects for the wastewater treatment plant include both treatment and effluent disposal projects.

For each CIP project, an estimated budget has been prepared. Costs include a 20 percent construction contingency and 25 percent for engineering, legal, and administrative expenses. (An exception to this are Project T1, T2, and T9, which have already been predesigned.) These budget level estimates are reliable within a margin of plus or minus 20 percent. Detailed cost spreadsheets are included in Appendix C. All costs are in 2000 dollars.

10.1 COLLECTION SYSTEM IMPROVEMENTS

The following collection system improvements are recommended, based on the analysis in Chapter 6.

C1—Treatment Plant Trunk Upgrade. This project includes upgrading the existing Treatment Plant trunk from the confluence of the Toliver Road trunk and the Bear Creek trunk to the treatment plant with 30-inch piping. The existing trunk from Highway 213 and Toliver Road to the confluence of the Toliver Road trunk and the Bear Creek trunk would also be upgraded to 24-inch piping. These improvements should be completed as needed based on future flow requirements. The estimated cost is $600,000.

C2—Toliver Road Trunk Upgrade. This project includes upgrading the existing Toliver Road trunk from the intersection of Molalla Avenue and Heintz Street to the end of Project C1 with 24-inch piping. Alternative alignments for these improvement should be investigated as discussed in Chapter 6. These improvements should be completed soon after Project C1, the Treatment Plant trunk upgrade, as needed. The estimated cost is $2,200,000.

C3—Molalla / Highway 211 Improvements. This project includes adding 12-inch piping to convey the flows from the manhole at the intersection of Molalla Avenue and Highway 211 to the north along Molalla Avenue. The existing piping downstream of the manhole at Molalla and Highway 211 (to the west) would be plugged. These improvements should be completed soon after Project C2, the Toliver Road trunk upgrade, as needed. The estimated cost is $50,000.

C4—Bear Creek Trunk Upgrade. This project includes upgrading the existing Bear Creek trunk from Highway 211 to the Treatment Plant trunk with 21-inch piping. These

improvements should be completed soon after Project C1, the Treatment Plant trunk upgrade, as needed. The estimated cost is $450,000.

C5—Industrial (South) Trunk Extension. This project includes constructing a 12-inch trunk to connect to the upstream end of the Bear Creek trunk. These improvements should be completed as soon as they are needed to enable development of this area. The estimated cost is $860,000.

C6—Highway 213 (South) Trunk Extension. This project includes constructing a 12-inch trunk to connect to the Bear Creek trunk at Highway 213. These improvements should be completed as soon as they are needed to enable development of this area. The estimated cost is $310,000.

10.2 WASTEWATER TREATMENT PLANT IMPROVEMENTS

The following treatment plant improvements projects include construction projects as well as regulatory requirements. Development of the cost estimates can be found in Chapters 7, 8, and 9.

Some of the immediate improvements may decrease the cost of long-term improvements. For example, the aerators added to Lagoon No. 1 in 2000 as Project T6 may be reused in the aeration basins of the Project T13 Phase 1 Secondary Treatment Upgrades recommended for construction around 2005. The cost estimates of the projects, however, were not adjusted for these minor potential reductions.

T1—Effluent/Irrigation Pump Station. This project (discussed in Chapter 8) includes the pump station at the wastewater treatment plant that will pump effluent to the irrigation site in the summer and the Molalla River in the winter on the east side of the City. This project also includes flow control facilities at the plant. The design of this pump station is complete and construction should be complete by the summer of 2000. The estimated cost is $1,410,000.

T2—Effluent/Irrigation Forcemain. This project (discussed in Chapter 8) includes the forcemain from the pump station of Project T1, Effluent/Irrigation Pump Station, east to Feyrer Park Road. The design of this forcemain is complete and construction should be complete by the summer of 2000. The estimated cost is $1,900,000.

T3—Effluent Reuse Plan. The City must prepare an Effluent Reuse Plan prior to applying effluent on the application site (as discussed in Chapter 8). This plan will be developed as part of the design and administration provided on Projects T1 and T2, Effluent/Irrigation Pump Station and Forcemain. The estimated cost is $20,000.

T4—Effluent Discharge Permit to Molalla River. The City must apply for an effluent discharge permit to the Molalla River from the Oregon Department of Environmental Quality (DEQ). The application requires a water quality analysis to determine the impact of the proposed discharge on the river. An environmental assessment (EA) or an environmental impact station (EIS) may also be required given enough controversy over the discharge. The CIP assumes that an EA or EIS will not be required. This application will be developed as part of the design and administration provided on Projects T1 and T2, Effluent/Irrigation Pump Station and Forcemain. The estimated cost is $80,000.

T5—Add Dechlorination Facilities. This project (discussed in Chapter 9) includes the addition of dechlorination facilities at the existing plant for chlorinated flows that are to be

discharged to Bear Creek. The project should be completed in the fall of 2000. The estimated cost is $20,000.

T6—Add Aeration in Lagoon No. 1. This project (discussed in Chapter 9) includes adding aeration in the west end of Lagoon No. 1. This project should be completed in the fall of 2000. The estimated cost is $50,000.

T7—Improve Piping out of Lagoon No. 2. This project (discussed in Chapter 9) includes modifications to the existing outlet structure in Lagoon No. 2. This project should be completed in the spring of 2001. The estimated cost is $60,000.

T8—Phase 1 Preliminary Treatment Upgrades. This project (discussed in Chapter 7) includes installation of facilities for fine screening with washing and compaction and flow measurement to replace the existing headworks. This project should be completed in the fall of 2001. The estimated cost is $630,000.

T9—Phase 1 Transfer Pumping Upgrades. This project (discussed in Chapter 7) includes installation of a transfer pump station with vertical turbine solids handling pumps and forcemain to Lagoon No. 1 to replace the existing pump station and forcemain. This project should be completed in the fall of 2001. The estimated cost is $1,380,000.

T10—Phase 1 Advanced Treatment Upgrades. This project (discussed in Chapter 7) includes upgrade of the advanced treatment facilities to double the existing capacity. This project should be completed in 2002. The estimated cost is $2,140,000.

T11—Effluent/Irrigation Forcemain Extension and Outfall. This project (discussed in Chapter 8) includes the extension of the forcemain in Project T2 through the Coleman Ranch to an outfall to the Molalla River. This project also includes additional irrigation equipment and a dechlorination station on City property west of Mathias Road. This project has been predesigned with Projects T1 and T2, Effluent/Irrigation Pump Station and Forcemain, but final design and construction will not be completed until the City has secured a permit to discharge to the Molalla River, which is targeted for the winter of 2002 when the City's current permit expires. The estimated cost is $1,270,000.

T12—Phase 1 Effluent Storage Facilities. This project (discussed in Chapter 8) includes the purchase of adjacent land and installation of half of the required storage lagoons for the 20-year planning period. This project should be completed as needed. The estimated cost is $3,030,000.

T13—Phase 1 Secondary Treatment Upgrades. This project (discussed in Chapter 7) includes construction of two aeration basins within the existing Lagoon No. 1 and installation of aeration equipment and piping. This project should be completed as needed. The estimated cost is $1,490,000.

T14—Phase 1 Solids Management Upgrades. This project (discussed in Chapter 7) includes construction of diking in the existing Lagoon No. 1 for the Phase 2 facultative sludge lagoon to be completed as part of Project T18. This project should be completed in conjunction with Project T13, Phase 1 Secondary Treatment Upgrades. The estimated cost is $350,000.

T15—Phase 1 Effluent Disinfection Upgrades. This project (discussed in Chapter 7) includes addition of a chlorine scrubber at the existing plant site and upgrades of the dechlorination facilities at the plant. This project should be completed as needed. The estimated cost is $380,000.

T16—Phase 1 Miscellaneous Improvements. This project (discussed in Chapter 7) includes renovation of the existing office/laboratory. This project should be completed as needed. The estimated cost is $50,000.

T17—Phase 2 Effluent Storage and Irrigation Facilities. This project (discussed in Chapter 8) includes the installation of the second half of the required storage lagoons (the first half is included in Project T12, Phase 1 Effluent Storage Facilities) and additional irrigation facilities at the Coleman Ranch. This project should be completed after 2010 as necessary to meet capacity requirements. The estimated cost is $2,630,000.

T18—Phase 2 Treatment Upgrades. This project (discussed in Chapter 7) includes facilities for secondary clarification, return activated sludge (RAS) and waste activated sludge (WAS) pumping, and additional aeration in the aeration basins. It also includes the installation of a liner and associated piping in the facultative sludge lagoon cell. This project should be completed after 2010 as necessary to meet capacity requirements. The estimated cost is $3,280,000.

T19—Biosolids Management Plan. The City needs to update the Biosolids Management Plan (as discussed in Chapter 7) based on the Phase 2 solids management facilities. This update should be completed in conjunction with Project T18, Phase 2 Treatment Upgrades. The estimated cost is $20,000.

10.3 CAPITAL IMPROVEMENT PLAN

The CIP consolidates the improvement recommendations into an overall plan. Table 10-1 summarizes the CIP, with recommended year of implementation and estimated budgets for each project.

Table 10-2 shows estimated annual costs for the proposed I/I program and treatment plant upgrade. These costs are shown in 2000 dollars. Annual costs for the collection system operation and maintenance are not expected to change significantly from their current levels and are therefore not included in Table 10-2.

TABLE 10-2.
ANNUAL COST ESTIMATES

	Annual Cost Estimate	
Project	Startup Condition	Phase 2 Condition
Collection System		
I/I Reduction Program	$40,000	$40,000
Wastewater Treatment Plant		
Plant Operation and Maintenance	$476,000	$630,000

TABLE 10-1.
CAPITAL IMPROVEMENT PROGRAM SUMMARY

Project	Chapter Discussed	Year (Range)	Project Budget
Collection System			
C1. Treatment Plant Trunk Upgrade	6	2002-2003	$600,000
C2. Toliver Road Trunk Upgrade	6	2003-2006	$2,200,000
C3. Molalla/Hwy 211 Improvements	6	2003-2006	$50,000
C4. Bear Creek Trunk Upgrade	6	2004-2008	$450,000
C5. Industrial (South) Trunk Extension	6	2005-2009	$860,000
C6. Highway 213 (South) Trunk Extension	6	2005-2009	$310,000
Wastewater Treatment Plant			
T1. Effluent/Irrigation Pump Station	8	2000	$1,410,000
T2. Effluent/Irrigation Forcemain	8	2000	$1,900,000
T3. Effluent Reuse Plan	8	2000	$20,000
T4. Effluent Discharge Permit to Molalla River	8	2000	$80,000
T5. Add Dechlorination Facilities	9	2000	$20,000
T6. Add Aeration in Lagoon No. 1	9	2000	$50,000
T7. Improve Piping out of Lagoon No. 2	9	2001	$60,000
T8. Phase 1 Preliminary Treatment Upgrades	7	2001	$630,000
T9. Phase 1 Transfer Pumping Upgrades	7	2001	$1,380,000
T10. Phase 1 Advanced Treatment Upgrades	7	2002	$2,140,000
T11. Effluent/Irrigation Forcemain Extension and Outfall	8	2003	$1,270,000
T12. Phase 1 Effluent Storage Facilities	8	2004-2006	$3,030,000
T13. Phase 1 Secondary Treatment Upgrades	7	2005-2008	$1,490,000
T14. Phase 1 Solids Management Upgrades	7	2005-2008	$350,000
T15. Phase 1 Effluent Disinfection Upgrades	7	2005-2008	$380,000
T16. Phase 1 Miscellaneous Upgrades	7	2005-2008	$50,000
T17. Phase 2 Effluent Storage and Irrigation Facilities	8	2010-2014	$2,630,000
T18. Phase 2 Treatment Upgrades	7	2010-2014	$3,280,000
T19. Biosolids Management Plan	7	2010-2014	$20,000

CHAPTER 11.
FUNDING PLAN

Potential funding opportunities for the City's wastewater system improvements include grants, bonds, and loans. Eligibility requirements, program availability, and terms are subject to change, so some programs described in this chapter may not be available in the future for Molalla's wastewater CIP. A recommendation for an interim system development charge (SDC) was made and is described in this chapter. The City will also prepare a cost-of-service and SDC analysis as a document separate from this report, presenting a methodology for equitably distributing costs between existing and future commercial, industrial and residential users.

11.1 FEDERAL PROGRAMS

11.1.1 Rural Utilities Service Water and Wastewater Loans and Grants

The U.S. Department of Agriculture's Rural Utilities Service (RUS) program provides funding for rural areas and towns with populations of up to 10,000. Assistance includes loans and grants. Funds may be used for installation, repair, improvement, or expansion of rural wastewater collection and treatment facilities. The costs of land acquisition and legal and engineering fees are eligible for funding if they are necessary to develop the facility.

Eligibility Requirements

Water and wastewater loans and grants are available to public entities including municipalities, counties, special purpose districts, Indian tribes and non-profit corporations. Applicants must be unable to obtain the required funds via commercial sources under reasonable terms. Entities must have the legal capacity to borrow and repay the loans, must pledge security for the loans, and must be able to efficiently maintain and operate the proposed facilities. The facilities to be funded must be consistent with development plans of the state, multi-jurisdictional area, county, or municipalities where the projects are to be constructed. The facilities must also comply with all relevant local, state, and federal laws including zoning, pollution control, and health and sanitation standards. The City of Molalla is eligible for funding from this program.

Terms

Borrowers of RUS loans must be able to demonstrate the following:

- They have monthly user rates higher than the "state wide average" of approximately $35 per month.
- They have legal authority to borrow and repay loans, to pledge security for loans, and to operate and maintain the facilities and services.
- They are financially sound and able to manage the facility effectively.
- They have a financially sound facility based on taxes, assessments, revenues, fees, or other satisfactory sources of income to pay for all facility

costs, including operations and maintenance, and to retire indebtedness and maintain a reserve.

The maximum loan term is 40 years but the term may not exceed statutory limitations on the agency borrowing the money or the expected useful life of the improvements. The reserve can typically be funded at 10 percent per year over a 10-year period. Loan interest rates and maximum grant amounts are based on median household income as shown in Table 11-1. The median household income for Molalla, currently based on 1990 census data, is $20,938. This funding source will use the 1990 data until the 2000 census data is available (likely in 2002). The City qualifies for the lowest loan rate and maximum grant based on the 1990 data, so it would be in the City's best interest to apply for this funding before the 2000 census data becomes available. Program staff have verbally indicated that the largest single grant that is likely under RUS is $750,000.

TABLE 11-1.
RUS GRANT FUNDS AND LOAN INTEREST RATES

Median Household Income	Maximum Grant (portion of total project cost)	Loan Interest Rate as of January 1, 1998
Less than 22,205	75 percent	4.5 percent
$22,205 to $27,756	45 percent	4.875 percent
Greater than $27,756	0 percent	5.255 percent

Availability of Funding

For fiscal year 1999, the funds available for Oregon communities total $10 million in loans and $5 million in grants.

Contact

More information on the RUS wastewater loan and grant program is available at the following:

Oregon Rural Development
101 SW Main, Suite 1410
Portland, OR 97204-3222
Contact: David Force
Phone: 503-414-3327

11.1.2 Department of Housing and Urban Development Community Development Block Grants

The Department of Housing and Urban Development provides grants under the Community Development Block Grant (CDBG) program to facilitate economic development by revitalizing neighborhoods with improved community facilities and services. In Oregon, this program is administered by the Oregon Economic Development Department (OEDD). In Clackamas County, the Clackamas County Community Development Department

(CCCDD) administers the program. Maximum CDBG funding is $750,000 per project. The County awards funding on a three-year cycle, and will be accepting applications for the next cycle in November 1999. Funds would be available in July 2000. The County contact for additional information is Doug Youngsma, who can be reached at (503) 655-8591.

Eligibility Requirements

The program is available to non-metropolitan cities and counties. Funding may be used for the construction, expansion, or rehabilitation of public water and sewer systems to meet federal and state mandates. To be eligible, the applicant must be out of compliance with federal or state rules, regulations or permits. Molalla received a notice of non-compliance (NON) from DEQ on April 28, 1999. It is expected that the existing treatment plant will be out of compliance more frequently as growth continues. The service area for the project must contain at least 51 percent low- and moderate-income residents. CCCDD staff have indicated that they believe the City of Molalla would be eligible for funding from this program.

11.1.3 DEQ Clean Water State Revolving Fund (CWSRF)

While this program is administered on a state level, it is included with federal funding options because the Environmental Protection Agency (EPA) provides seed money in the form of grants to all 50 states. The Oregon CWSRF provides low-cost loans for use in the planning, design, and construction of water pollution control facilities. The program is focused on achieving and maintaining water quality standards within Oregon communities.

Eligibility Requirements

Any public agency in the state of Oregon is eligible for a CWSRF loan for projects that are publicly owned. Loan proceeds may be used for planning, design, and construction of publicly owned wastewater treatment facilities or to build or rehabilitate sewer collection systems. Only projects that address or prevent water quality problems are eligible. Applicants are prioritized by project need during a periodic pre-application process. Projects are ranked according to factors including sensitivity of the receiving water body, population of the associated service area, and existing water quality violations or enforcement activities. The resulting project priority list is included in the program's proposed Intended Use Plan, which is subject to a 30-day public review. Once the list is finalized, applicants are invited to submit a final application providing additional detail.

Upon approval of the final application, projects are funded in sequential order according to the priority list until all funds are depleted. Funding for any single project is capped at 15 percent of the total funding available. As new funds become available they are allocated to remaining projects that are still in need of funding. Funding is based on the following criteria:

- The first 15 percent of funds are awarded to communities with a population of 5,000 or less that have severe water quality problems.
- Facility planning projects are funded in point order, with the next 10 percent of available funds.

- Remaining funds are allocated to remaining projects sequentially, including small community and facility planning projects.

Terms

The following terms apply for CWSRF funding:

- Up to 100 percent of planning, design, and construction costs are eligible for funding though the CWSRF.
- Loans are issued with terms of up to 20 years.
- Over the last five years, interest rates have ranged from 3.43 percent to 4.29 percent, with lower rates available for facility planning.
- Loan fees include a one-time fee of 1.5 percent of the loan amount with an annual servicing fee of 0.5 percent of the outstanding principal balance.
- Borrowers usually pledge sewer revenues to repay the loan. Corresponding rates are typically set to cover 105 percent of repayment requirements.
- A loan reserve of 100 percent of the average annual repayment amount is required.

Other funding options available include pairings of coverage and reserve amounts, or the issuance of general obligation or revenue bonds. When communities are unable to meet the above security requirements, a discretionary loan may be issued with terms tailored to the specific situation.

Contact

Further information on the CWSRF program may be obtained by calling the program coordinator in the local DEQ office, at (503) 229-6412.

11.1.4 Economic Development Administration (EDA) and the Economic Development Act of 1965

The EDA authorizes grants and loans under this program to assist communities in areas certified by the Secretary of Commerce as areas of substantial unemployment. Direct grants of up to 50 percent and supplementary grants of up to 80 percent of costs are authorized for wastewater improvements to alleviate economic hardship. The program is geared to projects stimulating permanent industrial and economic development, and communities qualify for funding of water and wastewater improvements that will help create new industry or maintain or substantially increase levels of employment. Eligibility is heavily weighted in favor of projects that will result in economic development. There is a $1 million maximum allowance per project. Actual funding limits are based on the number of jobs created. We recommend that this program not be pursued unless a large economic development opportunity is identified.

11.2 STATE PROGRAMS

11.2.1 Oregon Economic Development Department Special Public Works Fund (SPWF)

The Oregon State Legislature created the SPWF in 1985. The fund is capitalized through the issuance of state revenue bonds and through state lottery proceeds. The SPWF is intended to promote the creation of jobs for Oregonians. Loans and grants are issued to facilitate the construction of public infrastructure to support industrial/manufacturing development as well as commercial development that is marketed nationally or internationally and attracts business from outside Oregon.

Eligibility Requirements

Eligible municipalities are described in the SPWF Applicant's Handbook and generally include cities, counties, water supply districts, water and wastewater authorities, sanitary districts, port authorities, water control districts, county service districts, and tribal councils of Indian tribes. The handbook is available from the SPWF Project Coordinator (Janet Hillock) for Clackamas County, at 1-800-233-3306.

Eligible SPWF projects include public infrastructure needed to enable the location or expansion of eligible businesses. Specific projects include: wastewater collection and treatment capacity, publicly owned railroad spurs and sidings, purchase of rights of way and easements necessary for infrastructure, airports, port facilities, storm drainage, roadway and bridges, and water source, treatment, storage and distribution. Program funds are not eligible for equipment, wetlands mitigation, general administrative costs, construction of privately owned infrastructure, or the purchase of property not related to infrastructure.

Funding levels are determined by a financial analysis based on demonstrated need. The basis for this analysis includes debt capacity, repayment sources, and applicant's ability to afford loans from additional sources. To be eligible for the program, applicants must document recent interest by eligible businesses looking to locate in the municipality. Moreover, the applicant must demonstrate ongoing marketing efforts relating to economic development of industrial lands. Depending on how the City plans to promote industrial/commercial development, Molalla may be eligible for this program.

Terms

The following terms apply for SPWF funding:

- Maximum loan term is 25 years. A 20-year term is typical.
- Loans are typically repaid with utility revenues, general funds, voter approved bonds, or local improvement district revenue.
- Grant funding is typically unavailable unless the applicant is classified as "severely affected" or a "timber dependent" community. In such a case, up to $250,000 per project may be awarded to communities without a firm commitment for new business demand.

- Grants are available under the following conditions when there is a firm commitment from one or more eligible businesses:
  - Up to $10,000 in grant funds may be awarded for each full-time-equivalent job created, depending on demonstrated financial need.
  - Of the total jobs created, at least 30 percent must be "family wage" jobs.
  - Public and/or private investment must equal at least two times the infrastructure cost.

Financing Limits

The following financing limits apply for projects financed with bond funds:

- For communities with firm business commitments (all applicants)
  - Bond loan: $10,000,000
  - Collateral loan: $1,000,000
  - Grant funding: $500,000
  - Issuance costs and debt service reserve: actual costs
  - Total funding from collateral loans, grants, and issuance and debt service reserve may not exceed $1,500,000.
- For capacity building
  - Bond loan: $10,000,000
  - Collateral loan: $1,000,000
  - Issuance costs and debt service reserve: actual costs
  - Grant funding (severely affected communities only): $250,000
  - Total funding from collateral loans, grants, and issuance and debt service reserve costs may not exceed $1,000,000 ($1,250,000 for severely affected communities).

The following financing limits apply for projects financed with direct SPWF (Lottery) funds

- For communities with firm business commitments (all applicants)
  - Direct loan: $1,000,000
  - Grant funding: $500,000
  - Total funding may not exceed $1,000,000
- Capacity Building – Not eligible

For most projects, loan funding is split 83 percent/17 percent between bond loans and collateral loans (direct SPWF funds).

11.2.2 Oregon Economic Development Department Water/Wastewater Financing Program

The Oregon State Legislature created the water/wastewater financing program in 1993. It is capitalized by the sale of state revenue bonds as well as with a portion of state Lottery proceeds. Its primary purpose is to provide financing for construction of public infrastructure required to ensure compliance with the federal Safe Drinking Water Act or Clean Water Act. Specifically, it is intended to assist local governments facing state and federal mandates relating to public drinking water systems and wastewater systems.

Eligibility Requirements

The program is available to cities, counties, water supply districts, water and wastewater authorities, sanitary districts, port authorities, water control districts, county service districts and tribal councils of Indian tribes. Detailed application requirements are available in the Water/Wastewater Financing Program Applicants Handbook, which is available from Clackamas County coordinator Janet Hillock, at 1-800-233-3306. Funding levels awarded to qualified applicants are determined by a financial analysis based on demonstrated need and an inability to afford additional loans. The following activities are eligible for funding through the program:

- Water source, treatment, storage and distribution
- Wastewater collection and capacity
- Storm system
- Purchase of rights of way and easements necessary for infrastructure
- Design and construction engineering.

Program funds may not be used for privately owned facilities or infrastructure, general administrative costs or the purchase of property not related to infrastructure. Eligibility for program funding is contingent upon having received a Notice of Non-Compliance, from a regulatory agency regarding the Safe Drinking Water Act or the Clean Water Act. The City received a NON from DEQ on April 28, 1999 for violations that occurred in January through March, 1999. The frequency of violations is expected to increase as growth occurs.

To be eligible for grant funding, user rates must be above the statewide average of approximately $35 per month. Grant funding cannot exceed more than 50 percent of the total Water and Wastewater Financing Program funding package (i.e. to be eligible for a $500,000 grant would require that the funding package included a $500,000 or greater loan.)

Terms

The following terms apply:

- The maximum loan term is 25 years; a 20-year term is typical.
- Maximum grant amount is $500,000, including issuance costs and any debt service reserves (if required).

- Maximum direct loan amount is $500,000 when financed with state Lottery funds.
- Borrowers that are deemed "credit worthy" may be funded though the sale of state revenue bonds. Maximum bonded loan amount for this mechanism is $10,000,000.
- Loans are typically repaid with utility revenue, general funds, or voter approved bond issues.

11.3 LOCAL FUNDING ALTERNATIVES

11.3.1 General Obligation (GO) Bonds

Entities with taxing authority under the laws of the State of Oregon have the option of issuing general obligation (GO) bonds. A GO bond is a bond backed by the full credit of the issuer for the payment of which the issuer can levy *ad valorem taxes*. The issuer can make the required payments on the bonds solely from the tax levy or may instead use revenues from assessments, user charges or some other source. Since the bonds are secured by the power to tax, they usually justify a lower interest rate than other types of bonds. Generally, GO bonds lend themselves readily to competitive public sale at a reasonable interest rate because of their high degree of security, their tax exempt status, and their general acceptance.

These bonds can be revenue-supported because a portion of the user fee can be pledged toward payment of the debt service. This can eliminate the need to collect additional property taxes to retire the bonds. Revenue-supported GO bonds have most of the advantages of revenue bonds, but also maintain the low interest rate and the marketability of GO bonds.

Oregon law does not limit the total amount or the percentage of GO bonds that a community can issue. This portion of the property tax is outside the state constitutional restriction limiting property taxes to a fixed percentage of assessed value. State law limits the maximum term of GO bonds to 40 years. The typical term for GO bonds is 20 to 30 years. Under the present economic climate, lower interest rates will be associated with the shorter terms.

Financing of wastewater system improvements by GO bonds is usually accomplished as follows:

1. The capital costs required for the proposed improvement are determined.
2. A general election must be held to authorize the sale of the GO bonds.
3. Following voter approval, the GO bonds are offered for sale to banks and other investors.
4. The revenue from the bond sale is used to pay the capital costs associated with the project.
5. GO bond authorizations must be approved by a majority vote, and this generally limits proposals to projects benefiting all or the majority of a community. Some of the advantages of GO bonds over other types of bonds are as follows:

- The laws authorizing GO bonds are less restrictive than those governing improvement bonds under the Bancroft Act. Interest rates are not affected by the Bancroft limitations and costly assessment procedures are not required.
- Taxes paid in the retirement of GO bonds are Internal Revenue Service deductible.
- GO bonds can be sold prior to construction, providing funds before expenses must be paid.

The use of an *ad valorem tax* is a common method of repaying GO bonds for utility improvements. This method of financing results in the participation of all private property owners within the benefited area, whether the property is developed or undeveloped. The construction costs for the project are shared proportionally among all property owners based on the assessed value of each property.

11.3.2 Revenue Bonds

A revenue bond is a bond that is payable solely from charges made for the services provided. Such bonds cannot be paid from tax levies or special assessments, and their only security is the borrower's promise to operate the system in a way that will provide sufficient net revenue to meet the obligations of the bond issue. Revenue bonds are most commonly retired with revenue from user fees.

Successful issuance of revenue bonds depends on bond market evaluation of the dependability of the revenue pledged. Normally, there are no legal limitations on the amount of revenue bonds to be issued, but excessive amounts are generally unattractive to bond buyers because they represent high investment risk. In rating revenue bonds, buyers consider the economic justification for the project, the reputation of the borrower, methods for billing and collecting, rate structures, and the degree to which forecasts of net revenues are realistic.

11.3.3 Improvement Bonds

Improvement bonds can be issued under an Oregon law called the Bancroft Act. Cities and special districts are limited to improvement bonds not exceeding 3 percent of true cash value. For a specific improvement, all property within the assessment area is assessed on an equal basis, regardless of whether it is developed or undeveloped. This assessment becomes a direct lien against the property, and owners have the option of either paying the assessment in cash or applying for improvement bonds. If the improvement bond option is taken, the district sells Bancroft improvement bonds to finance the construction, and the assessment is paid over 20 years in 40 semi-annual installments with interest.

With improvement bond financing, an improvement district is formed, the boundaries are established, and the benefited properties and property owners are determined. The engineer usually determines an approximate assessment, either on a square-foot basis or a frontage basis. Property owners are then given an opportunity to remonstrate against the project. The assessments against the properties are usually not levied until the actual total cost of the project is determined. Since this determination is normally not possible until the

project is completed, funds are not available from assessments for the purpose of making monthly payments to the contractor. Therefore, some method of interim financing must be arranged, or a pre-assessment program, based on the estimated total costs, must be adopted.

The primary disadvantages to this source of revenue are as follows:

- The property to be assessed must have a true cash value at least equal to 50 percent of the total assessments to be levied.
- For projects that benefit the entire City, GO bonds can be issued in lieu of improvement bonds, and they are usually more favorable.

The construction of water and sewerage facilities through the formation of improvement districts is viable when the properties bordering or served by the improvements are specifically benefited. The establishment of an improvement district should be based on a thorough evaluation of the long-range plan for the entire area. Following is a brief schedule for the development of wastewater improvements by this method.

1. Receive written request or petition from the affected property owners for the improvement. If there is any question regarding the feasibility or approval of the project, the petitioners should provide sufficient funds to cover engineering, legal, and administrative costs associated with preliminary planning and establishing the district.
2. Establish an assessment district and preliminary cost estimates. The cost estimates presented at this time will be the basis for projecting the assessment; however, some revision may be necessary depending on the scope of the project.
3. If the project meets with the approval of the petitioners, authorize the preparation of plans and specifications. Obtain interim financing.
4. Advertise for bids.
5. Award the construction contract.
6. Construct the project.
7. Sell the bonds and repay the interim financing.

11.3.4 Capital Construction (Sinking) Fund

Sinking funds are often established by budget for a particular construction purpose. Budgeted amounts from each annual budget are carried in a sinking fund until sufficient revenues are available for the needed project. Such funds can also be developed with revenue derived from system development charges or serial levies.

11.3.5 System Development Charges

System development charges (SDCs) are fees the City collects from developers when they develop properties that will use the wastewater system or other municipal service. Fees are collected when building permits are issued. SDCs can be used to finance capital

improvements required to provide municipal services to the development. Operation, maintenance, and replacement costs cannot be financed or repaid by SDC revenues.

As established in ORS 223, an SDC has two principal elements: reimbursement and improvement. The reimbursement portion of the SDC is the fee for buying into existing or under-construction capital facilities. The reimbursement fee represents a charge for using excess capacity in an already paid-for facility. The revenue from this fee is typically used to pay back existing loans for improvements. The improvement portion of the SDC is a fee to cover the cost of capital improvements required to provide increased capacity to serve new development. Initially, the City will be able to charge an improvement fee SDC. After the facilities are constructed, the City must convert the SDC to a reimbursement fee SDC.

Interim SDC

Prior to the development of the recommended SDC, to be prepared as part of a separate document, an interim SDC was determined. The methodology used to calculate the recommended interim wastewater SDC was as follows:

- For the capital improvements identified in the capital improvement program (CIP) included in the draft of this plan, it was estimated that a $8,300,000 portion of the immediate and Phase 1 costs could be attributed to growth.
- It was estimated that the additional population that could be supported by these improvements was 7,600. Assuming 2.73 people per dwelling, this translates to 2,784 equivalent dwelling units (EDUs).
- Dividing the cost of improvements required to accommodate growth by the number of EDUs due to growth resulted in an SDC of $2,981 per EDU.

The City implemented this interim SDC in June 1999. It is very likely that the cost-of-service and SDC analysis will show that a higher SDC can be supported, but it was considered prudent to establish a conservative interim SDC until the analysis is completed. If the interim SDC were set higher than supported by the analysis, the City would have to refund the difference to those from whom interim SDCs were collected.

11.4 FUNDING RECOMMENDATIONS

User rates will depend on the amount of grant contribution, the SDC, and the number of EDUs added; these will be addressed in the City's cost-of-service and SDC analysis. Grant funding will require a user rate in the range of approximately $35 per month.

Available grant funding for public works projects has decreased in recent years, so it is imperative that the City aggressively pursue grant funding. The City already has conducted a "one stop meeting" with OEDD and other funding agencies. The City should pursue grant funding from programs to address water quality issues from RUS, OEDD (water and wastewater financing program) and the Clackamas County Community Development Department. Funding should be secured as soon as possible to allow the City to take advantage of funding under its current median household income levels. If economic development opportunities can be identified, the City should pursue funding from EDA and OEDD's Special Public Works Fund program.

CITY OF MOLALLA

WASTEWATER FACILITIES PLAN

Historical Monthly Flow and Rainfall Data

January 1996 through December 1998

Month	Flow (mgd)			Rainfall
	Max	Min	Avg	(in)
Jan-96	2.65	0.90	1.68	9.0
Feb-96	5.00	1.15	2.37	13.7
Mar-96	3.33	0.79	1.30	5.4
Apr-96	1.92	0.83	1.17	6.6
May-96	2.37	0.81	1.33	7.8
Jun-96	0.99	0.48	0.62	1.3
Jul-96	0.81	0.53	0.64	0.6
Aug-96	0.69	0.49	0.56	0.4
Sep-96	1.20	0.51	0.65	3.6
Oct-96	2.23	0.68	1.18	7.3
Nov-96	4.00	0.74	1.70	12.4
Dec-96	4.00	1.21	2.49	18.0
Jan-97	3.73	1.04	1.79	9.6
Feb-97	3.83	0.91	1.46	5.0
Mar-97	4.64	1.39	2.23	10.4
Apr-97	2.84	0.98	1.56	4.6
May-97	1.78	0.41	0.91	2.2
Jun-97	1.36	0.37	0.60	2.6
Jul-97	0.69	0.26	0.41	0.8
Aug-97	0.65	0.45	0.48	1.9
Sep-97	1.22	0.65	0.82	4.1
Oct-97	1.97	0.57	0.94	6.4
Nov-97	1.49	0.67	0.99	4.5
Dec-97	2.03	0.69	1.07	5.2
Jan-98	3.28	1.13	2.00	11.9
Feb-98	1.87	1.03	1.31	6.5
Mar-98	2.61	0.83	1.41	6.9
Apr-98	1.38	0.83	1.08	3.3
May-98	1.67	0.83	1.36	10.5
Jun-98	1.30	0.68	0.89	2.7
Jul-98	0.87	0.46	0.67	0.3
Aug-98	0.69	0.47	0.60	0.0
Sep-98	0.79	0.55	0.64	1.1
Oct-98	1.04	0.57	0.73	5.1
Nov-98	3.63	0.80	1.50	21.5
Dec-98	4.78	0.96	2.00	10.3

DEQ PROCEDURE FOR DETERMINING WASTEWATER FLOWS

The State of Oregon Department of Environmental Quality (DEQ) has developed a procedure for determining various wastewater flows using a statistical analysis. The documentation of this procedure is the DEQ Memorandum *Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon: MMDWF, MMWWF, PDAF, and PIF*, Rev. IV.96. This procedure uses historical wastewater treatment plant (WWTP) records and corresponding rainfall data and historical rainfall statistics to determine wastewater flows for existing conditions.

For the purpose of identifying flows, the following classifications will be used:

- Annual Average Flow (AAF)—Average daily wastewater flow.
- Maximum Month Dry Weather Flow (MMDWF)—Average daily flow for the highest-flow month during the dry weather season. The MMDWF design flows must be based on a 10-year recurrence interval (or 10 percent probability of occurrence during the dry season).
- Maximum Month Wet Weather Flow (MMWWF)—Average daily flow for the highest-flow month during the wet weather season. The MMWWF design flows must be based on a 5-year recurrence interval (or 20 percent probability of occurrence during the wet season).
- Peak Daily Average Flow (PDAF)—Highest average flow for a 24-hour period. The PDAF design flows must be based on a 5-year recurrence interval (or 20 percent probability of occurrence during the wet season).
- Peak Instantaneous Flow (PIF)—Highest instantaneous flow, often defined as the sustained one-hour peak. The PIF design flows must be based on a 5-year recurrence interval (or 20 percent probability of occurrence during the wet season).

Three years of WWTP records from January 1996 through December 1998 are included in this analysis. Data used for this procedure is typically limited to the most recent year of record to avoid growth effects that may skew or mask the flow/rainfall correlation. For Molalla, however, wet weather flows did not necessarily increase with population increases. As a result, flow and rainfall data from 1996 to 1998 were used to provide a more complete set of data from which to create the appropriate plots of this procedure. Each plot was checked to ensure that data from the earlier years of record show up above as well as below the best-fit straight line.

MMDWF AND MMWWF

For the DEQ procedure, MMDWF and MMWWF are determined using a plot of monthly rainfall versus average monthly WWTP flows for the months of January through May. Table 1 shows the historical data used for this plot. Monthly rainfall was measured at the WWTP and reported on the WWTP monitoring reports. WWTP flows are shown in million gallons per day (mgd).

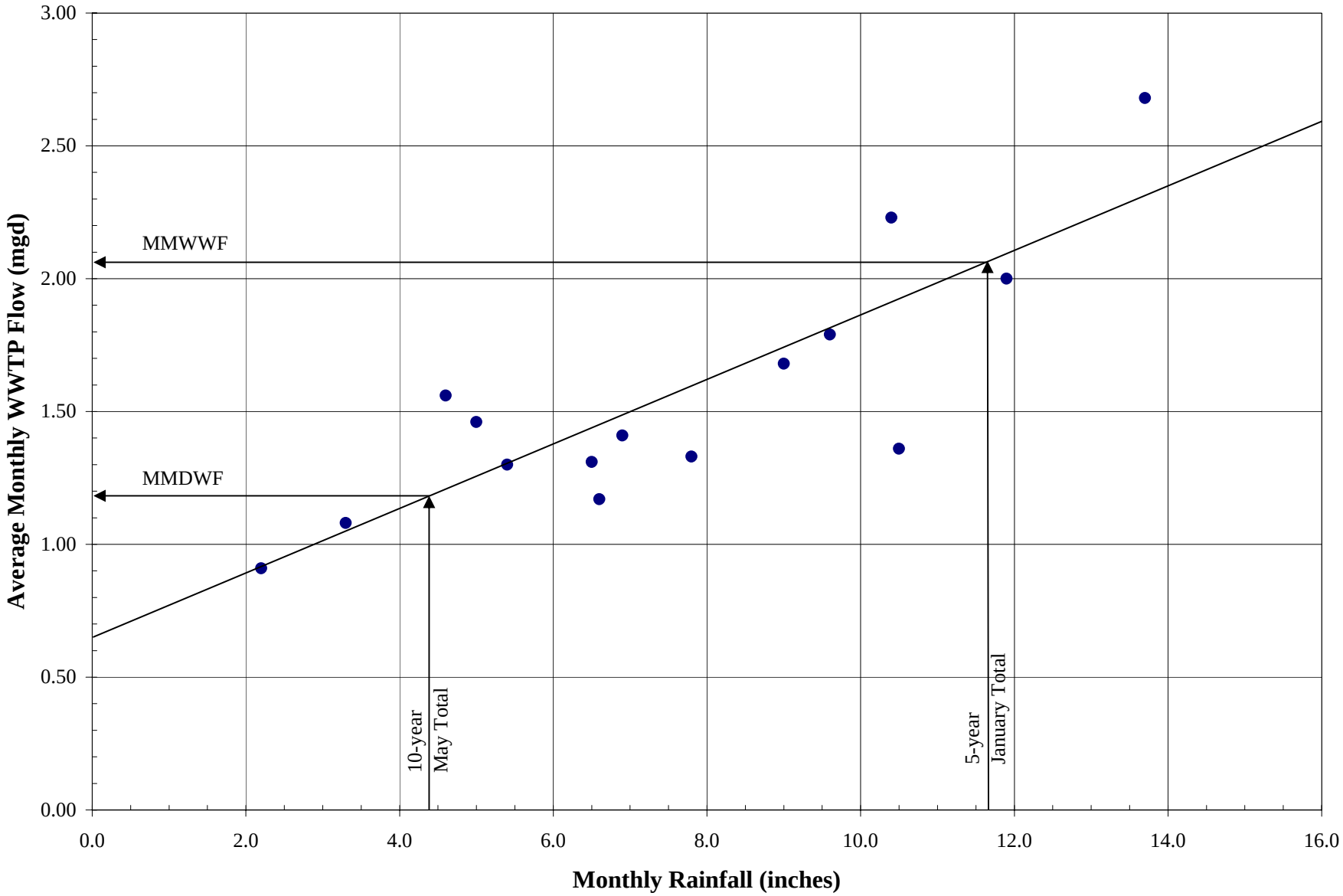
TABLE 1
WET WEATHER MONTHLY WWTP FLOW AND RAINFALL

Year	Month	Monthly Rainfall (inches)	WWTP Average Flow (mgd)
1996	Jan	9.0	1.68
1996	Feb	13.7	2.68
1996	Mar	5.4	1.30
1996	Apr	6.6	1.17
1996	May	7.8	1.33
1997	Jan	9.6	1.79
1997	Feb	5.0	1.46
1997	Mar	10.4	2.23
1997	Apr	4.6	1.56
1997	May	2.2	0.91
1998	Jan	11.9	2.00
1998	Feb	6.5	1.31
1998	Mar	6.9	1.41
1998	Apr	3.3	1.08
1998	May	10.5	1.36

Figure A-1 shows the plotted data from Table 1. Also plotted is a best-fit straight line determined using a least squares method with the Table 1 data. The slope of the line is 0.12 with a y-intercept of 0.65.

Using the straight-line approximation, MMDWF and MMWWF were determined by identifying the estimated WWTP flows corresponding with the 10-year May rainfall and the 5-year January rainfall, respectively. The statistical data for rainfall was found in National Climatic Data Center (NCDC) Climatological Summary No. 20 for the recording station nearest the City of Molalla (Oregon City). Oregon City is approximately 13 miles north of Molalla. There was an NCDC recording station in the City of Molalla from 1948 to 1976. To verify that the use of the Oregon City data is appropriate, the data from Molalla and Oregon City were compared and it was confirmed that these two stations recorded similar total annual rainfall and rainfall distribution throughout each year. The resulting rainfall used from the Oregon City statistical summary is 4.42 inches for the 10-year May rainfall and 11.56 inches for the 5-year January rainfall.

The results of this analysis indicate MMDWF at 1.18 mgd and MMWWF at 2.06 mgd. (See Figure A-1.) Based on the 1998 population in Molalla of 5,395, the resulting unit flows are 219 gallons per capita per day (gpcd) for MMDWF and 382 gpcd for MMWWF.


3.00
2.50
2.00
1.50
1.00
0.50
0.00
Average Monthly WWTP Flow (mgd)
MMWWF
MMDWF
10-year
May Total
5-year
January Total
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
Monthly Rainfall (inches)

PDAF

PDAF determination is similar to that for MMDWF and MMWWF. PDAF uses a plot of daily rainfall versus daily WWTP flows for high rainfall events. Table 2 shows the historical data used for this plot. Daily rainfall was measured at the WWTP and reported on the WWTP monitoring reports.

TABLE 2
WET WEATHER DAILY WWTP FLOW AND RAINFALL

Year	Month	Day	Rainfall (inches)	WWTP Flow (mgd)
1996	Feb	5	2.80	5.00
1996	Feb	17	1.10	2.13
1997	Jan	1	1.50	2.53
1997	Jan	17	1.60	2.30
1997	Jan	30	2.50	2.76
1997	Mar	1	1.30	1.92
1998	Jan	10	1.40	1.77
1998	Jan	12	1.30	2.75
1998	Jan	14	2.00	2.38
1998	Feb	20	1.50	1.57

Figure A-2 shows the plotted data from Table 2. Also plotted is a best-fit straight line determined using a least squares method with the Table 2 data. The slope of the line is 1.42 with a y-intercept of 0.19.

Using the straight-line approximation, PDAF was determined by identifying the estimated WWTP flows corresponding with the 5-year, 24-hour rainfall. The statistical data for the 5-year, 24-hour rainfall was found on the National Oceanic and Atmospheric Administration (NOAA) Atlas 2, Volume X isopluvial maps for the State of Oregon. The 5-year, 24-hour storm is approximately 3.4 inches.

The results of this analysis indicate PDAF at 5.04 mgd. (See Figure A-2.) Based on the 1998 population in Molalla of 5,395, the resulting unit flow is 934 gpcd.

TSS ANALYSIS OF PDAF

As stated in the DEQ Memorandum, upstream sewage overflows often make PDAF records unusable. DEQ has therefore developed a method for determining PDAF using records of influent total suspended solids (TSS). This method uses a plot of influent TSS versus daily WWTP flows for high influent flows.

Overflows are not an issue in Molalla, but influent flows in excess of the capacity of the Parshall flume make it difficult to determine peak flows. As a result, the TSS analysis was used as another way to estimate PDAF.

Data from the months of January and February for 1996 through 1998 were used to create the data points shown in Figure A-3. As shown on Figure A-3, the data points on the right-hand side of the graph are generally clustered at the lower end of the graph (i.e., the lower flows would have consistently higher influent TSS.) The data points then become more scattered toward the upper end of the graph on the left-hand side of the graph (i.e., the higher flows would have lower influent TSS indicating dilution.) The PDAF is then projected based on a curve approximated along the upper edge of the data points.

The results of this analysis indicate PDAF at approximately 4.38 mgd. (See Figure A-3.) Based on the 1998 population in Molalla of 5,395, the resulting unit flow is 819 gpcd. These PDAF results are less than those developed above and, as a result, the higher PDAF results from above are used to be conservative.

PIF

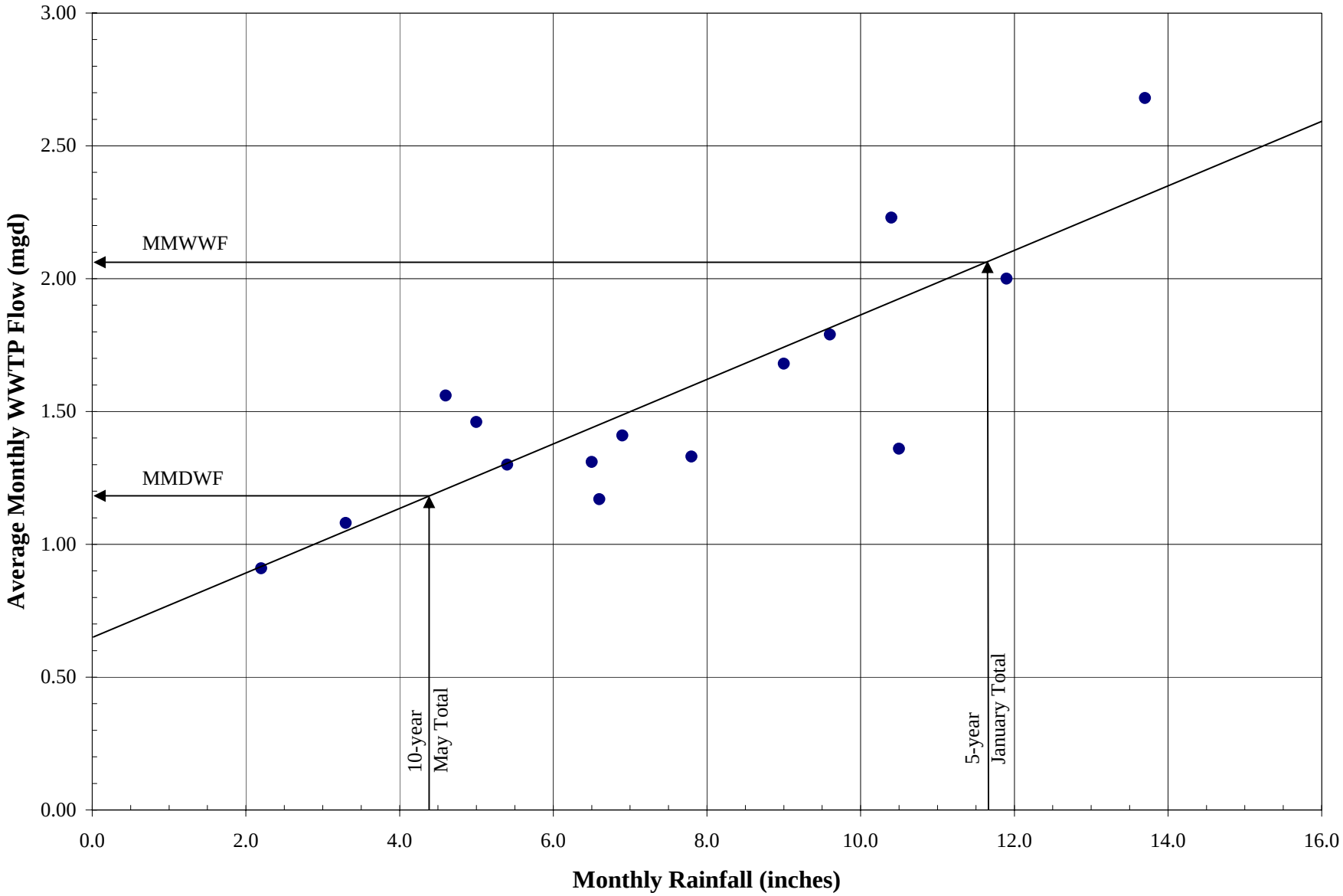
PIF is determined using a log-log plot of probability versus plant flows as shown in Figure A-4. The results of the MMWWF and PDAF determination for the DEQ procedure and the actual plant flows for 1998 are shown on this plot. Also shown are the flow estimates that result from applying the 1998 population to the overall unit flows from the 1996 to 1998 historical records (see Table 3-1). For example, the average of the overall average ADWF and the overall average AWWF is 245 gpcd ([159+331]/2 = 245) multiplied by 5,395 people is 1.32 mgd. Table 3 summarizes the data plotted on Figure A-4.

TABLE 3
HISTORICAL WASTEWATER FLOWS TO THE WWTP

Source	AAF (mgd)	MMWWF (mgd)	PDAF (mgd)
DEQ Procedure		2.06	4.56
1998 Actual Flows	1.18	2.00	4.78
1998 Estimates Using 1996-1998 Flow Data	1.32	2.98	6.00

The line shown on Figure A-4 plot is a straight-line approximation for both the points developed from the DEQ procedure and for the points that represent the 1998 flow estimates using the 1996-1998 flow data. The PIF is estimated from this line on the far right-hand side of the plot.

For the DEQ procedure straight-line approximation, the results indicate a PIF of approximately 6.2 mgd. Based on the 1998 population in Molalla of 5,395, the resulting unit flow is 1,150 gpcd.


3.00
2.50
2.00
1.50
1.00
0.50
0.00
Average Monthly WWTP Flow (mgd)
MMWWF
MMDWF
10-year
May Total
5-year
January Total
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
Monthly Rainfall (inches)

For the 1998 estimates using 1996-1998 flow data, the straight-line approximation indicates a PIF of approximately 10.2 mgd. Based on the 1998 population in Molalla of 5,395, the resulting unit flow is 1,890 gpcd.

CONCLUSIONS

Table 4 shows a comparison between the estimates for the DEQ procedure presented in this appendix and the overall unit flows from the 1996-1998 historical data summarized in Table 3-1. In conclusion, the unit flows developed from the historical data are higher than those determined using the DEQ procedure and will be used for the wastewater flow projections.

TABLE 4
UNIT FLOW COMPARISON

	MMDWF (gpcd)	MMWWF (gpcd)	PDAF (gpcd)	PIF (gpcd)
DEQ Procedure	219	382	934	1,150
1996-1998 Flow Data Summary	295	553	1,110	1,890

CITY OF MOLALLA
WASTEWATER FACILITIES PLAN

PEAK FLOW PROJECTIONS CHECK

Assumptions:			
	New Construction I/I *	2000	gpad
	Average Dwelling Density	4	dwellings per acre
	Average People per Dwelling	2.7	people per dwelling
	New Construction I/I per Capita (New I/I)	185	gpcd
	Existing ADWF per Capita (Ex ADWF)	159	gpcd

Year	Population	Existing Population					Future Population				Overall	
		% Increase Over Exist.	PDAF Unit Flow (1) (gpcd)	PIF Unit Flow (1) (gpcd)	PDAF (mgd)	PIF (mgd)	PDAF Unit Flow (2) (gpcd)	PIF Unit Flow (3) (gpcd)	PDAF (mgd)	PIF (mgd)	PDAF (mgd)	(PIF) (mgd)
EXISTING												
1998	5,395	0.0%	1,110	1,890	5.99	10.20	0	0	0.00	0.00	5.99	10.20
PROJECTIONS												
1999	5,770	0.0%	1,110	1,890	5.99	10.20	503	821	0.19	0.31	6.18	10.50
2004	7,950	2.5%	1,138	1,937	6.14	10.45	503	821	1.29	2.10	7.42	12.55
2009	9,950	5.0%	1,166	1,985	6.29	10.71	503	821	2.29	3.74	8.58	14.45
2014	11,530	7.5%	1,193	2,032	6.44	10.96	503	821	3.09	5.04	9.52	16.00
2019	13,370	10.0%	1,221	2,079	6.59	11.22	503	821	4.01	6.55	10.60	17.77

* A unit I/I flow of 1200 to 1600 gpad is often assumed for reasonably tight construction for new sewers. With the City's existing I/I somewhat higher than average for Western Oregon, we've assumed 2000 gpad.

1) To account for continuing deterioration of the City's existing sewer, a 10% increase in unit flows over the 20-year planning period was assumed.
2) PDAF Unit Flow for Future Population = (New I/I) + 2*(Ex ADWF)
3) PIF Unit Flow for Future Population = (New I/I) + 4*(Ex ADWF)

INVENTORY OF COLLECTION SYSTEM

The following pages present a spreadsheet inventory of the existing sanitary sewer collection system main lines in the City of Molalla. This inventory was developed from available as-built information. The collection system spreadsheet defines each individual pipe section by the upstream manhole. The collection system spreadsheet includes specific data for each pipe section including diameter, age, material, length, slope, invert elevations, full flow gravity capacity, and the as-built drawing set from which the information was referenced.

A collection system base map was also developed and is included in this appendix. On this sanitary sewer collection system map, each manhole is assigned an identification number. The identification numbers link the collection system map to the spreadsheet inventory of the collection system.

CITY OF MOLALLA - WASTEWATER FACILITY PLAN														
SANITARY SEWER SYSTEM INVENTORY														
MH / Junction Information					**Downstream Pipe Information**									**Plan Information**
MH /	Location	Branch	Split	Ground	Dia.	Age of	Conduit	n-Value	Length	Invert	Invert Elev.		Full Flow	As-Builts / Construction Plans
Junction			to:	Elev.		Pipe	Mat'l			Slope	Upstrm	Dnstrm	Capacity	
Label				(ft)	(in)				(ft)	(ft/ft)	(ft)	(ft)	(mgd)	
612	Hwy 211 and Molalla			374.0	12	1995	PVC	0.013	303	0.0105	364.1	360.9	2.36	Highway 211 SS Improvements
608	Hwy 211 and Metzler	610		369.7	12	1995	PVC	0.013	236	0.0081	360.7	358.8	2.08	Highway 211 SS Improvements
606	Hwy 211 and Kennel			367.4	12	1995	PVC	0.013	96	0.0029	358.6	358.3	1.25	Highway 211 SS Improvements
562	Hwy 211 and Hart	564		368.1	12	1995	PVC	0.013	281	0.0021	358.1	357.5	1.07	Highway 211 SS Improvements
550	Hwy 211 and RR	552		371.3	12	1995	PVC	0.013	58	0.0026	357.3	357.2	1.17	Highway 211 SS Improvements
548	Hwy 211 and RR			371.0	12	1995	PVC	0.013	24	0.0046	357.0	356.8	1.56	Highway 211 SS Improvements
546	Hwy 211 and RR			371.0	12	1995	PVC	0.013	414	0.0029	356.6	355.4	1.24	Highway 211 SS Improvements
544	Hwy 211 and Wittenberg			367.7	12	1995	PVC	0.013	460	0.0034	355.2	353.7	1.35	Highway 211 SS Improvements
542	Hwy 211 and Dixon			365.0	12	1995	PVC	0.013	457	0.0079	353.5	349.9	2.05	Highway 211 SS Improvements
540	Hwy 211 and Ridings			361.2	12	1995	PVC	0.013	442	0.0080	349.7	346.1	2.06	Highway 211 SS Improvements
538	Hwy 211			355.3	12	1995	PVC	0.013	434	0.0111	345.9	341.1	2.43	Highway 211 SS Improvements
536	Hwy 211 and Leroy			350.3	12	1995	PVC	0.013	346	0.0081	340.9	338.1	2.08	Highway 211 SS Improvements
534	Hwy 211			346.9	12	1995	PVC	0.013	252	0.0056	338.0	336.6	1.72	Highway 211 SS Improvements
532	Hwy 211			346.6	12	1995	PVC	0.013	218	0.0056	336.4	335.2	1.73	Highway 211 SS Improvements
530	Hwy 211			346.4	12	1995	PVC	0.013	455	0.0095	335.0	330.7	2.25	Highway 211 SS Improvements
528	Hwy 211			344.0	12	1995	PVC	0.013	516	0.0131	330.6	323.8	2.64	Highway 211 SS Improvements
526	Hwy 211 and Ona			336.6	12	1995	PVC	0.013	463	0.0199	323.6	314.4	3.26	Highway 211 SS Improvements
524	Bear Creek and Hwy 211			329.7	15	1988	CONC	0.013	227	0.0059	314.0	312.6	3.23	Bear Creek Interceptor
522	Bear Creek			324.3	15	1988	CONC	0.013	500	0.0108	312.5	307.1	4.35	Bear Creek Interceptor
520	Bear Creek			317.3	15	1988	CONC	0.013	392	0.0087	307.0	303.6	3.90	Bear Creek Interceptor
512	Toliver		22	329.7	15	1991	CONC	0.013	453	0.0097	319.0	314.6	4.12	Toliver / Bear Creek SS Diversion
511	south of Toliver			323.6	15	1991	CONC	0.013	255	0.0066	314.5	312.8	3.41	Toliver / Bear Creek SS Diversion
510	south of Toliver			322.8	15	1991	CONC	0.013	416	0.0050	312.7	310.6	2.96	Toliver / Bear Creek SS Diversion
509	south of Toliver			321.0	15	1991	CONC	0.013	331	0.0050	310.5	308.9	2.96	Toliver / Bear Creek SS Diversion
508	south of Toliver			321.4	15	1991	CONC	0.013	413	0.0050	308.8	306.7	2.96	Toliver / Bear Creek SS Diversion
507	south of Toliver			321.6	15	1991	CONC	0.013	500	0.0050	306.6	304.1	2.96	Toliver / Bear Creek SS Diversion
506	Bear Creek	520, 507		313.5	15	1988	CONC	0.013	553	0.0079	303.5	299.1	3.72	Bear Creek Interceptor
504	Bear Creek			308.0	15	1988	CONC	0.013	339	0.0040	299.1	297.7	2.66	Bear Creek Interceptor
502	Bear Creek			305.4	15	1988	CONC	0.013	388	0.0040	297.6	296.1	2.64	Bear Creek Interceptor
500	Bear Creek and Hwy 213			302.5	15	1988	CONC	0.013	546	0.0065	296.0	292.4	3.37	Bear Creek Interceptor

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

SANITARY SEWER SYSTEM INVENTORY

MH / Junction Information					**Downstream Pipe Information**									**Plan Information**
MH / Junction Label	Location	Branch	Split to:	Ground Elev. (ft)	Dia. (in)	Age of Pipe	Conduit Mat'l	n-Value	Length (ft)	Invert Slope (ft/ft)	Invert Elev. Upstrm (ft)	Invert Elev. Dnstrm (ft)	Full Flow Capacity (mgd)	As-Builts / Construction Plans
331	Molalla and Ross			368.4	12	1954	???	0.013	367	0.0035	359.1	357.8	1.36	Sewage Collection System
330	Molalla			367.1	12	1954	???	0.013	350	0.0035	357.6	356.4	1.37	Sewage Collection System
318	Molalla and Robbins			365.3	12	1954	???	0.013	310	0.0035	356.2	355.1	1.36	Sewage Collection System
224	Grange and Hwy 211	226		384.6	10	1954	???	0.013	354	0.0120	376.2	371.9	1.55	Sewage Collection System
222	Grange			378.7	10	1954	???	0.013	350	0.0120	371.7	367.5	1.55	Sewage Collection System
220	Grange			374.0	10	1954	???	0.013	360	0.0066	367.3	364.9	1.15	Sewage Collection System
218	Grange and Robbins			371.1	10	1954	???	0.013	301	0.0028	364.7	363.9	0.75	Sewage Collection System
176	Heintz and Grange	178		371.9	12	1954	???	0.013	477	0.0085	363.7	359.7	2.13	Sewage Collection System
174	Heintz and Center			365.8	12	1954	???	0.013	353	0.0037	359.5	358.1	1.41	Sewage Collection System
172	Heintz			364.1	12	1954	???	0.013	170	0.0169	357.9	355.1	3.00	Sewage Collection System
154	Shirley and Fenton	158, 156		374.9	8	1954	???	0.013	100	0.0052	367.5	367.0	0.56	Sewage Collection System
138	Shirley and Fenton	140		374.0	8	1954	???	0.013	388	0.0052	367.0	365.0	0.56	Sewage Collection System
136	Shirley and Grange			371.5	8	1954	???	0.013	487	0.0040	364.8	362.8	0.50	Sewage Collection System
134	Shirley and Center			376.0	8	1954	???	0.013	491	0.0040	362.6	360.7	0.49	Sewage Collection System
116	Shirley and Molalla	118		370.9	8	1954	???	0.013	399	0.0055	357.7	355.5	0.58	Sewage Collection System
114	Heintz and Molalla	318, 172, 116		363.9	15	1954	???	0.013	473	0.0063	354.9	351.9	3.33	Sewage Collection System
106	Kennel and Ross			363.3	8	1954	???	0.013	326	0.0040	357.2	355.9	0.49	Sewage Collection System
104	Kennel			362.0	8	1954	???	0.013	80	0.0040	355.7	355.4	0.50	Sewage Collection System
102	Kennel			362.0	8	1954	???	0.013	330	0.0040	355.2	353.8	0.50	Sewage Collection System
100	Kennel			362.4	8	1954	???	0.013	330	0.0040	353.6	352.3	0.50	Sewage Collection System
98	Heintz and Kennel	114, 100		361.4	15	1954	???	0.013	329	0.0015	351.7	351.2	1.61	Sewage Collection System
96	Kennel			359.7	15	1954	???	0.013	329	0.0015	351.0	350.5	1.63	Sewage Collection System
94	Toliver and Kennel			364.8	15	1954	???	0.013	247	0.0033	350.3	349.5	2.40	Sewage Collection System
82	Toliver	84			15	1954	???	0.013	200	0.0033	349.5	348.8	2.40	Sewage Collection System
76	Toliver	78		356.6	15	1954	???	0.013	460	0.0038	348.6	346.9	2.57	Sewage Collection System
74	Toliver and Wittenberg			355.7	15	1954	???	0.013	460	0.0076	346.7	343.2	3.64	Sewage Collection System
72	Toliver			350.4	15	1954	???	0.013	466	0.0076	343.0	339.4	3.65	Sewage Collection System
56	Toliver and Ridings	58		346.2	15	1954	???	0.013	427	0.0090	339.2	335.4	3.97	Sewage Collection System

CITY OF MOLALLA - WASTEWATER FACILITY PLAN														
SANITARY SEWER SYSTEM INVENTORY														
MH / Junction Information					**Downstream Pipe Information**									**Plan Information**
MH /	Location	Branch	Split	Ground	Dia.	Age of	Conduit	n-Value	Length	Invert	Invert Elev.		Full Flow	As-Builts / Construction Plans
Junction			to:	Elev.		Pipe	Mat'l			Slope	Upstrm	Dnstrm	Capacity	
Label				(ft)	(in)				(ft)	(ft/ft)	(ft)	(ft)	(mgd)	
54	Toliver			346.5	15	1954	???	0.013	229	0.0084	335.2	333.3	3.84	Sewage Collection System
50	Toliver	51			15	1954	???	0.013	229	0.0085	333.3	331.3	3.85	Sewage Collection System
28	Toliver and Leroy	30		343.0	15	1954	???	0.013	5	0.0015	331.1	331.1	1.62	Sewage Collection System
27	Toliver and Leroy		518		15	1954	???	0.013	444	0.0015	331.1	330.4	1.62	Sewage Collection System
26	Toliver			341.0	15	1954	???	0.013	486	0.0015	330.2	329.5	1.60	Sewage Collection System
24	Toliver			336.9	15	1954	???	0.013	481	0.0015	329.3	328.6	1.62	Sewage Collection System
23	Toliver			335.2	15	1954	???	0.013	481	0.0015	328.4	327.7	1.63	Sewage Collection System
22	Toliver	518		331.5	12	1977	???	0.013	275	0.0112	326.1	323.0	2.45	STP and Trunk Sewer
20	Toliver			329.5	12	1977	???	0.013	461	0.0110	323.0	318.0	2.42	STP and Trunk Sewer
18	Toliver			324.8	12	1977	???	0.013	462	0.0155	318.0	310.8	2.88	STP and Trunk Sewer
16	Toliver			317.7	12	1977	???	0.013	456	0.0110	308.8	303.8	2.42	STP and Trunk Sewer
14	Toliver			311.5	12	1977	???	0.013	500	0.0110	303.8	298.3	2.42	STP and Trunk Sewer
12	Toliver and Hwy 213			310.0	15	1977	???	0.013	230	0.0030	298.0	297.3	2.29	STP and Trunk Sewer
10	Toliver			311.7	15	1977	???	0.013	456	0.0051	297.3	295.0	2.98	STP and Trunk Sewer
8	Toliver			307.0	15	1977	???	0.013	495	0.0030	293.1	291.6	2.30	STP and Trunk Sewer
6	west of Hwy 211	500, 8		296.8	21	1977	???	0.013	390	0.0030	291.1	290.0	5.62	STP and Trunk Sewer
4	west of Hwy 211			294.0	21	1977	???	0.013	365	0.0030	290.0	288.9	5.61	STP and Trunk Sewer
2	west of Hwy 211			292.5	21	1977	???	0.013	25	0.0032	288.9	288.8	5.81	STP and Trunk Sewer

CITY OF MOLALLA - WASTEWATER FACILITY I										
SANITARY SEWER SYSTEM INVENTORY										
MH / Junction Information				**Capacity Analysis**	**(PDAF Analysis)**			**(PIF Analysis)**		
MH / Junction Label	Location	Branch	Split to:	Design Discharge	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)
612	Hwy 211 and Molalla			10% Ex. + 5% Fut.	0.9	1.46		1.5	0.86	
608	Hwy 211 and Metzler	610		10% Ex. + 5% Fut.	0.9	1.18		1.5	0.58	
606	Hwy 211 and Kennel			10% Ex. + 5% Fut.	0.9	0.35		1.5	***	13
562	Hwy 211 and Hart	564		10% Ex. + 5% Fut.	0.9	0.17		1.5	***	14
550	Hwy 211 and RR	552		10% Ex. + 5% Fut.	0.9	0.27		1.5	***	14
548	Hwy 211 and RR			10% Ex. + 5% Fut.	0.9	0.66		1.5	0.06	
546	Hwy 211 and RR			10% Ex. + 5% Fut.	0.9	0.34		1.5	***	13
544	Hwy 211 and Wittenberg			10% Ex. + 5% Fut.	0.9	0.45		1.5	***	13
542	Hwy 211 and Dixon			15% Ex. + 20% Fut.	1.9	0.15		3.2	***	15
540	Hwy 211 and Ridings			15% Ex. + 20% Fut.	1.9	0.16		3.2	***	15
538	Hwy 211			15% Ex. + 20% Fut.	1.9	0.53		3.2	***	14
536	Hwy 211 and Leroy			15% Ex. + 20% Fut.	1.9	0.18		3.2	***	15
534	Hwy 211			15% Ex. + 20% Fut.	1.9	***	13	3.2	***	16
532	Hwy 211			15% Ex. + 20% Fut.	1.9	***	13	3.2	***	16
530	Hwy 211			15% Ex. + 20% Fut.	1.9	0.35		3.2	***	14
528	Hwy 211			15% Ex. + 20% Fut.	1.9	0.74		3.2	***	13
526	Hwy 211 and Ona			15% Ex. + 20% Fut.	1.9	1.36		3.2	0.06	
524	Bear Creek and Hwy 211			15% Ex. + 35% Fut.	2.6	0.63		4.4	***	17
522	Bear Creek			15% Ex. + 35% Fut.	2.6	1.75		4.4	***	16
520	Bear Creek			15% Ex. + 35% Fut.	2.6	1.30		4.4	***	16
512	Toliver		22	20% Ex. + 10% Fut.	1.7	2.42		2.9	1.22	
511	south of Toliver			20% Ex. + 10% Fut.	1.7	1.71		2.9	0.51	
510	south of Toliver			20% Ex. + 10% Fut.	1.7	1.26		2.9	0.06	
509	south of Toliver			20% Ex. + 10% Fut.	1.7	1.26		2.9	0.06	
508	south of Toliver			20% Ex. + 10% Fut.	1.7	1.26		2.9	0.06	
507	south of Toliver			20% Ex. + 10% Fut.	1.7	1.26		2.9	0.06	
506	Bear Creek	520, 507		35% Ex. + 55% Fut.	4.8	***	17	8.1	***	21
504	Bear Creek			35% Ex. + 55% Fut.	4.8	***	19	8.1	***	23
502	Bear Creek			35% Ex. + 55% Fut.	4.8	***	19	8.1	***	23
500	Bear Creek and Hwy 213			35% Ex. + 55% Fut.	4.8	***	18	8.1	***	21

CITY OF MOLALLA - WASTEWATER FACILITY I										
SANITARY SEWER SYSTEM INVENTORY										
MH / Junction Information				**Capacity Analysis**	**(PDAF Analysis)**			**(PIF Analysis)**		
MH / Junction Label	Location	Branch	Split to:	Design Discharge	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)
331	Molalla and Ross			15% Ex.	0.9	0.46		1.6	***	13
330	Molalla			15% Ex.	0.9	0.47		1.6	***	13
318	Molalla and Robbins			15% Ex.	0.9	0.46		1.6	***	13
224	Grange and Hwy 211	226		20% Ex. + 5% Fut.	1.5	0.05		2.5	***	12
222	Grange			20% Ex. + 5% Fut.	1.5	0.05		2.5	***	12
220	Grange			20% Ex. + 5% Fut.	1.5	***	12	2.5	***	14
218	Grange and Robbins			20% Ex. + 5% Fut.	1.5	***	13	2.5	***	16
176	Heintz and Grange	178		20% Ex. + 5% Fut.	1.5	0.63		2.5	***	13
174	Heintz and Center			20% Ex. + 5% Fut.	1.5	***	13	2.5	***	15
172	Heintz			20% Ex. + 5% Fut.	1.5	1.50		2.5	0.50	
154	Shirley and Fenton	158, 156		10% Ex.	0.6	***	9	1.1	***	11
138	Shirley and Fenton	140		10% Ex.	0.6	***	9	1.1	***	11
136	Shirley and Grange			10% Ex.	0.6	***	9	1.1	***	11
134	Shirley and Center			10% Ex.	0.6	***	9	1.1	***	11
116	Shirley and Molalla	118		10% Ex.	0.6	***	9	1.1	***	11
114	Heintz and Molalla	318, 172, 116		60% Ex. + 20% Fut.	4.7	***	18	7.9	***	21
106	Kennel and Ross					0.49			0.49	
104	Kennel					0.50			0.50	
102	Kennel					0.50			0.50	
100	Kennel					0.50			0.50	
98	Heintz and Kennel	114, 100		60% Ex. + 20% Fut.	4.7	***	23	7.9	***	28
96	Kennel			60% Ex. + 20% Fut.	4.7	***	23	7.9	***	28
94	Toliver and Kennel			65% Ex. + 20% Fut.	5.0	***	20	8.5	***	25
82	Toliver	84		65% Ex. + 20% Fut.	5.0	***	20	8.5	***	25
76	Toliver	78		65% Ex. + 20% Fut.	5.0	***	20	8.5	***	24
74	Toliver and Wittenberg			80% Ex. + 20% Fut.	5.9	***	18	10.1	***	22
72	Toliver			80% Ex. + 20% Fut.	5.9	***	18	10.1	***	22
56	Toliver and Ridings	58		80% Ex. + 20% Fut.	5.9	***	18	10.1	***	22

CITY OF MOLALLA - WASTEWATER FACILITY I										
SANITARY SEWER SYSTEM INVENTORY										
MH / Junction Information				**Capacity Analysis**	**(PDAF Analysis)**			**(PIF Analysis)**		
MH / Junction Label	Location	Branch	Split to:	Design Discharge	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)	Design Discharge (mgd)	Excess Capacity (cfs)	Min. Circ. Pipe Req'd (in)
54	Toliver			80% Ex. + 20% Fut.	5.9	***	18	10.1	***	22
50	Toliver	51		80% Ex. + 20% Fut.	5.9	***	18	10.1	***	22
28	Toliver and Leroy	30		40% Ex. +10% Fut.	3.0	***	19	5.0	***	23
27	Toliver and Leroy		518	40% Ex. +10% Fut.	3.0	***	19	5.0	***	23
26	Toliver			40% Ex. +10% Fut.	3.0	***	19	5.0	***	23
24	Toliver			40% Ex. +10% Fut.	3.0	***	19	5.0	***	23
23	Toliver			40% Ex. +10% Fut.	3.0	***	19	5.0	***	23
22	Toliver	518		60% Ex. + 20% Fut.	4.7	***	16	7.9	***	19
20	Toliver			60% Ex. + 20% Fut.	4.7	***	16	7.9	***	19
18	Toliver			60% Ex. + 20% Fut.	4.7	***	15	7.9	***	18
16	Toliver			60% Ex. + 20% Fut.	4.7	***	16	7.9	***	19
14	Toliver			60% Ex. + 20% Fut.	4.7	***	16	7.9	***	19
12	Toliver and Hwy 213			65% Ex. + 45% Fut.	6.1	***	22	10.4	***	27
10	Toliver			65% Ex. + 45% Fut.	6.1	***	20	10.4	***	24
8	Toliver			65% Ex. + 45% Fut.	6.1	***	22	10.4	***	27
6	west of Hwy 211	500, 8		100% Ex. + 100% Fut.	10.9	***	27	18.5	***	33
4	west of Hwy 211			100% Ex. + 100% Fut.	10.9	***	27	18.5	***	33
2	west of Hwy 211			100% Ex. + 100% Fut.	10.9	***	27	18.5	***	33

IMPROVEMENT ALTERNATIVE COST ESTIMATES

The following pages include detailed spreadsheet cost estimates for the improvement alternatives. All the total project cost estimates include an allowance for mobilization/ demobilization as well as 20 percent construction contingencies and 25 percent for engineering, legal, and administrative costs for the design and implementation of the project. All total project cost estimates are shown in 2000 dollars. The total project cost estimates are for budget information only and may be regarded reliable within a margin of plus or minus 20 percent. The annual cost estimates may be regarded reliable within a margin of plus or minus 40 percent.

CITY OF MOLALLA								
WASTEWATER FACILITIES PLAN								
COLLECTION SYSTEM IMPROVEMENTS								
			Pipe	**Unit**			**Non-Growth**	
Item	**Qty**	**Unit**	**Size**	**Cost**	**Construction**	**Total Project**	**Size**	**Total Project**
			(in)		**Cost**	**Cost ***	**(in)**	**Cost ***
Upgrade WWTP Trunk	780	LF	30	$240.00	$187,000	$271,000	24	$217,000
	1,180	LF	24	$192.00	$227,000	$329,000	18	$246,000
Upgrade WWTP Trunk Totals					$414,000	$600,000		$463,000
Upgrade Toliver Trunk	7,900	LF	24	$192.00	$1,517,000	$2,200,000	18	$1,650,000
Molalla / Highway 211 Improvements	330	LF	12	$96.00	$32,000	$46,000		
Upgrade Bear Creek Trunk	1,830	LF	21	$168.00	$307,000	$445,000		
System Extension - New Industrial (South) Trunk	6,200	LF	12	$96.00	$595,000	$863,000		
System Extension - New Hwy 213 (South) Trunk	2,200	LF	12	$96.00	$211,000	$306,000		
TOTAL					**$3,080,000**	**$4,460,000**		
* Total Project Costs include the following:								
Construction Contingencies (% of total)						20%		
Engineering, Architecture, Administration, Legal Fees (% of total)						25%		

CITY OF MOLALLA					
WASTEWATER FACILITIES PLAN					
TREATMENT PLANT ALTERNATIVES					
Preliminary Treatment					
Fine Screen with Washing and Compaction (using Auger Monster)					
					Non-Growth
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**	**Costs**
Mobilization (percentage of total)	8%	LS	$32,200.00	$32,200	$27,000
Demolish Existing Headworks	1	LS	$5,000.00	$5,000	$5,000
Excavation/Backfill	300	CY	$20.00	$6,000	$6,000
Fine Screens, Two	1	LS	$168,000.00	$168,000	$134,400
Coarse Screen	1	LS	$10,000.00	$10,000	$10,000
Equipment Installation	1	LS	$53,000.00	$53,000	$42,400
Influent Flow Measurement	1	LS	$20,000.00	$20,000	$20,000
Concrete	40	CY	$500.00	$20,000	$16,000
Canopy	400	SF	$50.00	$20,000	$16,000
Piping	1	LS	$40,000.00	$40,000	$32,000
Electrical	1	LS	$60,000.00	$60,000	$60,000
Construction Subtotal				$434,000	$369,000
Construction Contingencies (% of total)			20%	$87,000	$74,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$109,000	$92,000
Construction Total				**$630,000**	**$535,000**

CITY OF MOLALLA					
WASTEWATER FACILITIES PLAN					
TREATMENT PLANT ALTERNATIVES					
Preliminary Treatment					
Fine Screen with Washing and Compaction (using Auger Monster)					
Grit Removal					
					Non-Growth
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**	**Costs**
Mobilization (percentage of total)	8%	LS	$64,000.00	$64,000	$53,000
Demolish Existing Headworks	1	LS	$5,000.00	$5,000	$5,000
Excavation/Backfill	400	CY	$20.00	$8,000	$8,000
Fine Screens, Two	1	LS	$ 168,000.00	$168,000	$134,400
Coarse Screen	1	LS	$ 10,000.00	$10,000	$10,000
Grit Equipment	1	LS	$ 220,000.00	$220,000	$176,000
Equipment Installation	1	LS	$119,000.00	$119,000	$95,200
Influent Flow Measurement	1	LS	$20,000.00	$20,000	$20,000
Concrete	40	CY	$500.00	$20,000	$16,000
Canopy	400	SF	$50.00	$20,000	$16,000
Grit Building	300	SF	$100.00	$30,000	$24,000
Piping	1	LS	$80,000.00	$80,000	$64,000
Electrical	1	LS	$100,000.00	$100,000	$100,000
Construction Subtotal				$864,000	$722,000
Construction Contingencies (% of total)			20%	$173,000	$144,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$216,000	$181,000
Construction Total				**$1,253,000**	**$1,047,000**

CITY OF MOLALLA					
WASTEWATER FACILITIES PLAN					
TREATMENT PLANT ALTERNATIVES					
Pump Station					
3 VFD Vertical Turbine Pumps					
					Non-Growth
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**	**Costs**
Mobilization (percentage of total)	8%	LS	$71,000.00	$71,000	$59,000
Pump Station Excavation/Backfill	320	CY	$20.00	$6,000	$6,000
Wet Well Walls, Base, and Top	86	CY	$500.00	$43,000	$34,400
Bypass Pumping	30	DAY	$1,000.00	$30,000	$30,000
Pumps	3	EA	$50,000.00	$150,000	$120,000
Equipment Installation	1	LS	$45,000.00	$45,000	$36,000
Pump Station Building	600	SF	$100.00	$60,000	$48,000
Piping and Valving	1	LS	$120,000.00	$120,000	$96,000
Electrical and Controls	1	LS	$140,000.00	$140,000	$140,000
Force Main, Parallel 18-in	1,800	LF	$160.00	$288,000	$230,400
Construction Subtotal				$953,000	$800,000
Construction Contingencies (% of total)			20%	$191,000	$160,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$238,000	$200,000
Construction Total				**$1,382,000**	**$1,160,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Secondary Treatment				
Aerated Lagoons				
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$76,000.00	$76,000
Lagoon Fill	15,000	CY	$20.00	$300,000
Lagoon Liner	10,500	SY	$10.00	$105,000
Aeration Equipment	1	LS	$60,000.00	$60,000
Moving Existing Equipment	1	LS	$30,000.00	$30,000
Aeration Equipment Installation	1	LS	$45,000.00	$45,000
Piping	1	LS	$150,000.00	$150,000
Electrical	1	LS	$200,000.00	$200,000
Pump/Blower/Control Building	600	SF	$100.00	$60,000
Construction Subtotal				$1,026,000
Construction Contingencies (% of total)			20%	$205,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$257,000
Construction Total				**$1,488,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Secondary Treatment				
Phase 2 Secondary Clarification and Additional Aeration				
Item	**Qty**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$146,000.00	$146,000
Aeration Equipment	1	LS	$60,000.00	$60,000
Aeration Equipment Installation	1	LS	$30,000.00	$30,000
Site Work	1	LS	$10,000.00	$10,000
Excavation/Backfill	2,600	CY	$20.00	$52,000
Secondary Clarifier Tankage - Base	700	CY	$400.00	$280,000
Secondary Clarifier Tankage - Walls	520	CY	$600.00	$312,000
Secondary Clarifier Equipment	2	EA	$110,000.00	$220,000
Secondary Clarifier Equipment Instal.	1	LS	$110,000.00	$110,000
RAS/WAS Pumping	1	LS	$90,000.00	$90,000
RAS/WAS Pump Building	600	SF	$100.00	$60,000
Piping	1	LS	$300,000.00	$300,000
Electrical	1	LS	$300,000.00	$300,000
Construction Subtotal				$1,970,000
Construction Contingencies (% of total)			20%	$394,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$493,000
Construction Total				**$2,857,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Advanced Treatment				
Expansion of Existing DAF and Filter Facilities				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$109,000.00	$109,000
DAF Tankage - Base	40	CY	$400.00	$16,000
DAF Tankage - Walls	25	CY	$600.00	$15,000
DAF Equipment	1	LS	$150,000.00	$150,000
DAF Equipment Installation	1	LS	$75,000.00	$75,000
Effluent Filter Equipment	1	LS	$360,000.00	$360,000
Effluent Filter Equipment Instal.	1	LS	$108,000.00	$108,000
Existing Equipment Upgrades	1	LS	$100,000.00	$100,000
Canopy	3,200	SF	$50.00	$160,000
Effluent Flow Measurement	1	EA	$30,000.00	$30,000
Piping	1	LS	$150,000.00	$150,000
Electrical	1	LS	$200,000.00	$200,000
Construction Subtotal				$1,473,000
Construction Contingencies (% of total)			20%	$295,000
Engineering, Architecture, Admin, Legal Fees (% of total)			25%	$368,000
Construction Total				**$2,136,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Secondary Treatment				
Oxidation Ditch - Concrete Tanks				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$376,000.00	$376,000
Site Work	1	LS	$40,000.00	$40,000
Excavation/Backfill	11,400	CY	$20.00	$228,000
Oxidation Ditch Tankage - Base	2,240	CY	$400.00	$896,000
Oxidation Ditch Tankage - Walls	920	CY	$600.00	$552,000
Oxidation Ditch Equipment	1	LS	$260,000.00	$260,000
Oxidation Ditch Equipment Installation	1	LS	$78,000.00	$78,000
Secondary Clarifier Tankage - Base	700	CY	$400.00	$280,000
Secondary Clarifier Tankage - Walls	520	CY	$600.00	$312,000
Secondary Clarifier Equipment	2	EA	$110,000.00	$220,000
Secondary Clarifier Equipment Installation	1	LS	$110,000.00	$110,000
RAS/WAS Pumping	1	LS	$90,000.00	$90,000
RAS/WAS Pump Building	600	SF	$100.00	$60,000
Effluent Flow Measurement	1	EA	$30,000.00	$30,000
Piping	1	LS	$625,000.00	$625,000
Electrical	1	LS	$625,000.00	$625,000
Control Building	3,000	SF	$100.00	$300,000
Construction Subtotal				$5,082,000
Construction Contingencies (% of total)			20%	$1,016,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$1,271,000
Construction Total				**$7,369,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Secondary Treatment				
Sequencing Batch Reactor				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$379,800.00	$379,800
Site Work	1	LS	$40,000.00	$40,000
Excavation/Backfill	13,600	CY	$20.00	$272,000
SBR Tankage - Slab	2,500	CY	$400.00	$1,000,000
SBR Tankage - Walls	1,050	CY	$600.00	$630,000
SBR Equipment	1	LS	$800,000.00	$800,000
SBR Equipment Installation	1	LS	$240,000.00	$240,000
Equalization Basin - Slab	480	CY	$400.00	$192,000
Equalization Basin - Walls	90	CY	$600.00	$54,000
Flow Equalization Equipment	1	LS	$50,000.00	$50,000
Effluent Flow Measurement	1	EA	$30,000.00	$30,000
Piping	1	LS	$520,000.00	$520,000
Electrical	1	LS	$620,000.00	$620,000
Pump/Blower/Control Building	3,000	SF	$100.00	$300,000
Construction Subtotal				$5,128,000
Construction Contingencies (% of total)			20%	$1,026,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$1,282,000
Construction Total				**$7,436,000**

CITY OF MOLALLA			
WASTEWATER FACILITIES PLAN			
TREATMENT PLANT ALTERNATIVES			
Annual Cost Estimates for Secondary and Advanced Treatment Facilities *			
	Expand with	**Convert to**	**Convert to**
O&M Item	**Similar Treatment**	**Oxidation Ditch**	**SBR**
@ START-UP CONDITION (1)			
Manpower (3)	67,000	67,000	67,000
Electricity (4)	67,000	32,000	48,000
Maintenance / Replacement (5)	76,000	36,000	48,000
Miscellaneous (6)	42,000	27,000	33,000
TOTAL Annual Cost @ Start-up Condition	$252,000	$162,000	$196,000
@ DESIGN CONDITION(2)			
Manpower (3)	85,000	85,000	85,000
Electricity (4)	93,000	54,000	74,000
Maintenance / Replacement (5)	82,000	40,000	50,000
Miscellaneous (6)	52,000	36,000	42,000
TOTAL Annual Cost @ Design Condition	$312,000	$215,000	$251,000
PRESENT WORTH @ 6% and 20 years	**$4,870,000**	**$3,280,000**	**$3,870,000**
* All costs shown in 2000 dollars.			
(1) Start-up condition based on the MMDWF of 1.7 mgd.			
(2) Design condition based on the MMDWF of 4.0 mgd.			
(3) Based on two full-time operators			
(4) Based on $0.06 per kilowatt-hour.			
(5) Includes equipment and chemical costs.			
(6) For insurance, contingencies, and testing.			

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Effluent Disinfection				
Gaseous Chlorine Disinfection with Sodium Bisulfite Disinfection				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$20,000.00	$20,000
Site Work	1	LS	$10,000.00	$10,000
Chemical Feed System Upgrades	1	LS	$40,000.00	$40,000
Chlorine Residual Analyzer	1	EA	$10,000.00	$10,000
Chemical Equipment Installation	1	LS	$15,000.00	$15,000
Piping and Valving	1	LS	$20,000.00	$20,000
Electrical	1	LS	$40,000.00	$40,000
Chlorine Scrubber	1	LS	$110,000.00	$110,000
Construction Subtotal				$265,000
Construction Contingencies (% of total)			20%	$53,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$66,000
Construction Total				**$384,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Sludge Management				
Facultative Lagoon Only				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$39,400.00	$39,400
Lagoon Fill	12,000	CY	$16.00	$192,000
Lagoon Liner	20,000	SY	$10.00	$200,000
Piping	1	LS	$100,000.00	$100,000
Construction Subtotal				$531,000
Construction Contingencies (% of total)			20%	$106,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$133,000
Construction Total				**$770,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Sludge Management (not including Concrete Tank Diffused Air Secondary Treatment)				
Aerobic Digestion with Thickening and FSL Storage				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$115,600.00	$115,600
Digester Excavation/Backfill	3,450	CY	$20.00	$69,000
Digester Tankage - Base	690	CY	$400.00	$276,000
Digester Tankage - Walls	472	CY	$600.00	$283,200
Digester Aeration	1	LS	$80,000.00	$80,000
Digester Aeration Installation	1	LS	$40,000.00	$40,000
Storage Lagoon Fill	12,000	CY	$16.00	$192,000
Storage Lagoon Liner	12,500	SY	$10.00	$125,000
Piping	1	LS	$160,000.00	$160,000
Electrical / Controls	1	LS	$160,000.00	$160,000
Building	600	SF	$100.00	$60,000
Construction Subtotal				$1,561,000
Construction Contingencies (% of total)			20%	$312,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$390,000
Construction Total				**$2,263,000**

CITY OF MOLALLA				
WASTEWATER FACILITIES PLAN				
TREATMENT PLANT ALTERNATIVES				
Sludge Management				
Aerobic Digestion with Thickening and Lime Stabilization				
Item	**Quantity**	**Unit**	**Unit Cost**	**Total Cost**
Mobilization (percentage of total)	8%	LS	$171,500.00	$171,500
Digester Excavation/Backfill	3,450	CY	$20.00	$69,000
Digester Tankage - Base	170	CY	$400.00	$68,000
Digester Tankage - Walls	110	CY	$600.00	$66,000
Digester Aeration	1	LS	$40,000.00	$40,000
Digester Aeration Installation	1	LS	$20,000.00	$20,000
Belt Filter Press	1	LS	$420,000.00	$420,000
Lime Feed and Conveyance	1	LS	$50,000.00	$50,000
Lime Storage	36	CY	$400.00	$14,400
Lime/Sludge Mixing Tanks	2	CY	$5,000.00	$10,000
Storage - Base	620	CY	$400.00	$248,000
Storage - Wall	80	CY	$600.00	$48,000
Pole Barn over Storage	16,800	SF	$25.00	$420,000
Piping	1	LS	$250,000.00	$250,000
Electrical / Controls	1	LS	$300,000.00	$300,000
Building	1,200	SF	$100.00	$120,000
				.
Construction Subtotal				$2,315,000
Construction Contingencies (% of total)			20%	$463,000
Engineering, Architecture, Administration, Legal Fees (% of total)			25%	$579,000
Construction Total				**$3,357,000**

CITY OF MOLALLA
WASTEWATER FACILITIES PLAN
TREATMENT PLANT ALTERNATIVES

Annual Cost Estimates for Recommended Treatment Improvements *

Expand the Existing Plant Using Similar Treatment Processes, FSL Solids Management, Winter Discharge, and Summer Land Application

O&M Item	Prelim. Treatment	Influent PS	Sec./Adv. Treatment	Effluent Disinfec.	SUB TOTALS	Effluent Disposal	Sludge Mgmt	TOTALS
@ START-UP CONDITION (1)								
Manpower - total of 3 manyears (3)	6,000	6,000	67,000	6,000	85,000	9,000	6,000	100,000
Electricity (4)	1,000	8,000	67,000	2,000	78,000	40,000	4,000	122,000
Maintenance / Replacement (5)	4,000	7,000	76,000	4,000	91,000	40,000	44,000	175,000
Miscellaneous (6)	2,000	4,000	42,000	2,000	50,000	18,000	11,000	79,000
TOTAL Annual Cost @ Start-up Condition	$13,000	$25,000	$252,000	$14,000	$304,000	$107,000	$65,000	$476,000
@ DESIGN CONDITION(2)								
Manpower - total of 4 manyears	6,000	9,000	85,000	9,000	109,000	12,000	12,000	133,000
Electricity (4)	2,000	17,000	93,000	4,000	116,000	67,000	8,000	191,000
Maintenance / Replacement (5)	6,000	7,000	82,000	4,000	99,000	40,000	62,000	201,000
Miscellaneous (6)	3,000	7,000	52,000	3,000	65,000	24,000	16,000	105,000
TOTAL Annual Cost @ Design Condition	$17,000	$40,000	$312,000	$20,000	$389,000	$143,000	$98,000	$630,000
PRESENT WORTH @ 6% and 20 years					**$6,000,000**			**$9,610,000**

* All costs shown in 2000 dollars.
(1) Start-up condition based on the MMDWF of 1.7 mgd.
(2) Design condition based on the MMDWF of 4.0 mgd.
(3) Based on $3000 per man-month.
(4) Based on $0.06 per kilowatt-hour.
(5) Includes equipment and chemical costs.
(6) For insurance, contingencies, and testing.

FLOW MONITORING PROGRAM

The following spreadsheets and maps provide detail on the flow monitoring program discussed in Chapter 6.

CITY OF MOLALLA - WASTEWATER FACILITY PLAN
FLOW MONITORING

Flow Calculations based on field data

January 14, 1999 - 8:00a.m. to 12:00p.m.
0.4 inches rain measured at the STP overnight, raining lightly during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)
			Input Data			*Calculations*	
Toliver near Bear Creek diversion	22	east	15	5.50	5.1	1.34	3.8
		southeast	15	2.00			
Toliver and Leroy	28	east	15	10.00	1.6	0.90	2.9
		south	8	5.50	0.2	0.03	1.5
Toliver and Ridings	56	east	15	5.00	4.9	1.13	3.9
		south	10	2.50			
Toliver and Revilot	82	east	15	6.25	1.8	0.56	2.2
		north	12	2.75			
Heintz and Kennel	98	east	15	6.25	2.5	0.78	3.4
		south	8	1.75			
Molalla and Heintz	114	east	12	4.00	4.3	0.64	3.2
		south	12	3.00	1.0	0.10	3.0
		north	8	3.00	3.5	0.27	3.5
Hwy 211 and Lola	226	east	8	5.00	1.1	0.16	1.8
		south	8				
Heintz and Fenton	178	east	8	2.00	3.7	0.16	3.8
		south	8	2.00	0.8	0.04	3.7
Molalla and Shirley	116	east	8	4.50	1.9	0.25	5.6
		north	8	2.00	0.6	0.03	0.9
Hwy 211 and Ona	526	east	12	4.50	1.3	0.23	1.1
Hwy 211 and Shaver	546	south	12	4.75	1.2	0.22	1.5
Hart and 4th	570	east	8	4.00	1.5	0.17	4.0
		south	8	5.00			

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

January 22, 1999 - 9:00a.m. to 4:00p.m.

Rain and showers throughout preceding week, raining lightly but steadily during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Toliver east of Hwy 213	16	east	15					Surcharged
Toliver near Bear Creek	22	east	15	15.00	3.4	2.70	7.6	
diversion		southeast	15	8.25	1.3	0.58		
Toliver and Leroy	28	east	15	15.00	3.5	2.78	9.0	
		south	8					Surcharged
Leroy and Unnamed St	32	east						Surcharged
		south						Surcharged
Unnamed St and Carol	38	north	8					Low flows
		south	8					Low flows
Toliver and Ridings	56	east	15	8.00	5.7	2.45	8.3	
		south	10	6.50	1.1	0.27	11.1	
Ridings and Heintz	60	south	10	4.00	2.4	0.32	17.2	
Toliver and Revilot	82	east	15	14.00	3.1	2.39	9.2	
		north	12					Surcharged
Heintz and Kennel	98	east	15	15.00	2.5	1.98	8.6	
		south	8	1.75				Surcharged

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

January 22, 1999 - 9:00a.m. to 4:00p.m.

Rain and showers throughout preceding week, raining lightly but steadily during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Kennel to the south	100	south						Steady but low flows
Molalla and Heintz	114	east	12					Surcharged and splashing inside MH
		south	12					Surcharged and splashing inside MH
		north	8					Surcharged and splashing inside MH
Heintz east of Molalla	172	east	12	7.25	3.6	1.15	6.0	
Heintz and Grange	176	east	10	9.50	1.1	0.38	6.7	
		south	10	8.25	2.5	0.78	6.1	
Hwy 211 and Lola	226	east	8					Splashing inside MH due to drop from south
		south	8					Long drop from south
Lola and 2nd	302	east	8	3.75	1.6	0.17	17.3	
		south	8	1.75	1.1	0.04	5.6	
Hwy 211 and Fenton	228	east	8	4.75	3.9	0.54	6.1	
Hwy 211 west of Cole	230	east	8	6.00	1.2	0.22	2.5	Creek crossing between 230 and 232
		south	6					Low flows
Hwy 211 east of Cole	232	east	8	3.50	6.4	0.61	7.9	Creek crossing between 230 and 232
		south	6	2.00	1.0	0.04	6.6	

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

January 22, 1999 - 9:00a.m. to 4:00p.m.

Rain and showers throughout preceding week, raining lightly but steadily during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Heintz and Fenton	178	east	8	3.00	3.6	0.28	6.6	
		south	8	3.00	0.8	0.06	6.4	
Heintz and Cole	182	south	8	4.25	3.4	0.41	20.7	
		east	8	4.25				Low flows
		north	8					Low flows
Cole and Patrol	200	east	8					Steady but low flows
		south	8					Low flows
Molalla and Shirley	116	east	8					Surcharged
		north	8					Surcharged
Shirley east of Molalla	134	east	8	4.50	1.2	0.16	3.9	
Shirley and Fenton	154	east	8	2.25	2.6	0.14	7.4	
		south	8					Low flows
Shirley and Cole	160	east	8					Steady but low flows
		north	8					Low flows
Molalla and Francis	118	east	8					Too much traffic
		north	8					Too much traffic
Francis east of Molalla	122	east	8					Low flows

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

January 22, 1999 - 9:00a.m. to 4:00p.m.

Rain and showers throughout preceding week, raining lightly but steadily during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Hwy 213 and Bear Creek	500	southeast	15	8.50	5.3	2.46	10.3	
		north	12	4.25	1.0	0.16	2.3	From Big Meadow development
Hwy 213 north of Bear		north	12	2.50	1.8	0.14	2.0	From Big Meadow development
Industrial Way	510	east	15	5.50	2.9	0.76		Diversion - no unit flow calculated
Hwy 211 and Shaver	546	south	12	8.00	2.4	0.86	5.7	
Hart and 3rd	566	east	8					Low flows
		south	8	3.75	4.1	0.43	6.9	
Hart and 4th	570	east	8	6.75	3.3	0.67	15.8	
		south	8					Low flows
Hart and 5th	572	east	8	1.50				Low flows
		south	8	3.50				Low flows; sand in pipe
Metzler and 5th	580	east	8	6.00	1.6	0.29	9.6	
		south	8	6.00	0.4	0.07	10.1	
Molalla and 5th	586	east	8	4.50	1.1	0.14	8.6	
		south	8	3.00	1.2	0.09	8.3	

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

February 18-19, 1999 - 11:00p.m. to 3:00a.m.

Rain and showers throughout preceding week up to four hours before monitoring, no rain during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Toliver west of Hwy 213	10	east	15	11.25	2.7	1.72	4.7	Confirm pipe size
Toliver and Ridings	56	east	15	5.50	5.5	1.45	4.9	
		south	10	2.00				Low flows
Heintz and Kennel	98	east	15	7.50	3.1	1.23	5.3	
		south	8	3.00				Low flows
Hwy 211 and Fenton	228	east	8	5.00	1.2	0.18	2.0	
Hwy 211 east of Cole	232	east	8	2.00	4.0	0.18	2.3	Creek crossing between 230 and 232
		south	6	6.00				Low flows
Hwy 211 and Stowers	238	south	8	2.75	2.8	0.19	2.6	
		east	8	0.50				Low flows
Heintz and Fenton	178	east	8	2.00	2.5	0.11	2.6	
		south	8	1.50				Low flows
Molalla and Shirley	116	east	8	4.50	2.2	0.29	6.5	
		north	8	2.75	1.2	0.08	2.7	

CITY OF MOLALLA - WASTEWATER FACILITY PLAN

FLOW MONITORING

Flow Calculations based on field data

February 18-19, 1999 - 11:00p.m. to 3:00a.m.

Rain and showers throughout preceding week up to four hours before monitoring, no rain during the flow monitoring

MH Location	MH Number	Pipe Location	Dia. (in)	Depth (in)	Velocity (fps)	Flow (mgd)	Unit Flow (gpd/in.dia.-ft.)	Comments
			Input Data			*Calculations*		
Hwy 213 and Bear Creek	500	southeast	15	5.50	4.5	1.19	5.0	
		north	12	2.00	2.1	0.12	1.7	From Big Meadow development
Industrial Way	510	east	15	2.50	1.8	0.16		Diversion - no unit flow calculated
Toliver / Bear split	518	east	12	5.75	1.2	0.29		Diversion - no unit flow calculated
Hwy 211 and Shaver	546	south	12	6.00	2.2	0.56	3.7	
Hwy 213 and Hart	562	east	12	3.50	2.9	0.36	5.4	
		south	8	3.00	4.8	0.37	5.8	
Hart and 4th	570	east	8	4.25	2.4	0.29	6.9	
		south	8	6.00				Low flows
Metzler and 5th	580	east	8	3.50	1.4	0.13	4.4	
		south	8	3.00	0.2	0.02	2.1	

WATER BALANCE EVALUATION

The following spreadsheets include the water balance evaluation used in Chapters 7 and 8. The spreadsheets include an estimation of existing conditions and projections of future conditions based on the improvement recommendations in Chapter 7.

CITY OF MOLALLA
WASTEWATER FACILITIES PLAN

MONTHLY WATER BALANCE **Existing Flows with Existing WWTP Facilities**

Influent Flow Information:	Existing (1998) AWWF:	1.55	mgd
	Existing (1998) ADWF:	0.82	mgd
Lagoon Information:	Average Lagoon area:	27.50	acre
	Assumed Level in Storage at end of Winter:	3.0	ft
	Maximum Storage Water Depth:	12.0	ft
	Maximum Storage Available:	300.0	ac-ft
Irrigation Information:	Irrigation Area:	85.0	acre

Month	Influent (1)		Precipitation (2)		Evaporation (3)		Discharge (4)		Irrigation (5)		Net Storage	Stor. Accum.
	(MG)	(ac-ft)	(in)	(ac-ft)	(in)	(ac-ft)	(mgd)	(ac-ft)	(in)	(ac-ft)	(ac-ft)	(ac-ft)
												82.5
May	25.4	78.0	2.59	5.9	4.10	-9.4	0.00	0.0	0.00	0.0	74.6	157.1
Jun	24.6	75.5	2.07	4.7	5.10	-11.7	0.00	0.0	3.23	-22.9	45.7	202.7
Jul	25.4	78.0	0.52	1.2	6.90	-15.8	0.00	0.0	6.85	-48.5	14.8	217.6
Aug	25.4	78.0	1.07	2.5	6.20	-14.2	0.00	0.0	5.23	-37.1	29.2	246.8
Sep	24.6	75.5	2.02	4.6	4.20	-9.6	0.00	0.0	0.00	0.0	70.5	317.3
Oct	25.4	78.0	4.29	9.8	1.90	-4.4	0.00	0.0	0.00	0.0	83.5	400.8
Nov	46.5	142.7	6.38	14.6	0.00	0.0	1.58	-145.5	0.00	0.0	11.9	412.6
Dec	48.1	147.5	7.13	16.3	0.00	0.0	1.58	-150.3	0.00	0.0	13.5	426.1
Jan	48.1	147.5	7.31	16.8	0.00	0.0	1.58	-150.3	0.00	0.0	13.9	440.0
Feb	43.4	133.2	4.99	11.4	0.00	0.0	1.58	-135.8	0.00	0.0	8.9	448.9
Mar	48.1	147.5	5.13	11.8	0.00	0.0	1.58	-150.3	0.00	0.0	8.9	457.8
Apr	46.5	142.7	3.20	7.3	3.10	-7.1	1.58	-145.5	0.00	0.0	-2.5	455.2
			46.70	**Total**	**31.50**	**Total**			**15.32**	**Total**	**Required**	**457.8**

1) Influent based on AWWF and ADWF.
2) Precipitation based on historical means at the Oregon City NCDC Station.
3) Evaporation based on historical means for Corvalis in the Climatoligical Handbook, September 1969.
4) Discharge to Bear Creek based on allowable discharge with 5 mg/l BOD in effluent and 66 pound BOD per day permit limit.
5) Irrigation based on application rates for corn (multiplied by 1.15 for feed corn) in "Consumptive Use and Net Irrigation Requirements for Oregon", OSU, 1968.

CITY OF MOLALLA
WASTEWATER FACILITIES PLAN

MONTHLY WATER BALANCE **Existing Flows with Existing WWTP Facilities**

Because the Water Balance for Existing Flows with Existing WWTP Facilities in the previous sheet indicated that a water balance over the year is not achieved, additional effluent disposal was added in this sheet to approximate existing conditions.

Additional means of effluent disposal are:

Leakage from the lagoon.
Additional discharge allowed because of better than 5 mg/l BOD effluent.
Over-irrigation on the existing land application site.

Month	Initial Net Storage (ac-ft)	Leakage (1) (in)	Leakage (1) (ac-ft)	Additional Discharge (2) (mgd)	Additional Discharge (2) (ac-ft)	Additional Irrigation (3) (in)	Additional Irrigation (3) (ac-ft)	Updated Net Storage (ac-ft)	Updated Stor. Accum. (ac-ft)
									82.5
May	74.6	1.94	-4.4	0.00	0.0	2.00	-14.2	55.9	138.4
Jun	45.7	1.88	-4.3	0.00	0.0	3.23	-22.9	18.5	156.9
Jul	14.8	1.94	-4.4	0.00	0.0	6.85	-48.5	-38.1	118.8
Aug	29.2	1.94	-4.4	0.00	0.0	5.23	-37.1	-12.3	106.5
Sep	70.5	1.88	-4.3	0.00	0.0	2.00	-14.2	52.0	158.5
Oct	83.5	1.94	-4.4	0.00	0.0	0.00	0.0	79.1	237.6
Nov	11.9	1.88	-4.3	0.00	0.0	0.00	0.0	7.6	245.1
Dec	13.5	1.94	-4.4	0.40	-38.1	0.00	0.0	-29.0	216.1
Jan	13.9	1.94	-4.4	0.40	-38.1	0.00	0.0	-28.6	187.5
Feb	8.9	1.75	-4.0	0.40	-34.4	0.00	0.0	-29.5	158.0
Mar	8.9	1.94	-4.4	0.40	-38.1	0.00	0.0	-33.6	124.4
Apr	-2.5	1.88	-4.3	0.40	-36.8	0.00	0.0	-43.7	82.5
		22.81	**Total**			**19.32**	**Total**	**Required**	**245.1**

1) Leakage assumes 1/16-inch per day leakage.
2) Discharge to Bear Creek is in addition to the discharge of 1.58 mgd shown on the previous sheet. Discharge is based on allowable discharge with 4 mg/l BOD in effluent and 66 pound BOD per day permit limit from December through April.
3) Irrigation is in addition to the irrigation shown on the previous sheet. Irrigation assumes application of double the recommended hydraulic load for irrigation and irrigating 2 inches in each May and September.

CITY OF MOLALLA
EFFLUENT PUMP STATION, FORCEMAIN, AND OUTFALL

MONTHLY WATER BALANCE **Year 2000 Flows with Effluent PS and Piping**

Influent Flow Information:	Year 2000 AWWF:		1.93	mgd
	Year 2000 ADWF:		1.07	mgd
Lagoon Information:	Average Lagoon area:	Existing:	27.50	acre
	Assumed Level at beginning of Summer:	Existing:	3.0	ft
	Maximum Storage Water Depth:	Existing:	12.0	ft
	Maximum Storage Available:	Existing:	300.0	ac-ft
Irrigation Information:	Irrigation Area:	@ WWTP and Slim:	85.0	acre

@ Coleman: 231.0 acre

Month	Influent (1) (MG)	Influent (1) (ac-ft)	Precipitation (2) (in)	Precipitation (2) (ac-ft)	Evaporation (3) (in)	Evaporation (3) (ac-ft)	Discharge (4) Bear (mgd)	Discharge (4) Molalla (mgd)	Discharge (4) Total (ac-ft)	Irrigation (5) (in)	Irrigation (5) (ac-ft)	Net Storage (ac-ft)	Stor. Accum. (ac-ft)
													82.5
May	50.6	155.3	2.59	5.9	6.04	-13.8	0.00	0.00	0.0	1.10	-29.0	118.4	200.9
Jun	28.9	88.8	2.07	4.7	6.98	-16.0	0.00	0.00	0.0	3.12	-82.2	-4.6	196.3
Jul	24.2	74.3	0.52	1.2	8.84	-20.3	0.00	0.00	0.0	5.28	-139.1	-83.8	112.5
Aug	23.2	71.3	1.07	2.5	8.14	-18.6	0.00	0.00	0.0	4.55	-119.8	-64.6	47.8
Sep	29.3	90.0	2.02	4.6	6.08	-13.9	0.00	0.00	0.0	1.95	-51.3	29.4	77.2
Oct	40.6	124.5	4.29	9.8	3.84	-8.8	0.00	0.00	0.0	0.00	0.0	125.5	202.8
Nov	49.6	152.2	6.38	14.6	1.88	-4.3	2.23	0.00	-205.3	0.00	0.0	-42.8	160.0
Dec	67.4	206.9	7.13	16.3	1.94	-4.4	2.23	0.00	-212.2	0.00	0.0	6.7	166.7
Jan	67.8	208.0	7.31	16.8	1.94	-4.4	2.23	0.00	-212.2	0.00	0.0	8.1	174.8
Feb	57.3	175.8	4.99	11.4	1.75	-4.0	2.23	0.00	-191.6	0.00	0.0	-8.4	166.4
Mar	61.8	189.8	5.13	11.8	1.94	-4.4	2.23	0.00	-212.2	0.00	0.0	-15.1	151.3
Apr	45.8	140.4	3.20	7.3	4.98	-11.4	2.23	0.00	-205.3	0.00	0.0	-68.9	82.4
			46.70	**Total**	**54.31**	**Total**				**16.00**	**Total**	**Required**	**202.8**

1) Influent based on AWWF and ADWF and historical distribution of influent flows.
2) Precipitation based on historical means at the Oregon City NCDC Station.
3) Evaporation based on historical means for Corvalis in the Climatoligical Handbook, September 1969 and Leakage based on 1/16-inch per day.
4) Discharge to Bear Creek based on total discharge equal to capacity of the existing DAF and gravity filters, 2.23 mgd.
5) Irrigation based on application rate and distribution for pasture grass (15.27 inches per year) ratioed up to 16 inches per year.
Areas shown do not include buffers.

CITY OF MOLALLA
EFFLUENT PUMP STATION, FORCEMAIN, AND OUTFALL

MONTHLY WATER BALANCE **5-year Flows with Proposed Phase 1 WWTP Facilities**

Influent Flow Information:	5-year (2004) AWWF:		2.23 mgd				
	5-year (2004) ADWF:		1.25 mgd				
Lagoon Information:	Average Lagoon area:	Existing:	22.37 acre		Add'l:	1.83 acre	
	Assumed Level at beginning of Summer:	Existing:	3.0 ft		Add'l:	3.0 ft	
	Maximum Storage Water Depth:	Existing:	12.0 ft		Add'l:	12.0 ft	
	Maximum Storage Available:	Existing:	244.0 ac-ft		Add'l:	20.0 ac-ft	
Irrigation Information:	Irrigation Area:	@ WWTP and Slim:	55.0 acre		@ Coleman:	290.0 acre	

Month	Influent (1) (MG)	Influent (1) (ac-ft)	Precipitation (2) (in)	Precipitation (2) (ac-ft)	Evaporation (3) (in)	Evaporation (3) (ac-ft)	Discharge (4) Bear (mgd)	Discharge (4) Molalla (mgd)	Discharge (4) Total (ac-ft)	Irrigation (5) (in)	Irrigation (5) (ac-ft)	Net Storage (ac-ft)	Stor. Accum. (ac-ft)
													67.1
May	59.1	181.4	2.59	5.2	6.04	-12.2	0.00	0.00	0.0	1.10	-31.6	142.8	209.9
Jun	33.8	103.8	2.07	4.2	6.98	-14.1	0.00	0.00	0.0	3.12	-89.8	4.1	214.0
Jul	28.3	86.8	0.52	1.0	8.84	-17.8	0.00	0.00	0.0	5.28	-151.8	-81.8	132.3
Aug	27.1	83.3	1.07	2.2	8.14	-16.4	0.00	0.00	0.0	4.55	-130.7	-61.7	70.6
Sep	34.3	105.2	2.02	4.1	6.08	-12.3	0.00	0.00	0.0	1.95	-56.0	41.0	111.5
Oct	47.4	145.4	4.29	8.7	3.84	-7.7	0.00	0.00	0.0	0.00	0.0	146.3	257.9
Nov	57.3	175.9	6.38	12.9	1.88	-3.8	0.79	1.86	-244.0	0.00	0.0	-59.0	198.9
Dec	77.9	239.1	7.13	14.4	1.94	-3.9	0.79	1.86	-252.1	0.00	0.0	-2.6	196.3
Jan	78.3	240.3	7.31	14.7	1.94	-3.9	0.79	1.86	-252.1	0.00	0.0	-1.0	195.3
Feb	66.2	203.2	4.99	10.1	1.75	-3.5	0.79	1.86	-227.7	0.00	0.0	-18.0	177.3
Mar	71.4	219.3	5.13	10.3	1.94	-3.9	0.79	1.86	-252.1	0.00	0.0	-26.4	150.9
Apr	52.9	162.3	3.20	6.5	4.98	-10.0	0.79	1.86	-244.0	0.00	0.0	-85.3	65.6
			46.70	**Total**	**54.31**	**Total**				**16.00**	**Total**	**Required**	**257.9**

1) Influent based on AWWF and ADWF and historical distribution of influent flows.
2) Precipitation based on historical means at the Oregon City NCDC Station.
3) Evaporation based on historical means for Corvalis in the Climatoligical Handbook, September 1969 and Leakage based on 1/16-inch per day.
4) Discharge to Bear Creek based on allowable discharge with 10 mg/l BOD in effluent and 66 pound BOD per day permit limit.
Discharge to Molalla River based equalizing lagoon storage, but not to exceed total discharge equal to capacity of the prop. DAF & gravity filters, 4.46 mgd.
5) Irrigation based on application rate and distribution for pasture grass (15.27 inches per year) ratioed up to 16 inches per year.
Areas shown do not include buffers.

CITY OF MOLALLA
EFFLUENT PUMP STATION, FORCEMAIN, AND OUTFALL

MONTHLY WATER BALANCE **10-year Flows with Proposed Phase 1 WWTP Facilities and Future Storage**

Influent Flow Information:	10-year (2009) AWWF:		2.61	mgd			
	10-year (2009) ADWF:		1.58	mgd			
Lagoon Information:	Average Lagoon area:	Existing:	22.37	acre	Add'l:	8.25	acre
	Assumed Level at beginning of Summer:	Existing:	3.0	ft	Add'l:	3.0	ft
	Maximum Storage Water Depth:	Existing:	12.0	ft	Add'l:	12.0	ft
	Maximum Storage Available:	Existing:	244.0	ac-ft	Add'l:	90.0	ac-ft
Irrigation Information:	Irrigation Area:	@ WWTP and Slim:	55.0	acre	@ Coleman:	380.0	acre

Month	Influent (1) (MG)	Influent (1) (ac-ft)	Precipitation (2) (in)	Precipitation (2) (ac-ft)	Evap./Leakage (3) (in)	Evap./Leakage (3) (ac-ft)	Discharge (4) Bear (mgd)	Discharge (4) Molalla (mgd)	Discharge (4) Total (ac-ft)	Irrigation (5) (in)	Irrigation (5) (ac-ft)	Net Storage (ac-ft)	Stor. Accum. (ac-ft)
													91.9
May	74.7	229.3	2.59	6.6	6.04	-15.4	0.00	0.00	0.0	1.10	-39.9	180.6	272.5
Jun	42.7	131.2	2.07	5.3	6.98	-17.8	0.00	0.00	0.0	3.12	-113.2	5.5	277.9
Jul	35.8	109.7	0.52	1.3	8.84	-22.5	0.00	0.00	0.0	5.28	-191.4	-102.9	175.0
Aug	34.3	105.3	1.07	2.7	8.14	-20.8	0.00	0.00	0.0	4.55	-164.8	-77.6	97.4
Sep	43.3	132.9	2.02	5.2	6.08	-15.5	0.00	0.00	0.0	1.95	-70.6	52.0	149.4
Oct	59.9	183.8	4.29	10.9	3.84	-9.8	0.00	0.00	0.0	0.00	0.0	185.0	334.3
Nov	67.1	205.9	6.38	16.3	1.88	-4.8	0.79	2.35	-289.1	0.00	0.0	-71.7	262.6
Dec	91.2	279.8	7.13	18.2	1.94	-4.9	0.79	2.35	-298.7	0.00	0.0	-5.7	256.9
Jan	91.6	281.3	7.31	18.7	1.94	-4.9	0.79	2.35	-298.7	0.00	0.0	-3.8	253.2
Feb	77.5	237.8	4.99	12.7	1.75	-4.5	0.79	2.35	-269.8	0.00	0.0	-23.8	229.4
Mar	83.6	256.6	5.13	13.1	1.94	-4.9	0.79	2.35	-298.7	0.00	0.0	-34.0	195.4
Apr	61.9	189.9	3.20	8.2	4.98	-12.7	0.79	2.35	-289.1	0.00	0.0	-103.7	91.7
			46.70	**Total**	**54.31**	**Total**				**16.00**	**Total**	**Required**	**334.3**

1) Influent based on AWWF and ADWF and historical distribution of influent flows.
2) Precipitation based on historical means at the Oregon City NCDC Station.
3) Evaporation based on historical means for Corvalis in the Climatoligical Handbook, September 1969 and Leakage based on 1/16-inch per day.
4) Discharge to Bear Creek based on allowable discharge with 10 mg/l BOD in effluent and 66 pound BOD per day permit limit.
Discharge to Molalla River based equalizing lagoon storage, but not to exceed total discharge equal to capacity of the proposed DAF and gravity filters, 4.46 mgd.
5) Irrigation based on application rate and distribution for pasture grass (15.27 inches per year) ratioed up to 16 inches per year. Areas shown do not include buffers.

CITY OF MOLALLA
EFFLUENT PUMP STATION, FORCEMAIN, AND OUTFALL

MONTHLY WATER BALANCE **20-year Flows with Proposed Phase 2 WWTP Facilities and Future Storage**

Influent Flow Information:	20-year (2019) AWWF:		3.23 mgd				
	20-year (2019) ADWF:		2.12 mgd				
Lagoon Information:	Average Lagoon area:	Existing:	22.37 acre		Add'l:	16.13 acre	
	Assumed Level at beginning of Summer:	Existing:	3.0 ft		Add'l:	3.0 ft	
	Maximum Storage Water Depth:	Existing:	12.0 ft		Add'l:	12.0 ft	
	Maximum Storage Available:	Existing:	244.0 ac-ft		Add'l:	176.0 ac-ft	
Irrigation Information:	Irrigation Area:	@ WWTP and Slim:	55.0 acre		@ Coleman:	550.0 acre	

Month	Influent (1) (MG)	Influent (1) (ac-ft)	Precipitation (2) (in)	Precipitation (2) (ac-ft)	Evap./Leakage (3) (in)	Evap./Leakage (3) (ac-ft)	Discharge (4) Bear (mgd)	Discharge (4) Molalla (mgd)	Discharge (4) Total (ac-ft)	Irrigation (5) (in)	Irrigation (5) (ac-ft)	Net Storage (ac-ft)	Stor. Accum. (ac-ft)
													115.5
May	100.3	307.7	2.59	8.3	6.04	-19.4	0.00	0.00	0.0	1.10	-55.5	241.1	356.6
Jun	57.3	176.0	2.07	6.6	6.98	-22.4	0.00	0.00	0.0	3.12	-157.4	2.8	359.5
Jul	48.0	147.3	0.52	1.7	8.84	-28.4	0.00	0.00	0.0	5.28	-266.2	-145.7	213.8
Aug	46.0	141.3	1.07	3.4	8.14	-26.1	0.00	0.00	0.0	4.55	-229.3	-110.7	103.1
Sep	58.1	178.4	2.02	6.5	6.08	-19.5	0.00	0.00	0.0	1.95	-98.3	67.1	170.2
Oct	80.4	246.6	4.29	13.8	3.84	-12.3	0.00	0.00	0.0	0.00	0.0	248.1	418.3
Nov	83.0	254.8	6.38	20.5	1.88	-6.0	0.79	3.10	-358.2	0.00	0.0	-88.9	329.4
Dec	112.8	346.3	7.13	22.9	1.94	-6.2	0.79	3.10	-370.1	0.00	0.0	-7.1	322.2
Jan	113.4	348.1	7.31	23.5	1.94	-6.2	0.79	3.10	-370.1	0.00	0.0	-4.8	317.5
Feb	95.9	294.3	4.99	16.0	1.75	-5.6	0.79	3.10	-334.3	0.00	0.0	-29.6	287.8
Mar	103.5	317.6	5.13	16.5	1.94	-6.2	0.79	3.10	-370.1	0.00	0.0	-42.3	245.6
Apr	76.6	235.1	3.20	10.3	4.98	-16.0	0.79	3.10	-358.2	0.00	0.0	-128.8	116.8
			46.70	**Total**	**54.31**	**Total**				**16.00**	**Total**	**Required**	**418.3**

1) Influent based on AWWF and ADWF and historical distribution of influent flows.
2) Precipitation based on historical means at the Oregon City NCDC Station.
3) Evaporation based on historical means for Corvalis in the Climatoligical Handbook, September 1969 and Leakage based on 1/16-inch per day.
4) Discharge to Bear Creek based on allowable discharge with 10 mg/l BOD in effluent and 66 pound BOD per day permit limit.
Discharge to Molalla River based equalizing lagoon storage - only stored flows need to be treated by the advanced treatment facilities.
5) Irrigation based on application rate and distribution for pasture grass (15.27 inches per year) ratioed up to 16 inches per year. Areas shown do not include buffers.

NPDES PERMIT

The following is the City's current National Pollution Discharge Elimination System (NPDES) permit.

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM WASTE DISCHARGE PERMIT

Department of Environmental Quality
811 S.W. Sixth Avenue, Portland, OR 97204
Telephone: (503) 229-5696

Issued pursuant to ORS 468.740 and The Federal Clean Water Act

ISSUED TO:

CITY OF MOLALLA
P O BOX 248
MOLALLA OR 97038

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Domestic Sewage	001	R.M. 10.0
	002	Reuse

PLANT TYPE AND LOCATION:

Pre-Aerated Lagoon with
Effluent Filtration
12424 Toliver Rd
Treatment System Class: III
Collection System Class: II

RECEIVING SYSTEM INFORMATION:

Basin: Willamette
Sub-Basin: Pudding
Stream: Bear Creek
Hydro Code: 22K-PUDD 10.0 D
County: Clackamas

EPA REFERENCE NO: OR 002238-1

Issued in response to Application No. 997738 received October 22, 1991.

This permit is issued based on the land use findings in the permit record.

Charles K. Ashbaker, Manager
Water Quality, Northwest Region

APR 30 1992
Date

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

	Page
Schedule A - Waste Disposal Limitations not to be Exceeded...	2-3
Schedule B - Minimum Monitoring and Reporting Requirements...	4-5
Schedule C - Compliance Conditions and Schedules.....	6
Schedule D - Special Conditions.....	7-8
General Conditions.....	Attached

Each other direct and indirect discharge to public waters is prohibited.

This permit does not relieve the permittee from responsibility for compliance with any other applicable federal, state, or local law, rule, standard, ordinance, order, judgment, or decree.

SCHEDULE A

1. **Waste Discharge Limitations not to be Exceeded After Permit Issuance.**

a. Outfall Number 001 (Bear Creek)

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
(1) BOD-5	10 mg/l	15 mg/l	66	99	132
(2) TSS	10 mg/l	15 mg/l	66	99	132
(3) FC/100ml	200	400			

(4) BOD and TSS efficiency shall not be less than 85%.

(5) Effluent BOD-5 concentrations in mg/l, divided by the dilution factor (ratio of receiving streamflow to effluent flow) shall not exceed one.

*Based on average dry weather design flow to the facility equaling 0.79 MGD.

b. Outfall Number 002 (reuse)

(1) Prior to Department approval of the permittee's Reclaimed Water (Effluent) Use Plan as required in Schedule D, effluent wastewater shall be sufficiently well treated and disinfected to reduce fecal coliform to 10 organisms/100 ml monthly average with no sample to exceed 23 organisms/100 ml.

(2) Upon approval of the permittee's Reclaimed Water Use Plan, effluent shall receive at least Level II treatment as defined in Oregon Administrative Rules Chapter 340, Division 55 to reduce total coliform to 240 organisms/100 ml in two consecutive samples, and a 7-day median of 23 organisms/100 ml.

(3) All wastewater distributed on land shall dissipate by evapotranspiration and controlled seepage using sound irrigation practices so as to prevent:

a. Prolonged ponding of wastewater onto the ground surface;

b. Surface runoff or subsurface drainage through drainage tile;

c. The creation of odors, fly and mosquito breeding and other nuisance conditions;

d. The overloading of land with nutrients or organics.

c. Other parameters (year-round)

pH	Shall be within the range 6.0 - 9.0

2. Not withstanding the effluent limitations established by this permit, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-41-445 except in the defined mixing zone:

From the point of discharge to 50 feet downstream.

SCHEDULE B

1. Minimum Monitoring and Reporting Requirements.
(unless otherwise approved in writing by the Department)

a. Outfall Numbers 001 (discharge to Bear Creek) and Outfall 002 (reuse)

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD) (Influent and Effluent)	Daily	Continuous Recorder
Flow Meter Calibration	Annual	Verification
Quantity of Chlorine Used	Daily	Weight
Eff. Chlorine Residual	Daily	Grab
BOD-5 (Influent and Effluent)	2/week	Composite
TSS (Influent and Effluent)	2/week	Composite
Average Percent Removal (BOD-5 and TSS)	Monthly	Calculation
pH (Influent and Effluent)	3/week	Grab
Fecal Coliform (Effluent)	weekly	Grab

Following Department approval of a Reclaimed Water Reuse Plan, Outfall Number 002 discharges will require at minimum weekly sampling for Total Coliform.

b. Sludge Management

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total solids (% dry wt.), Volatile solids (% dry wt.), Sludge nitrogen: NH3-N; NO3-N; & TKN (% dry wt), Sludge metals content for Pb; Zn; Cu; Ni; & Cd (mg/kg), Phosphorous (% dry wt.), Potassium (% dry wt.), pH (standard units)	Annually if sludge is to be applied	Composite sample to be representative of the product to be land applied
Record of locations where sludge is applied on land (Site location map to be maintained at treatment facility for review upon request by DEQ)	Each Occurrence	Date, volume & locations where sludge is applied recorded on site location map.

Monitoring reports (DMRs) shall include a record of the location, quantity and method of use of all sludge removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

2. Reporting Procedures

Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department by the 15th day of the following month.

All monitoring reports shall indicate the wastewater system classification as shown on page 1 of this permit and include the name of each principal operator designated by the permittee as responsible for supervising the system during the reporting period, and their certificate classification and grade level.

SCHEDULE C

Compliance Schedules and Conditions

1. By no later than 180 days after issuance of this permit, the permittee shall submit a reclaimed water use plan in accordance with Oregon Administrative Rule 340, Division 55, "Regulations Pertaining to the Use of Reclaimed Water (Treated Effluent) from Sewage Treatment Plants". Upon approval of the plan by the Department, the plan shall be implemented by the permittee.

2. By no later than March 1st of each year, the permittee shall submit to the Department an annual report describing the effectiveness of the reclaimed water system to comply with the approved reclaimed water use plan, the rules of Division 55, and the limitations and conditions of this permit applicable to the reuse of reclaimed water.

3. The permittee shall continue to implement a program to identify and reduce inflow and infiltration (I/I). A report summarizing the methods used, the estimated I/I reduction and plans for the year shall be submitted to the Department in May of each year.

4. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he determines good and valid cause resulting from events over which the permittee has little or no control.

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SCHEDULE D

Special Conditions

1. All sludge shall be managed in accordance with a sludge management plan approved by the Department of Environmental Quality. No substantial changes shall be made in sludge management activities which significantly differ from operations specified under the approved plan without the prior written approval of the Department.

2. The permittee shall meet the requirements for use of reclaimed water under Division 55, including the following:

   a. All reclaimed water shall be managed in accordance with the approved Reclaimed Water Use Plan. No substantial changes shall be made in the plan without written approval of the Department.

   b. No Treated effluent shall be released by the permittee to another person, as defined in Oregon Revised Statute (ORS) 468.005, for use unless there is a valid contract between the permittee and that person that meets the requirements of Oregon Administrative Rule (OAR) 340-55-015(9).

   c. The permittee shall notify the Department within 24 hours if it is determined that the treated effluent is being used in a manner not in compliance with OAR 340-55. When the Department offices are not open, the permittee shall report the incident of non-compliance to the Oregon Emergency Response System (Telephone Number 1-800-452-4011).

   d. No recycled water shall be made available to a person proposing to recycle unless the person certifies in writing that they have read and understand the provisions in these rules. This written certification shall be kept on file by the sewage treatment system owner and be made available to the Department for inspection.

3. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

   a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification <u>and</u> grade level (equal to or greater) that corresponds with the classification (collection and /or treatment) of the system to be supervised as specified on page one of this permit.

Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

b. The permittee's wastewater system may not be without supervision (as required by Special Condition [ ] a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower than the system classification.

c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

d. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

e. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation (including shifts). The notice shall be filed with the Water Quality Division, Operator Certification Program (see address on page one). This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

4. The permittee shall notify the DEQ Northwest Region Office (phone: 229-5263) of any malfunction so corrective action can be coordinated between the permittee and the Department.

NPDES GENERAL CONDITIONS

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468.720 and is grounds for enforcement action; for permit termination; suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Violations of Permit Conditions

Oregon Law (ORS 468.990) classifies a willful or negligent violation of the terms of a permit or failure to get a permit as a misdemeanor and a person convicted thereof shall be punishable by a fine of no more than $25,000 or by imprisonment for not more than one year, or by both. Each day of violation constitutes a separate offense.

In addition to the criminal penalties specified above, Oregon Law (ORS 468.140) also allows the Director to impose civil penalties up to $10,000 per day for violation of the terms or conditions of a permit.

3. Duty to Mitigate

The permittee shall take all reasonable steps to minimize or correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application should be submitted at least 180 days before the expiration date of this permit.

The Director may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, suspended, or terminated for cause including, but not limited to, the following:

a. Violation of any terms or conditions of this permit, rule, or statute;

b. Obtaining this permit by misrepresentation or failure to disclose fully all relevant facts; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

The filing of a request by the permittee for a permit modification or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights

The issuance of this permit does not convey any property rights of any sort, or any exclusive privileges, nor does it authorize anyinjury to private property or any invasion of personal rights, nor any violation of federal, state or local laws or regulations.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance includes effective performance, adequate funding, adequate operator staffing and training, and adequate laboratory and process controls, including appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems only when necessary to achieve compliance with the conditions of the permit.

2. Duty to Halt or Reduce Activity

Upon reduction, loss, or failure of the treatment facility, the permittee shall, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means diversion of waste streams from any portion of the conveyance system or treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.

(1) Bypass is prohibited and the Director may take enforcement action against a permittee for bypass, unless:

(a) Bypass was unavoidable to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary pumping, conveyance, or treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if the permittee could have installed adequate backup equipment to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under paragraph c of this section.

(2) The Director may approve an anticipated bypass, after considering its adverse effects, when the Director determines that it will meet the three conditions listed above in paragraph b(1) of this section.

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in Section D, Paragraph D-5 (24-hour notice).

d. Bypass not exceeding limitations.

The permittee may allow any bypass to occur which does not cause effluent limitations to be exceeded, but only if it also is for essential maintenance to assure efficient operation. These bypasses are not subject to the provisions of paragraphs b and c of this section.

4. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters shall be disposed of in such a manner as to prevent any pollutant from such materials from entering public waters, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples shall be taken at the monitoring points specified in this permit and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points shall not be changed without notification to and the approval of the Director.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to insure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected shall be capable of measuring flows with a maximum deviation of less than ± 10% from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate, any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

5. Reporting of Monitoring Results

Monitoring results shall be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports shall be submitted monthly and are to be postmarked by the 14th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR 136 or as specified in this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the DMR. Such increased frequency shall also be indicated.

7. Averaging of Measurements

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean, except for coliform and fecal coliform bacteria which shall be averaged based on a geometric or log mean.

8. Retention of Records

The permittee shall retain records of all monitoring information, including all calibration and maintenance records of all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit, for a period of at least 3 years from the date of the sample, measurement, or report of application. This period may be extended by request of the Director at any time.

9. Records Contents

Records of monitoring information shall include:

a. The date, exact place, time and methods of sampling or measurements;

b. The individual(s) who performed the sampling or measurements;

c. The date(s) analyses were performed;

d. The individual(s) who performed the analyses;

e. The analytical techniques or methods used; and

f. The results of such analyses.

10. Inspection and Entry

The permittee shall allow the Director, or an authorized representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;

b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;

c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes

The permittee shall give notice to the Director as soon as possible of any planned physical alterations or additions to the permitted facility which will result in a change in the character of pollutants to be discharged or which will result in a new or increased discharge of pollutants.

2. Anticipated Noncompliance

The permittee shall give advance notice to the Director of any planned changes in the permitted facility or activity which may result in noncompliance with permit requirements.

3. Transfers

This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit shall be transferred to a third party without prior written approval from the Director. The permittee shall notify the Department when a transfer of property interest takes place.

4. Compliance Schedule

Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit shall be submitted no later than 14 days following each schedule date. Any reports of noncompliance shall include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting

The permittee shall report any noncompliance which may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours from the time the permittee becomes aware of the circumstances. A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. The written submission shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

The following shall be included as information which must be reported within 24 hours:

a. Any unanticipated bypass which exceeds any effluent limitation in the permit.

b. Any upset which exceeds any effluent limitation in the permit.

6. Other Noncompliance

The permittee shall report all instances of noncompliance not reported under Section D, Paragraphs D-4 and D-5, at the time monitoring reports are submitted. The reports shall contain the information listed in Paragraph D-5.

7. Duty to Provide Information

The permittee shall furnish to the Department, within a reasonable time, any information which the Department may request to determine compliance with this permit. The permittee shall also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or any report to the Department, it shall promptly submit such facts or information.

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Reports

State law provides that any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance shall, upon conviction be punished by a fine of not more than $1,000 per violation, or by imprisonment for not more than six months per violation, or by both.

SECTION E. DEFINITIONS AND ACRONYMS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids (non-filterable residue).

3. mg/l means milligrams per liter.

4. kg means kilograms.

5. m^3/d means cubic meters per day.

4. MGD means million gallons per day.

5. Composite sample means a combination of samples collected, generally at equal intervals over a 24-hour period, and apportioned according to the volume of the flow at the time of the sampling.

6. FC means fecal coliform bacteria.

WATER QUALITY ANALYSIS FOR DISCHARGE TO THE MOLALLA RIVER

PURPOSE AND SCOPE

The City of Molalla must address the need to expand the effluent disposal facilities at the wastewater treatment plant (WWTP), particularly for wet weather flows. It is recommended that the City pursue an effluent discharge permit to the Molalla River. This water quality analysis evaluates the water quality impacts of such a discharge on the Molalla River.

This water quality analysis includes a mixing zone analysis to determine the dilution of the effluent in the river within the regulatory mixing zone (RMZ). The mixing zone analysis is used to model the levels of toxic substances, specifically ammonia and chlorine. Other water quality parameters of concern are also evaluated including dissolved oxygen, temperature, bacteria, and pH levels.

RIVER AND EFFLUENT FLOWS

Critical parameters to this water quality analysis, particularly the mixing zone analysis, are the assumed flows in the river and corresponding effluent flows. Typically, worst-case conditions are assumed when river flows are at their lowest and effluent flows are estimated to correspond with the low river flows. The worst-case condition often used for effluent discharge modeling is the seven-day duration, 10-year frequency (7Q10) low flow.

River Flows

The 7Q10 low flows for the Molalla River at the proposed discharge point at approximately River Mile 20 were determined in Chapter 8. For this application, the proposed discharge will only occur during wet weather months; river flows for November through June are shown in Table 2. Average flows are also shown in Table 2 for reference purposes. Flows are shown in million gallons per day (mgd).

Effluent Flows

Effluent flows must be estimated to correspond with the low river flows. It was assumed for this study that the maximum effluent flows during a 7Q10 low flow condition would be the lowest 7-day average flow for the given month recorded from 1996 to 1998. This assumption is conservative since the period from 1996 to 1998 was a relatively wet period and the actual 7Q10 low for influent flows are likely to be lower, thereby resulting in a lower discharge to the river. It was also assumed that there would be no Bear Creek discharge during 7Q10 low flow conditions. This is also a conservative assumption since it is likely that at least a limited discharge would be allowed during the wet weather months of November to June, thereby reducing the discharge to the Molalla River. The resulting projected 20-year (2019) average flows for each month are summarized in Table 1 and are restated in Table 2.

TABLE 1
ASSUMED EFFLUENT LOW FLOWS FOR THE MOLALLA RIVER DISCHARGE

Month	Existing 7-day Low Flow (1) (mgd)	20-yr Projected 7-day Low Flow (2) (mgd)
November	0.74	1.84
December	0.75	1.86
January	1.19	2.95
February	1.10	2.73
March	0.85	2.11
April	0.88	2.18
May	0.49	1.22
June	0.47	1.17

1) Represents lowest 7-day average influent flow based on the WWTP records from 1996 to 1998.
2) Represents the Existing 7-day Low Flow projected to 20-year (2019) 7-day Low Flow using ratio of projected 20-year population of 13,370 and estimated 1998 population of 5,395, or 2.48.

TABLE 2
MOLALLA RIVER AND WWTP EFFLUENT FLOW DATA

Month	Molalla River		WWTP Effluent
	7Q10 Low Flow (1) (mgd)	Average Flow (2) (mgd)	2019 7-day Low (3) (mgd)
November	33	631	1.84
December	113	948	1.86
January	179	1,006	2.95
February	182	793	2.73
March	224	708	2.11
April	208	609	2.18
May	135	459	1.22
June	70	258	1.17

1) From Table 8-1 of this plan.
2) From U.S.G.S. gauging station statistical summary for the Molalla River at Goods Bridge, River Mile 6.0 with 64 percent multiplier for drainage area correction.
3) From Table 1 of this appendix.

MIXING ZONE ANALYSIS

The mixing zone analysis was performed to determine the dilution within the regulatory mixing zone (RMZ) and the resulting levels of toxic substances.

Model Description

The computer model CORMIX was used to approximate the dilution achieved within the mixing zone. Cornell University developed CORMIX for the U.S. Environmental Protection Agency (EPA). CORMIX is an expert system that models the hydrodynamic mixing of a submerged discharge.

The CORMIX model uses several hydraulic parameters in calculating the effluent plume and corresponding dilution that result from a discharge to a receiving stream. The input parameters required include the following:

- Regulatory requirements including the standards for toxic substances and the RMZ.
- Receiving stream data including streamflow, channel characteristics, and temperature and effluent data including flow, temperature, and pollutant concentrations.
- Discharge design including port location, orientation, and size.

Regulatory Requirements

Regulatory requirements include the standards for toxic substances, in this case ammonia and chlorine, and the RMZ.

Standards for Toxic Substances

The toxic substances of concern for this mixing zone analysis are chlorine and ammonia. Standards for chlorine toxicity are included in the Oregon Administrative Rules Chapter 340 (OAR 340) Division 41. These standards are shown in Table 3.

The EPA regulates the concentration of ammonia in receiving streams because of the toxicity of ammonia to fish. Standards for ammonia toxicity are regulated by the EPA and defined in the "Quality Criteria for Water 1986". The EPA limits for ammonia concentrations are defined as acute toxicity, or one-hour average concentrations, and chronic toxicity, or four-day average concentrations.

The concentration limits for ammonia depend on the temperature and pH of the receiving stream and whether or not salmonids are present in the stream. As stream temperature increases and as pH increases, ammonia concentration limits become more stringent. For this application with proposed discharge from November to June, the worst-case condition for these parameters is likely to occur during June when temperatures are relatively high. Table 3 shows the assumptions made with respect to the stream conditions for the development of the ammonia standards to be used in this analysis.

TABLE 3
TOXIC SUBTANCE STANDARDS

Substance	Stream Temperature	Stream pH	Salmonids Present	Standards (mg/l)	
				Acute	Chronic
Chlorine	-	-	-	0.019	0.011
Ammonia	16 degrees C	7.1	YES	21	2.0

Mixing Zone

The RMZ defines the zone within which chronic standards must be met. An RMZ for a given discharge is defined in the National Pollutant Discharge Elimination System (NPDES) wastewater discharge permit issued by the Oregon Department of Environmental Quality (DEQ). A typical RMZ for discharges the size of the proposed discharge is "not to exceed beyond 100 feet downstream from the point of discharge". To provide a conservative analysis, it was assumed that an RMZ of 50 feet would be included in the City's permit, though the City should negotiate with DEQ for the more typical RMZ of 100 feet.

The zone of initial dilution (ZID) is often defined as 10 percent of the RMZ, or 5 feet downstream from the point of discharge based on the assumed permit limit. In relation to the standards for toxic substances described above, the acute limits must be met within the ZID and the chronic limits must be met within the RMZ.

Creek and Effluent Data

Three conditions will be run for the mixing zone analysis. The first condition is the November condition, which represents the lowest flow condition for this application. The second condition is the December condition, which will provide another winter month condition that is still significantly less than the average November river flow (see Table 2). The third condition is the June condition, which will indicate whether or not extending the discharge into the early summer months of May and June is feasible on a water quality basis.

The river and effluent data used for the mixing zone analysis for each of the three conditions and the basis for this data are shown in Table 4. Velocities are shown in feet per second (fps), temperatures are shown in degrees Celsius (deg C), and concentrations are shown in milligrams per liter (mg/l).

TABLE 4
MOLALLA RIVER AND WWTP EFFLUENT DATA FOR MIXING ZONE ANALYSIS

Parameter	November Condition	December Condition	June Condition	Basis for Analysis Data
Molalla River				
Flow	33 mgd	133 mgd	70 mgd	From Table 2.
Water Depth	18 in	24 in	21 in	Estimated based on observation.
Velocity	1.5 fps	2.5 fps	2.0 fps	Estimated based on observation.
Temperature	14 deg C	12 deg C	16 deg C	Maximums from WTP records. (1)
WWTP Effluent				
Flow	1.84 mgd	1.86 mgd	1.17 mgd	From Table 2.
Temperature	18 deg C	15 deg C	19 deg C	Maximums from WWTP records.
Chlorine	0.1 mg/l	0.1 mg/l	0.1 mg/l	Estimated maximum value for downstream of dechlorination chemical and re-aeration facilities.
Ammonia, Intermediate	16.0 mg/l	16.0 mg/l	16.0 mg/l	Actual ammonia measurement of unfiltered effluent in 07/98.
Ammonia, Design	1.0 mg/l	1.0 mg/l	1.0 mg/l	Estimated for recommended design treatment facilities with extended aeration activated sludge process.

1) Based on raw water sampling at the WTP intake located at approximately River Mile 22.

Outfall Dilution Evaluation

Based on the information provided above, and the design parameters discussed below, the outfall design was evaluated for mixing.

Design Parameters

The outfall preliminary design consists of a 24" header of high density polyethylene (HDPE) with branches off the header for each port. There are three 8-inch ports spaced approximately 5 feet apart. The ports are oriented to point downstream parallel to the flow and to each other.

Predicted Dilution Results

For the outfall design, the mixing zone analysis was performed to determine the dilution within the mixing zone for the three different conditions modeled. The CORMIX results for all three conditions are shown in Table 5. The dilution calculated by CORMIX is the dilution along the centerline of the plume and is the most conservative measurement within the plume. The CORMIX input, model results, and graphical output are included in Attachment 1 of this study.

TABLE 5
PREDICTED DILUTION RESULTS FROM CORMIX MODEL

Condition	River Flow (mgd)	Effluent Flow (mgd)	Dilution at ZID	Dilution at RMZ
November	33	1.84	9.6	12.6
December	113	1.86	19.7	27.5
June	70	1.17	18.9	25.9

Chlorine Toxicity

Based on the dilution results from the mixing zone analysis, chlorine levels in the river resulting from the discharge can be evaluated. For the acute chlorine standard of 0.019 mg/l and the maximum chlorine residual in the effluent of 0.1 mg/l, the required dilution at the ZID is 5.3. All low flow conditions modeled meet this dilution. For the chronic chlorine standard of 0.011, the required dilution at the RMZ is 9.1. All low flow conditions modeled meet this dilution. As a result, the predicted dilution results indicate that chlorine standards will be met.

Ammonia Toxicity

Based on the dilution results from the mixing zone analysis, ammonia levels in the river resulting from the discharge can also be evaluated. There are two conditions of ammonia effluent levels, a level of 16.0 mg/l for effluent prior to the installation of the recommended extended aeration activated sludge facilities and a level of 1.0 mg/l for effluent out of the recommended extended aeration activated sludge facilities.

For an effluent ammonia level of 16.0 mg/l, the effluent ammonia concentration is less than the ammonia standard at the ZID and the required dilution at the RMZ is 8.0. All low flow conditions meet this dilution and ammonia standards will therefore be met.

For an effluent ammonia level of 1.0 mg/l, the effluent ammonia concentration is less than the ammonia standards at both the ZID and the RMZ. No dilution is therefore required and the ammonia standards will be met.

DISSOLVED OXYGEN ANALYSIS

Dissolved oxygen (DO) is a critical water quality parameter with respect to the quality of a stream. An effluent discharge into a stream directly impacts the DO levels within the stream and must be evaluated.

Regulatory Requirements

The DEQ regulates DO in receiving streams primarily through OAR 340 Division 41. The regulations pertaining to DO requirements in the Willamette River basin for this application are as follows:

- For waterbodies identified as providing salmonid spawning, during the periods from spawning until fry emergence from the gravels, the DO shall not be less than 11.0 mg/l. Where conditions of barometric pressure, altitude, and temperature preclude attainment of this criteria, DO levels shall not be less than 95 percent of saturation.
- For waterbodies identified as providing cold-water aquatic life, the DO shall not be less than 8.0 mg/l as an absolute minimum. Where conditions of barometric pressure, altitude, and temperature preclude attainment of this criteria, DO shall not be less than 90 percent of saturation.

Review of STORET data on the river indicates that these regulations are not always met even in the river background levels (without the discharge). In meeting with the DEQ regarding requirements specific to the proposed discharge, the DEQ indicated that a requirement that must be met under all conditions is no drop in DO concentrations greater than 0.1 mg/l below background levels.

Analysis

Evaluation of DO reduction cannot be limited to the boundary of the RMZ since the travel time of the flow through the RMZ is on the order of minutes while biochemical oxidation and nitrification processes that reduce DO take place on the order of days. There are several methods that can be used to evaluate the impact of a discharge on a receiving stream with regard to DO. For this analysis, the Streeter-Phelps equation was used, which is a DO sag analysis. The Streeter-Phelps equation assumes a continuous discharge into a river without tributaries.

The river and effluent data used for the DO analysis and the basis for this data are shown in Table 6. Data is presented for each of the three flow conditions evaluated for this study. The DO percent saturation used for this analysis was the minimum recorded in the STORET data from October through June. No STORET data was available for the months of November, December, or June, but October is likely to represent the worst-case for river water quality. Using the minimum value for DO percent saturation ensures that the background levels of DO are not overestimated, though under most conditions the background levels of DO will be greater and the resulting impact of the discharge on the river should be less.

Using this data, the Streeter-Phelps equation was used to determine the time at which the critical (low) DO concentration occurs as a result of the discharge for each flow condition. The critical time, critical distance, and critical DO, as well as the resulting drop in DO, for each condition are also shown in Table 6.

TABLE 6
DISSOLVED OXYGEN ANALYSIS

Parameter	November Data	December Data	June Data	Basis for Analysis Data
Molalla River				
Flow	33 mgd	133 mgd	70 mgd	From Table 2.
Velocity	1.5 fps	2.5 fps	2.0 fps	Estimated based on observation.
Temperature	14 deg C	12 deg C	16 deg C	Maximums from WTP records.
DO % Saturation	95%	95%	95%	Minimum for month of October from STORET data. (1)
DO Conc.	9.72 mg/l	10.16 mg/l	9.30 mg/l	Determined based on temperature and DO % saturation. (2)
BOD	1.3 mg/l	1.3 mg/l	1.3 mg/l	Maximum for month of October from STORET data. (1)
WWTP Effluent				
Flow	1.84 mgd	1.86 mgd	1.17 mgd	From Table 2.
Temperature	18 deg C	15 deg C	19 deg C	Maximums from WWTP records.
DO Conc.	6.0 mg/l	6.0 mg/l	6.0 mg/l	Downstream of reaeration facilities.
BOD	10.0 mg/l	10.0 mg/l	10.0 mg/l	Proposed discharge permit limit.
Analysis Results				
Critical Time	1.0 days	0.55 days	0.93 days	Calculated from Streeter-Phelps equation.
Critical Distance	24.5 miles	22.6 miles	30.6 miles	Calculated from critical time and velocity.
Critical DO	9.48 mg/l	10.08 mg/l	9.21 mg/l	Calculated from Streeter-Phelps equation.
Drop in DO	0.24 mg/l	0.08 mg/l	0.09 mg/l	Must be less than 0.10 mg/l.

1) Based on STORET data provided by DEQ for the Molalla River at River Miles 18.7 and 21.7.
2) Based on Appendix E of *"Wastewater Engineering"*, 3rd Ed., Metcalf and Eddy, 1991.

Results

The use of the Streeter-Phelps equation results in a conservative analysis for on the following reasons:

- Several tributaries enter the creek within the critical distance from the discharge, thereby increasing the flows and dissolved oxygen available for assimilation.
- The proposed discharge is at approximately River Mile 20 and therefore enters the Willamette River beyond the critical distances shown in Table 6. This confluence with a

significantly larger river increases the flows and dissolved oxygen available for assimilation.

- For the first mile just downstream of the proposed discharge, the river is relatively steep (just under a 7 percent slope) and the turbulence resulting from this steep slope is likely to provide aeration in the river beyond what is modeled by the equation. Additionally, the river bottom is largely river rock and cobbles and, during periods of low flow such as those evaluated as part of this study, the shallow water depth running over these rocks further increases turbulence and aeration of the river.

Based on the results of the DO analysis presented in Table 6, the requirement of no more than a 0.1 mg/l drop in DO increase will not be met for the November condition, but will be met for the December and June condition.

TEMPERATURE ANALYSIS

Temperature is a critical water quality parameter with respect to the quality of a stream. An effluent discharge into a stream typically impacts the temperature within the stream and must be evaluated. One of the reasons the DEQ has listed the Molalla River as water quality limited is that the temperatures in the river are high during the summer.

Regulatory Requirements

The DEQ regulates temperature in receiving streams primarily through OAR 340 Division 41. The regulations pertaining to temperature requirements in the Willamette River basin for this application are that no measurable surface water temperature increase is allowed. A measureable increase in temperature is typically considered to be 0.25 deg F, or 0.14 deg C.

Analysis

The river and effluent data used for the temperature analysis and the basis for this data are shown in Table 7. Data is presented for each of the three flow conditions evaluated for this study.

The temperature used for this study is from the historical records at the WWTP for influent temperature. Prior to the year 2000, only the influent temperatures were recorded, but the operators now record effluent temperature as well. Under current conditions, the effluent temperature is somewhat less than the influent temperature due to the long residence time in the relatively deep facultative lagoons. With the recommended improvements to the WWTP, however, future conditions will result in less to no residence time in the lagoons which will also be operated as storage lagoons with water depths that are likely to drop as low as 3 feet. As a result, it is assumed that effluent temperatures will roughly equal influent temperatures during the period of proposed discharge (November to June). There is also the issue of potential heat loss along the approximately five miles of buried forcemain to the river. Calculations to quantify the heat loss are relatively complex and depend on a multitude of assumptions about the ambient soil, groundwater, and air conditions that are difficult to determine and would also vary dramatically under various climatic conditions as well as varying along the length of the forcemain. To be conservative, this analysis assumes no heat loss along the forcemain.

Using the data summarized in Table 7, the temperature at the edge of the RMZ was determined for each flow condition by a mass balance equation based on the dilution at the edge of the RMZ determined in the mixing zone analysis discussed above. The resulting temperature at the edge of the RMZ and corresponding temperature increase for each condition is also shown in Table 7. The calculations for this temperature analysis are included in Attachment 3 of this study.

TABLE 7
TEMPERATURE ANALYSIS

Parameter	November Data	December Data	June Data	Basis for Analysis Data
Molalla River				
Flow	33 mgd	133 mgd	70 mgd	From Table 2.
Temperature	14 deg C	12 deg C	16 deg C	Maximums from WTP records.
WWTP Effluent				
Flow	1.84 mgd	1.86 mgd	1.17 mgd	From Table 2.
Temperature	18 deg C	15 deg C	19 deg C	Maximums from WWTP records.
River and Effluent Mix				
Dilution at edge of RMZ	12.6	27.5	25.9	From mixing zone analysis discussed above.
Temperature at edge of RMZ	14.32 deg C	12.11 deg C	16.12 deg C	From mass balance analysis.
Temperature Increase	0.32 deg C	0.11 deg C	0.12 deg C	Must be less than 0.14 deg C.

Results

Based on the results of the temperature analysis presented in Table 7, the regulatory requirement of no more than an 0.14 deg C increase in temperature will not be met for the November condition, but will be met for the December and June condition. As discussed above, this is a conservative analysis with regard to the temperature of the effluent entering the river; the actual temperature impact is expected to be less than that determined. Once the forcemain is constructed, actual temperatures can be measured and this analysis updated.

OTHER WATER QUALITY PARAMETERS OF CONCERN

As mentioned above, the Molalla River is currently listed by DEQ to be water quality limited. In addition to high temperatures in the summer (temperature impacts were discussed above), the water quality parameters of concern for the Molalla River are high bacteria levels in the fall, winter, and spring. Another water quality parameter that is often evaluated with respect to a surface water discharge is pH. The impacts of the proposed effluent discharge on the river with respect to bacteria and pH are discussed below.

Bacteria Levels

The OAR 340 basin standards for bacteria are the same as what is expected to be on the City's NPDES permit limits for bacteria. Meeting the discharge permit limits will ensure that basin standards are met for bacteria, or that at least the background levels of bacteria will not be worsened by the discharge. As a result, the impact of the proposed discharge on the river with regard to bacteria levels is within regulatory requirements.

pH

The OAR 340 Division 41 requires that pH values not fall outside the range of 6.5 to 8.5. The permit limits require that the effluent pH be within the range of 6.0 and 9.0. The historical WWTP effluent data summarized in Chapter 5 indicates that during the past six years the pH ranged from 6.4 to 7.4. Implementation of the WWTP improvement recommendations will increase nitrification and may subsequently decrease effluent pH to as low as the lower permit limit of 6.0. The pH in the river ranges between approximately 6.8 and 7.1 according to the water treatment plant operator with lower pH readings only after heavy rains.

Assuming the lowest calculated dilution at the edge of the RMZ of 12.6 (for the November conditions) a river pH of 6.8 and an effluent pH of 6.0, the pH at the edge of the mixing zone will be approximately 6.74. (The calculations for this pH analysis are included in Attachment 3 of this study.) As a result, the impact of the discharge on the river with regard to pH is within regulatory requirements.

CONCLUSIONS

Based on this analysis, the proposed discharge to the Molalla River will meet the regulatory requirements with respect to chlorine and ammonia toxicity, dissolved oxygen, temperature, bacteria, and pH during all extreme low flow conditions from December to June, but not during the extreme low flow conditions in November.

For discharge during November, the City would have to determine whether or not flow is adequate for the discharge, but there are no active gauging stations near the proposed discharge and the installation of a permanent gauging station is not feasible because of the environmental damage it could cause. Only low flows would have to be measured, all flows at or above the December 7Q10 low flow would be adequate. During years of dry weather during the fall, the City would have two options. The first option is to hold flows if there is available storage in the storage lagoons. The second option is to perform a river survey to develop a topographic cross-section and velocity gradient to determine flows in the river and to approximate flows at various river depths. Discharge could then occur when it is established that flows exceed the December 7Q10 low flow. The river survey would have to be redone for each dry weather year since the cross-section of the river continues to change.

It is recommended that the City pursue a discharge permit to the Molalla River for discharge from November through June. The discharge in November would be restricted during dry weather years as discussed above. Additionally, May and June discharge would occur only during wet weather when no irrigation occurs on the Coleman Ranch. (Discharge and irrigation will not occur simultaneously due to the requirement to dechlorinate the discharge flows.)

ATTACHMENT 1
CORMIX MODEL INPUT, RESULTS, AND GRAPHICAL OUTPUT

CORMIX DATA SHEET

Site Name	Molalla	Date	02/00
Design Case	NOV 7Q10 LOW; 2019 NOV AVE; 3-9" PORTS	Prepared by	Kay
DOS File Name	MOL03-9		

AMBIENT DATA:

Water body depth	18"	0.46	m		Water body is		(bounded) / unbounded		
Depth at discharge	18"	0.46	m		If bounded: Width	23'	6.9	m	
Ambient flowrate	51cfs	1.44	m^3/s	or:	Appearance		1 / (2) / 3		
Manning's n		0.050		or:	Ambient Velocity	1.5fps	0.46	m/s	
Wind speed		2	m/s		Darcy-Weisbach f		–		

Density Data:

UNITS: Density (kg/ m^3); Temperature (deg C)

Fresh or saltwater?	(F) / SW	If stratified, D/T at surface	–
Density or temperature?	D / (T)	D/T at bottom	–
If uniform, Average D/T	14	Stratification Type	A / B / C
		Pycnocline height	– m
		If C, D/T jump	– m

DISCHARGE DATA:

SUBMERGED SINGLE PORT DISCHARGE – CORMIX 1

Nearest bank is on	left / right		Distance to nearest bank		m
Vertical angle THETA		deg	Horizontal angle SIGMA		deg
Port diameter		m	or: Port area		m^2
Port height		m			

SUBMERGED MULTIPORT DISCHARGE – CORMIX 2

Nearest bank is on	(left) / right						
Diffuser length	12'	3.7	m	Distance to one endpoint	5'	1.6	m
Port height		0.15	m MIN	other endpoint	17'	5.3	m
Total number of ports		3					
Port diameter	8"	0.20	m with:	Contraction ratio		1.0	
Diffuser arrangement:	(unidirectional) / staged / alternating or vertical						
Vertical angle THETA		0	deg	Alignment angle GAMMA		90	deg
Horizontal angle SIGMA		0	deg	Relative orientation BETA		90	deg

BUOYANT SURFACE DISCHARGE – CORMIX 3

Discharge located on	left / right	bank	Configuration:	flush / protruding / co-flowing	
Horizontal angle SIGMA		deg	If protruding, Dist. from bank		m
Depth at discharge		m	Bottom Slope		deg
If rectangular, Width		m	or: If circular, Diameter		m
Depth		m	Bottom invert depth		m

Effluent flow rate	1.84mgd	0.0806	m^3/s	or: Effluent velocity	–	m/s
Effluent density		–	kg/m^3	or: Effluent temperature	18	deg C
Heated discharge?	yes / (no)			If yes, Heat loss coefficient	–	W/m^2
Concentration units	ppm			Effluent concentration	1.0	
Conservative substance?	(yes) / no			If no, Decay coefficient	–	/day

MIXING ZONE DATA:

If effluent toxic?	(yes) / no	If yes,	CMC value		22	
			CCC value		2.2	
WQ std for conv. pollutant?	yes / (no)	If yes,	value of standard		–	
Mixing zone specified?	(yes) / no	If yes,	distance	50'	15.2	m
			or width		–	% or m
Region of Interest	200 m		or area		–	% or m^2
Grid intervals for display	50					

CORMIX DATA SHEET

Site Name	Molalla	Date	02/00
Design Case	DEC 7Q10 low; 2019 Dec Avg; 3-9" Ports	Prepared by	Kay
DOS File Name	MODE3-9		

AMBIENT DATA:

Water body depth	24"	0.61	m		Water body is		(bounded) / unbounded	
Depth at discharge	24"	0.61	m		If bounded: Width	35'	10.7	m
Ambient flowrate	175cfs	4.96	m^3/s	or:	Appearance		1 / (2) / 3	
Manning's n		0.050		or:	Ambient Velocity	2.5fps	0.76	m/s
Wind speed		2	m/s		Darcy-Weisbach f		-	

Density Data:

UNITS: Density (kg/ m^3); Temperature (deg C)

Fresh or saltwater?	(F) / SW	If stratified, D/T at surface	-
Density or temperature?	D / (T)	D/T at bottom	-
If uniform, Average D/T	12	Stratification Type	A / B / C
		Pycnocline height	- m
		If C, D/T jump	- m

DISCHARGE DATA:

SUBMERGED SINGLE PORT DISCHARGE – CORMIX 1

Nearest bank is on	left / right			Distance to nearest bank		m
Vertical angle THETA		deg		Horizontal angle SIGMA		deg
Port diameter		m	or:	Port area		m^2
Port height		m				

SUBMERGED MULTIPORT DISCHARGE – CORMIX 2

Nearest bank is on	(left) / right						
Diffuser length	12'	3.7	m	Distance to one endpoint	8'	2.4	m
Port height		0.15	m	other endpoint	20'	6.1	m
Total number of ports		3					
Port diameter	6"	0.20	m	with: Contraction ratio		1.0	
Diffuser arrangement:	(unidirectional) / staged / alternating or vertical						
Vertical angle THETA		0	deg	Alignment angle GAMMA		90	deg
Horizontal angle SIGMA		0	deg	Relative orientation BETA		90	deg

BUOYANT SURFACE DISCHARGE – CORMIX 3

Discharge located on	left / right	bank		Configuration:	flush / protruding / co-flowing	
Horizontal angle SIGMA		deg		If protruding, Dist. from bank		m
Depth at discharge		m		Bottom Slope		deg
If rectangular, Width		m	or:	If circular, Diameter		m
Depth		m		Bottom invert depth		m

Effluent flow rate	1.86mgd	0.0815	m^3/s	or:	Effluent velocity	-	m/s
Effluent density		-	kg/m^3	or:	Effluent temperature	15	deg C
Heated discharge?	yes / (no)				If yes, Heat loss coefficient	-	W/m^2
Concentration units	PPM				Effluent concentration	1.0	
Conservative substance?	(yes) / no				If no, Decay coefficient	-	/day

MIXING ZONE DATA:

If effluent toxic?	(yes) / no	If yes,	CMC value		22	
			CCC value		2.2	
WQ std for conv. pollutant?	yes / (no)	If yes,	value of standard		-	
Mixing zone specified?	(yes) / no	If yes,	distance	50'	15.2	m
			or width		-	% or m
Region of Interest	200 m		or area		-	% or m^2
Grid intervals for display	50					

CORMIX DATA SHEET

Site Name	Molalla	Date	02/00
Design Case	JUN 7Q10 LOW; 2019 JUNE AVG; 3-9" PORTS	Prepared by	Kay
DOS File Name	MOJU3-9		

AMBIENT DATA:

Water body depth	21"	0.53	m		Water body is		(bounded)/ unbounded		
Depth at discharge	21"	0.53	m		If bounded: Width	31'	9.5	m	
Ambient flowrate	109 cfs	3.09	m^3/s	or:	Appearance		1 /(2)/ 3		
Manning's n		0.050		or:	Ambient Velocity	2.0 fps	0.61	m/s	
Wind speed		2	m/s		Darcy-Weisbach f		–		

Density Data:

UNITS: Density (kg/ m^3); Temperature (deg C)

Fresh or saltwater?	(F)/ SW	If stratified, D/T at surface	–	
Density or temperature?	D /(T)	D/T at bottom	–	
If uniform, Average D/T	16	Stratification Type	A / B / C	
		Pycnocline height	–	m
		If C, D/T jump	–	m

DISCHARGE DATA:

SUBMERGED SINGLE PORT DISCHARGE – CORMIX 1

Nearest bank is on	left / right		Distance to nearest bank			m
Vertical angle THETA		deg	Horizontal angle SIGMA			deg
Port diameter		m	or: Port area			m^2
Port height		m				

SUBMERGED MULTIPORT DISCHARGE – CORMIX 2

Nearest bank is on	(left)/ right						
Diffuser length	12'	3.7	m	Distance to one endpoint	7'	2.1	m
Port height		0.15	m	other endpoint	19'	5.8	m
Total number of ports		3					
Port diameter	8"	0.20	m	with: Contraction ratio		1.0	
Diffuser arrangement:	(unidirectional)/ staged / alternating or vertical						
Vertical angle THETA		0	deg	Alignment angle GAMMA		90	deg
Horizontal angle SIGMA		0	deg	Relative orientation BETA		90	deg

BUOYANT SURFACE DISCHARGE – CORMIX 3

Discharge located on	left / right	bank	Configuration:	flush / protruding / co-flowing	
Horizontal angle SIGMA		deg	If protruding, Dist. from bank		m
Depth at discharge		m	Bottom Slope		deg
If rectangular, Width		m	or: If circular, Diameter		m
Depth		m	Bottom invert depth		m

Effluent flow rate	1.35 mgd	0.0593 MIN	m^3/s	or: Effluent velocity	–	m/s
Effluent density		–	kg/m^3	or: Effluent temperature	19	deg C
Heated discharge?	yes /(no)			If yes, Heat loss coefficient	–	W/m^2
Concentration units	ppm			Effluent concentration	1.0	
Conservative substance?	(yes)/ no			If no, Decay coefficient	–	/day

MIXING ZONE DATA:

If effluent toxic?	(yes)/ no	If yes,	CMC value	22	
			CCC value	2.2	
WQ std for conv. pollutant?	yes /(no)	If yes,	value of standard	–	
Mixing zone specified?	(yes)/ no	If yes,	distance 50'	15.2	m
			or width	–	% or m
Region of Interest	200 m		or area	–	% or m^2
Grid intervals for display	50				

```
CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                    CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX2:                                          Subsystem version:
 Submerged Multiport Diffuser Discharges    CORMIX-GI Version 4.03b
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          Molalla
 Design case:              Nov 7Q10 Low; 2019 Nov 7-day Low; 3-8" Ports
 FILE NAME:                C:\Program Files\CORMIX-GI\My Files\molo3-8.prd
 Time of Fortran run:      02/04/2000--09:05:28

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =       6.90  AS     =       3.17  QA     =       1.44  ICHREG= 2
 HA     =        .46  HD     =        .46
 UA     =        .454 F      =        .254 USTAR = .8084E-01
 UW     =      2.000 UWSTAR= .2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  999.2460

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       3.45  YB1    =       1.60  YB2    =       5.30
 LD     =       3.70 NOPEN =    3        SPAC   =       1.85
 D0     =        .200 A0     =        .031 H0     =        .15
 Nozzle/port arrangement:   unidirectional_without_fanning
 GAMMA =      90.00  THETA =        .00  SIGMA =        .00  BETA   =      90.00
 U0     =        .855 Q0     =        .081        = .8060E-01
 RHO0   =  998.5967  DRHO0 = .6494E+00  GP0    = .6373E-02
 C0     = .1000E+01  CUNITS=  mg/l
 IPOLL =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     = .2178E-01  m0     = .1863E-01  j0     = .1388E-03  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =        .025 lM     =       6.94  lm     =        .09
 lmp    =  99999.00  lbp    =  99999.00  la     =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     = .8060E-01  M0     = .6893E-01  J0     = .5136E-03
 Associated 3-d length scales (meters)
 LQ     =        .31  LM     =       5.94  Lm     =        .58  Lb     =        .01
                                            Lmp    =  99999.00  Lbp    =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =      67.12  FRD0   =      23.96  R      =       1.88
 (slot)               (port/nozzle)

FLOW CLASSIFICATION
 22222222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =       .46  2
 22222222222222222222222222222222222222222

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+01  CUNITS=  mg/l
 NTOX   =  1         CMC    = .2200E+02  CCC    =  CSTD
 NSTD   =  1         CSTD   = .2200E+01
```

```
 REGMZ =  1
 REGSPC=  1          XREG  =     15.20  WREG  =        .00  AREG  =        .00
 XINT  =    200.00  XMAX  =    200.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         3.45 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP = 50 display intervals per module
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X         Y         Z         S        C        BV        BH
      .00       .00       .15       1.0  .100E+01     .02      1.85
** CMC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below CMC value of  .220E+02
   due to mixing in this control volume.
 The actual extent of the TOXIC DILUTION ZONE will be smaller than control
   volume outflow values predicted below.

** WATER QUALITY STANDARD OR CCC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below water quality standard
   or CCC value of  .220E+01 due to mixing in this control volume.
 The actual extent of the zone at whose boundary the water quality
   standard or the CCC is exceeded will be smaller than the control
   volume outflow values predicted below.


END OF MOD201: DIFFUSER DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =      .46m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X         Y         Z         S        C        BV        BH
      .00       .00       .15       1.0  .100E+01     .02      1.85
      .04       .00       .15       2.3  .428E+00     .03      1.84
      .07       .00       .15       2.9  .346E+00     .04      1.83
      .11       .00       .15       3.3  .302E+00     .04      1.83
      .15       .00       .16       3.7  .272E+00     .05      1.82
      .19       .00       .16       4.0  .251E+00     .06      1.81
```

.22	.00	.16	4.3	.234E+00	.06	1.81
.26	.00	.16	4.5	.221E+00	.07	1.80
.30	.00	.16	4.8	.209E+00	.08	1.80
.33	.00	.16	5.0	.200E+00	.09	1.79
.37	.00	.17	5.2	.191E+00	.09	1.79
.41	.00	.17	5.4	.184E+00	.10	1.78
.44	.00	.17	5.6	.178E+00	.11	1.78
.48	.00	.17	5.8	.172E+00	.12	1.77
.52	.00	.17	6.0	.167E+00	.13	1.77
.56	.00	.17	6.2	.162E+00	.14	1.76
.59	.00	.18	6.3	.158E+00	.15	1.76
.63	.00	.18	6.5	.154E+00	.16	1.76
.67	.00	.18	6.7	.150E+00	.17	1.75
.70	.00	.18	6.8	.147E+00	.17	1.75
.74	.00	.18	7.0	.143E+00	.18	1.75
.78	.00	.18	7.1	.140E+00	.19	1.75
.81	.00	.19	7.3	.138E+00	.20	1.74
.85	.00	.19	7.4	.135E+00	.21	1.74
.89	.00	.19	7.5	.133E+00	.22	1.74
.93	.00	.19	7.7	.130E+00	.23	1.73
.96	.00	.19	7.8	.128E+00	.24	1.73
1.00	.00	.19	7.9	.126E+00	.25	1.73
1.04	.00	.19	8.1	.124E+00	.26	1.73
1.07	.00	.20	8.2	.122E+00	.27	1.73
1.11	.00	.20	8.3	.120E+00	.28	1.72
1.15	.00	.20	8.4	.119E+00	.29	1.72
1.18	.00	.20	8.6	.117E+00	.29	1.72
1.22	.00	.20	8.7	.115E+00	.30	1.72
1.26	.00	.20	8.8	.114E+00	.31	1.72
1.29	.00	.21	8.9	.112E+00	.32	1.72
1.33	.00	.21	9.0	.111E+00	.33	1.72
1.37	.00	.21	9.1	.110E+00	.34	1.71
1.41	.00	.21	9.2	.108E+00	.35	1.71
1.44	.00	.21	9.3	.107E+00	.36	1.71
1.48	.00	.21	9.4	.106E+00	.37	1.71
1.52	.00	.22	9.6	.105E+00	.38	1.71
1.55	.00	.22	9.7	.104E+00	.39	1.71
1.59	.00	.22	9.8	.102E+00	.40	1.71
1.63	.00	.22	9.9	.101E+00	.40	1.71
1.66	.00	.22	10.0	.100E+00	.41	1.71
1.70	.00	.22	10.1	.994E-01	.42	1.71
1.74	.00	.23	10.2	.985E-01	.43	1.71
1.78	.00	.23	10.3	.975E-01	.44	1.71
1.81	.00	.23	10.3	.966E-01	.45	1.71
1.85	.00	.23	10.4	.957E-01	.46	1.71

Cumulative travel time = 3. sec

Plume centerline may exhibit slight discontinuities in transition to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the entire layer depth.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

The initial plume WIDTH values in the next far-field module will be CORRECTED by a factor 1.12 to conserve the mass flux in the far-field!

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .744E-02 m^2/s
Horizontal diffusivity (initial value) = .186E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width, measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
1.85	.00	.46	10.4	.957E-01	.46	1.91	.46	.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

3.20	.00	.46	10.7	.936E-01	.46	1.95	.46	.00
4.56	.00	.46	10.9	.917E-01	.46	1.99	.46	.00
5.91	.00	.46	11.1	.898E-01	.46	2.04	.46	.00
7.26	.00	.46	11.4	.881E-01	.46	2.08	.46	.00
8.62	.00	.46	11.6	.865E-01	.46	2.12	.46	.00
9.97	.00	.46	11.8	.849E-01	.46	2.15	.46	.00
11.33	.00	.46	12.0	.834E-01	.46	2.19	.46	.00
12.68	.00	.46	12.2	.820E-01	.46	2.23	.46	.00
14.03	.00	.46	12.4	.807E-01	.46	2.27	.46	.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds the regulatory value = 15.20 m.
This is the extent of the REGULATORY MIXING ZONE.

15.39	.00	.46	12.6	.794E-01	.46	2.30	.46	.00
16.74	.00	.46	12.8	.782E-01	.46	2.34	.46	.00
18.09	.00	.46	13.0	.771E-01	.46	2.37	.46	.00
19.45	.00	.46	13.2	.760E-01	.46	2.41	.46	.00
20.80	.00	.46	13.3	.749E-01	.46	2.44	.46	.00
22.15	.00	.46	13.5	.739E-01	.46	2.47	.46	.00
23.51	.00	.46	13.7	.729E-01	.46	2.51	.46	.00

24.86	.00	.46	13.9	.720E-01	.46	2.54	.46	.00
26.21	.00	.46	14.1	.711E-01	.46	2.57	.46	.00
27.57	.00	.46	14.2	.702E-01	.46	2.60	.46	.00
28.92	.00	.46	14.4	.694E-01	.46	2.64	.46	.00
30.28	.00	.46	14.6	.686E-01	.46	2.67	.46	.00
31.63	.00	.46	14.8	.678E-01	.46	2.70	.46	.00
32.98	.00	.46	14.9	.670E-01	.46	2.73	.46	.00
34.34	.00	.46	15.1	.663E-01	.46	2.76	.46	.00
35.69	.00	.46	15.2	.656E-01	.46	2.79	.46	.00
37.04	.00	.46	15.4	.649E-01	.46	2.82	.46	.00
38.40	.00	.46	15.6	.642E-01	.46	2.85	.46	.00
39.75	.00	.46	15.7	.636E-01	.46	2.88	.46	.00
41.10	.00	.46	15.9	.630E-01	.46	2.90	.46	.00
42.46	.00	.46	16.0	.624E-01	.46	2.93	.46	.00
43.81	.00	.46	16.2	.618E-01	.46	2.96	.46	.00
45.17	.00	.46	16.3	.612E-01	.46	2.99	.46	.00
46.52	.00	.46	16.5	.606E-01	.46	3.02	.46	.00
47.87	.00	.46	16.6	.601E-01	.46	3.04	.46	.00
49.23	.00	.46	16.8	.596E-01	.46	3.07	.46	.00
50.58	.00	.46	16.9	.590E-01	.46	3.10	.46	.00
51.93	.00	.46	17.1	.585E-01	.46	3.12	.46	.00
53.29	.00	.46	17.2	.581E-01	.46	3.15	.46	.00
54.64	.00	.46	17.4	.576E-01	.46	3.18	.46	.00
55.99	.00	.46	17.5	.571E-01	.46	3.20	.46	.00
57.35	.00	.46	17.6	.567E-01	.46	3.23	.46	.00
58.70	.00	.46	17.8	.562E-01	.46	3.25	.46	.00
60.05	.00	.46	17.9	.558E-01	.46	3.28	.46	.00
61.41	.00	.46	18.1	.554E-01	.46	3.30	.46	.00
62.76	.00	.46	18.2	.549E-01	.46	3.33	.46	.00
64.12	.00	.46	18.3	.545E-01	.46	3.35	.46	.00
65.47	.00	.46	18.5	.541E-01	.46	3.38	.46	.00
66.82	.00	.46	18.6	.538E-01	.46	3.40	.46	.00
68.18	.00	.46	18.7	.534E-01	.46	3.43	.46	.00
69.53	.00	.46	18.9	.530E-01	.46	3.45	.46	.00

Cumulative travel time = 145. sec

Surface plume centerline coincides with the mid-width of bounded channel.
Thus, the plume interacts SIMULTANEOUSLY WITH BOTH BANKS.

Plume Stage 2 (bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
69.53	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
72.14	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00

Effluent is FULLY MIXED over the entire channel cross-section.
Except for possible far-field decay or reaction processes, there are
NO FURTHER CHANGES with downstream direction.

74.75	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
77.36	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
79.97	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
82.58	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
85.19	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
87.80	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
90.41	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
93.01	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
95.62	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
98.23	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
100.84	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
103.45	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
106.06	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00

108.67	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
111.28	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
113.89	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
116.50	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
119.11	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
121.72	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
124.33	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
126.94	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
129.55	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
132.16	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
134.77	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
137.37	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
139.98	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
142.59	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
145.20	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
147.81	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
150.42	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
153.03	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
155.64	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
158.25	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
160.86	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
163.47	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
166.08	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
168.69	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
171.30	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
173.91	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
176.52	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
179.12	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
181.73	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
184.34	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
186.95	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
189.56	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
192.17	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
194.78	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
197.39	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00
200.00	3.45	.46	18.9	.530E-01	.46	6.90	.46	.00

Cumulative travel time = 417. sec

Simulation limit based on maximum specified distance = 200.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Because of the fairly LARGE SPACING between adjacent risers/nozzles/ports, the above results may be unreliable in the immediate near-field of the diffuser.

A SUBSEQUENT APPLICATION OF CORMIX1 IS RECOMMENDED to provide more detail for one of the individual jets/plumes in the initial region before merging.

CORMIX2: Submerged Multiport Diffuser Discharges End of Prediction File

2222222222222222222222222222222222222222222222222222222222222222222222222222

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CORMIX2 PREDICTION FILE:
2222222222222222222222222222222222222222222222222222222222222222222222222222
                    CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX2:                                          Subsystem version:
 Submerged Multiport Diffuser Discharges    CORMIX-GI Version 4.03b
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          Molalla
 Design case:              Dec 7Q10 Low; 2019 Dec Ave Flow; 3-8" Ports
 FILE NAME:                C:\Program Files\CORMIX-GI\My Files\mode3-8.prd
 Time of Fortran run:      02/04/2000--08:54:02

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      10.70  AS     =       6.53  QA     =       4.96  ICHREG= 2
 HA     =        .61  HD     =        .61
 UA     =        .760 F      =        .231 USTAR = .1292E+00
 UW     =       2.000 UWSTAR= .2198E-02
 Uniform density environment
 STRCND=  U          RHOAM =  999.4994

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:      DITYPE= unidirectional_perpendicular
 BANK   =  LEFT      DISTB =       4.25  YB1    =       2.40  YB2    =       6.10
 LD     =       3.70 NOPEN =    3        SPAC   =       1.85
 D0     =        .200 A0     =        .031 H0     =        .15
 Nozzle/port arrangement:   unidirectional_without_fanning
 GAMMA =      90.00  THETA =        .00  SIGMA =        .00  BETA   =      90.00
 U0     =        .865 Q0     =        .082        = .8150E-01
 RHO0   =  999.1011  DRHO0 = .3983E+00  GP0    = .3908E-02
 C0     = .1000E+01  CUNITS=  mg/l
 IPOLL =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     = .2203E-01  m0     = .1905E-01  j0     = .8608E-04  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =        .025 lM     =       9.75  lm     =        .03
 lmp    =  99999.00  lbp    =  99999.00  la     =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     = .8150E-01  M0     = .7048E-01  J0     = .3185E-03
 Associated 3-d length scales (meters)
 LQ     =        .31  LM     =       7.66  Lm     =        .35  Lb     =        .00
                                           Lmp    =  99999.00  Lbp    =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =      86.67  FRD0   =      30.93  R      =       1.14
 (slot)               (port/nozzle)

FLOW CLASSIFICATION
 2222222222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =       .61  2
 2222222222222222222222222222222222222222222

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+01  CUNITS=  mg/l
 NTOX   =  1         CMC    = .2200E+02  CCC    =  CSTD
 NSTD   =  1         CSTD   = .2200E+01
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 REGMZ =  1
 REGSPC=  1          XREG  =     15.20  WREG  =        .00  AREG  =        .00
 XINT  =    200.00  XMAX  =    200.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         4.25 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
      .00      .00      .15      1.0  .100E+01     .02     1.85
** CMC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below CMC value of  .220E+02
   due to mixing in this control volume.
 The actual extent of the TOXIC DILUTION ZONE will be smaller than control
   volume outflow values predicted below.

** WATER QUALITY STANDARD OR CCC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below water quality standard
   or CCC value of  .220E+01 due to mixing in this control volume.
 The actual extent of the zone at whose boundary the water quality
   standard or the CCC is exceeded will be smaller than the control
   volume outflow values predicted below.


END OF MOD201: DIFFUSER DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =      .61m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
      .00      .00      .15      1.0  .100E+01     .02     1.85
      .04      .00      .15      3.9  .256E+00     .03     1.85
      .07      .00      .16      5.1  .195E+00     .04     1.84
      .11      .00      .16      6.0  .165E+00     .04     1.84
      .15      .00      .16      6.8  .146E+00     .05     1.84
      .19      .00      .17      7.5  .133E+00     .06     1.84
```

.22	.00	.17	8.1	.123E+00	.07	1.84
.26	.00	.17	8.7	.115E+00	.09	1.83
.30	.00	.17	9.2	.108E+00	.10	1.83
.33	.00	.18	9.7	.103E+00	.11	1.83
.37	.00	.18	10.2	.979E-01	.12	1.83
.41	.00	.18	10.7	.938E-01	.13	1.83
.44	.00	.19	11.1	.902E-01	.15	1.83
.48	.00	.19	11.5	.869E-01	.16	1.82
.52	.00	.19	11.9	.840E-01	.17	1.82
.56	.00	.20	12.3	.814E-01	.18	1.82
.59	.00	.20	12.7	.790E-01	.20	1.82
.63	.00	.20	13.0	.769E-01	.21	1.82
.67	.00	.21	13.4	.749E-01	.22	1.82
.70	.00	.21	13.7	.730E-01	.23	1.82
.74	.00	.21	14.0	.713E-01	.24	1.82
.78	.00	.22	14.3	.697E-01	.26	1.82
.81	.00	.22	14.7	.682E-01	.27	1.81
.85	.00	.22	15.0	.668E-01	.28	1.81
.89	.00	.22	15.3	.655E-01	.29	1.81
.93	.00	.23	15.6	.642E-01	.31	1.81
.96	.00	.23	15.9	.631E-01	.32	1.81
1.00	.00	.23	16.1	.620E-01	.33	1.81
1.04	.00	.24	16.4	.609E-01	.34	1.81
1.07	.00	.24	16.7	.599E-01	.35	1.81
1.11	.00	.24	17.0	.590E-01	.37	1.81
1.15	.00	.25	17.2	.581E-01	.38	1.81
1.18	.00	.25	17.5	.572E-01	.39	1.81
1.22	.00	.25	17.7	.564E-01	.40	1.81
1.26	.00	.26	18.0	.556E-01	.41	1.81
1.29	.00	.26	18.2	.548E-01	.43	1.81
1.33	.00	.26	18.5	.541E-01	.44	1.81
1.37	.00	.26	18.7	.534E-01	.45	1.81
1.41	.00	.27	19.0	.527E-01	.46	1.81
1.44	.00	.27	19.2	.521E-01	.48	1.81
1.48	.00	.27	19.4	.515E-01	.49	1.80
1.52	.00	.28	19.7	.509E-01	.50	1.80
1.55	.00	.28	19.9	.503E-01	.51	1.80
1.59	.00	.28	20.1	.497E-01	.52	1.80
1.63	.00	.29	20.3	.492E-01	.54	1.80
1.66	.00	.29	20.5	.487E-01	.55	1.80
1.70	.00	.29	20.8	.482E-01	.56	1.80
1.74	.00	.30	21.0	.477E-01	.57	1.80
1.78	.00	.30	21.2	.472E-01	.59	1.80
1.81	.00	.30	21.4	.467E-01	.60	1.80
1.85	.00	.30	21.6	.463E-01	.61	1.80

Cumulative travel time = 2. sec

Plume centerline may exhibit slight discontinuities in transition to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the entire layer depth.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .158E-01 m^2/s
Horizontal diffusivity (initial value) = .394E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
1.85	.00	.61	21.6	.463E-01	.61	1.87	.61	.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

3.67	.00	.61	22.5	.445E-01	.61	1.94	.61	.00
5.49	.00	.61	23.3	.429E-01	.61	2.02	.61	.00
7.30	.00	.61	24.2	.414E-01	.61	2.09	.61	.00
9.12	.00	.61	24.9	.401E-01	.61	2.16	.61	.00
10.94	.00	.61	25.7	.389E-01	.61	2.22	.61	.00
12.76	.00	.61	26.5	.378E-01	.61	2.29	.61	.00
14.58	.00	.61	27.2	.368E-01	.61	2.35	.61	.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds the regulatory value = 15.20 m.
This is the extent of the REGULATORY MIXING ZONE.

16.40	.00	.61	27.9	.358E-01	.61	2.41	.61	.00
18.21	.00	.61	28.6	.350E-01	.61	2.47	.61	.00
20.03	.00	.61	29.3	.342E-01	.61	2.53	.61	.00
21.85	.00	.61	29.9	.334E-01	.61	2.59	.61	.00
23.67	.00	.61	30.6	.327E-01	.61	2.64	.61	.00
25.49	.00	.61	31.2	.321E-01	.61	2.70	.61	.00
27.30	.00	.61	31.8	.314E-01	.61	2.75	.61	.00
29.12	.00	.61	32.4	.308E-01	.61	2.80	.61	.00
30.94	.00	.61	33.0	.303E-01	.61	2.86	.61	.00
32.76	.00	.61	33.6	.298E-01	.61	2.91	.61	.00
34.58	.00	.61	34.2	.293E-01	.61	2.96	.61	.00
36.40	.00	.61	34.7	.288E-01	.61	3.00	.61	.00

38.21	.00	.61	35.3	.283E-01	.61	3.05	.61	.00
40.03	.00	.61	35.8	.279E-01	.61	3.10	.61	.00
41.85	.00	.61	36.4	.275E-01	.61	3.15	.61	.00
43.67	.00	.61	36.9	.271E-01	.61	3.19	.61	.00
45.49	.00	.61	37.4	.267E-01	.61	3.24	.61	.00
47.31	.00	.61	38.0	.263E-01	.61	3.28	.61	.00
49.12	.00	.61	38.5	.260E-01	.61	3.33	.61	.00
50.94	.00	.61	39.0	.257E-01	.61	3.37	.61	.00
52.76	.00	.61	39.5	.253E-01	.61	3.41	.61	.00
54.58	.00	.61	40.0	.250E-01	.61	3.46	.61	.00
56.40	.00	.61	40.4	.247E-01	.61	3.50	.61	.00
58.21	.00	.61	40.9	.244E-01	.61	3.54	.61	.00
60.03	.00	.61	41.4	.242E-01	.61	3.58	.61	.00
61.85	.00	.61	41.9	.239E-01	.61	3.62	.61	.00
63.67	.00	.61	42.3	.236E-01	.61	3.66	.61	.00
65.49	.00	.61	42.8	.234E-01	.61	3.70	.61	.00
67.31	.00	.61	43.2	.231E-01	.61	3.74	.61	.00
69.12	.00	.61	43.7	.229E-01	.61	3.78	.61	.00
70.94	.00	.61	44.1	.227E-01	.61	3.82	.61	.00
72.76	.00	.61	44.6	.224E-01	.61	3.85	.61	.00
74.58	.00	.61	45.0	.222E-01	.61	3.89	.61	.00
76.40	.00	.61	45.4	.220E-01	.61	3.93	.61	.00
78.21	.00	.61	45.9	.218E-01	.61	3.97	.61	.00
80.03	.00	.61	46.3	.216E-01	.61	4.00	.61	.00
81.85	.00	.61	46.7	.214E-01	.61	4.04	.61	.00
83.67	.00	.61	47.1	.212E-01	.61	4.07	.61	.00
85.49	.00	.61	47.5	.210E-01	.61	4.11	.61	.00
87.31	.00	.61	47.9	.209E-01	.61	4.15	.61	.00
89.12	.00	.61	48.3	.207E-01	.61	4.18	.61	.00
90.94	.00	.61	48.7	.205E-01	.61	4.22	.61	.00
92.76	.00	.61	49.1	.203E-01	.61	4.25	.61	.00

Cumulative travel time = 120. sec

Plume Stage 2 (bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
92.76	4.25	.61	49.1	.203E-01	.61	8.50	.61	.00
94.91	4.25	.61	49.3	.203E-01	.61	8.52	.61	.00
97.05	4.25	.61	49.4	.203E-01	.61	8.54	.61	.00
99.19	4.25	.61	49.5	.202E-01	.61	8.56	.61	.00
101.34	4.25	.61	49.6	.202E-01	.61	8.58	.61	.00
103.48	4.25	.61	49.7	.201E-01	.61	8.60	.61	.00
105.63	4.25	.61	49.8	.201E-01	.61	8.62	.61	.00
107.77	4.25	.61	50.0	.200E-01	.61	8.64	.61	.00
109.92	4.25	.61	50.1	.200E-01	.61	8.66	.61	.00
112.06	4.25	.61	50.2	.199E-01	.61	8.68	.61	.00
114.21	4.25	.61	50.3	.199E-01	.61	8.70	.61	.00
116.35	4.25	.61	50.4	.198E-01	.61	8.72	.61	.00
118.50	4.25	.61	50.5	.198E-01	.61	8.74	.61	.00
120.64	4.25	.61	50.6	.197E-01	.61	8.76	.61	.00
122.79	4.25	.61	50.8	.197E-01	.61	8.78	.61	.00
124.93	4.25	.61	50.9	.197E-01	.61	8.80	.61	.00
127.08	4.25	.61	51.0	.196E-01	.61	8.82	.61	.00
129.22	4.25	.61	51.1	.196E-01	.61	8.84	.61	.00
131.37	4.25	.61	51.2	.195E-01	.61	8.86	.61	.00
133.51	4.25	.61	51.3	.195E-01	.61	8.88	.61	.00
135.66	4.25	.61	51.4	.194E-01	.61	8.90	.61	.00
137.80	4.25	.61	51.5	.194E-01	.61	8.91	.61	.00
139.95	4.25	.61	51.6	.194E-01	.61	8.93	.61	.00
142.09	4.25	.61	51.8	.193E-01	.61	8.95	.61	.00

144.24	4.25	.61	51.9	.193E-01	.61	8.97	.61	.00
146.38	4.25	.61	52.0	.192E-01	.61	8.99	.61	.00
148.52	4.25	.61	52.1	.192E-01	.61	9.01	.61	.00
150.67	4.25	.61	52.2	.192E-01	.61	9.03	.61	.00
152.81	4.25	.61	52.3	.191E-01	.61	9.05	.61	.00
154.96	4.25	.61	52.4	.191E-01	.61	9.07	.61	.00
157.10	4.25	.61	52.5	.190E-01	.61	9.09	.61	.00
159.25	4.25	.61	52.6	.190E-01	.61	9.11	.61	.00
161.39	4.25	.61	52.7	.190E-01	.61	9.12	.61	.00
163.54	4.25	.61	52.9	.189E-01	.61	9.14	.61	.00
165.68	4.25	.61	53.0	.189E-01	.61	9.16	.61	.00
167.83	4.25	.61	53.1	.188E-01	.61	9.18	.61	.00
169.97	4.25	.61	53.2	.188E-01	.61	9.20	.61	.00
172.12	4.25	.61	53.3	.188E-01	.61	9.22	.61	.00
174.26	4.25	.61	53.4	.187E-01	.61	9.24	.61	.00
176.41	4.25	.61	53.5	.187E-01	.61	9.26	.61	.00
178.55	4.25	.61	53.6	.187E-01	.61	9.27	.61	.00
180.70	4.25	.61	53.7	.186E-01	.61	9.29	.61	.00
182.84	4.25	.61	53.8	.186E-01	.61	9.31	.61	.00
184.99	4.25	.61	53.9	.185E-01	.61	9.33	.61	.00
187.13	4.25	.61	54.0	.185E-01	.61	9.35	.61	.00
189.28	4.25	.61	54.1	.185E-01	.61	9.37	.61	.00
191.42	4.25	.61	54.3	.184E-01	.61	9.38	.61	.00
193.57	4.25	.61	54.4	.184E-01	.61	9.40	.61	.00
195.71	4.25	.61	54.5	.184E-01	.61	9.42	.61	.00
197.86	4.25	.61	54.6	.183E-01	.61	9.44	.61	.00
200.00	4.25	.61	54.7	.183E-01	.61	9.46	.61	.00

Cumulative travel time = 259. sec

Simulation limit based on maximum specified distance = 200.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Because of the fairly LARGE SPACING between adjacent risers/nozzles/ports, the above results may be unreliable in the immediate near-field of the diffuser.

A SUBSEQUENT APPLICATION OF CORMIX1 IS RECOMMENDED to provide more detail for one of the individual jets/plumes in the initial region before merging.

CORMIX2: Submerged Multiport Diffuser Discharges End of Prediction File

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CORMIX2 PREDICTION FILE:
22222222222222222222222222222222222222222222222222222222222222222222222222
                    CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX2:                                          Subsystem version:
 Submerged Multiport Diffuser Discharges    CORMIX-GI Version 4.03b
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          Molalla
 Design case:              Jun 7Q10 Low; 2019 June Ave Flow; 3-8" Ports
 FILE NAME:                C:\Program Files\CORMIX-GI\My Files\moju3-8.prd
 Time of Fortran run:      02/04/2000--08:49:40

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =       9.50  AS     =       5.03  QA     =       3.09  ICHREG= 2
 HA     =        .53  HD     =        .53
 UA     =       .614  F      =       .242  USTAR = .1068E+00
 UW     =      2.000  UWSTAR= .2198E-02
 Uniform density environment
 STRCND=  U           RHOAM =  998.9443

DIFFUSER DISCHARGE PARAMETERS (metric units)
 Diffuser type:       DITYPE= unidirectional_perpendicular
 BANK   =  LEFT       DISTB =       3.95  YB1    =       2.10  YB2    =       5.80
 LD     =       3.70  NOPEN =    3        SPAC   =       1.85
 D0     =       .200  A0     =       .031  H0     =        .15
 Nozzle/port arrangement:   unidirectional_without_fanning
 GAMMA =      90.00   THETA =        .00  SIGMA =        .00  BETA   =      90.00
 U0     =       .629  Q0     =       .059        = .5930E-01
 RHO0   =  998.4063   DRHO0 = .5381E+00  GP0    = .5282E-02
 C0     = .1000E+01   CUNITS=  mg/l
 IPOLL =  1           KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES - PER UNIT DIFFUSER LENGTH (metric units)
 q0     = .1603E-01   m0     = .1008E-01  j0     = .8466E-04  SIGNJ0=       1.0
 Associated 2-d length scales (meters)
 lQ=B   =       .025  lM     =       5.22  lm     =        .03
 lmp    =  99999.00   lbp    =  99999.00  la     =  99999.00

FLUX VARIABLES - ENTIRE DIFFUSER (metric units)
 Q0     = .5930E-01   M0     = .3731E-01  J0     = .3132E-03
 Associated 3-d length scales (meters)
 LQ     =        .31  LM     =       4.80  Lm     =        .31  Lb     =        .00
                                            Lmp    =  99999.00  Lbp    =  99999.00

NON-DIMENSIONAL PARAMETERS
 FR0    =      54.24  FRD0   =      19.36  R      =       1.03
 (slot)               (port/nozzle)

FLOW CLASSIFICATION
 2222222222222222222222222222222222222222222
 2  Flow class (CORMIX2)       =    MU2    2
 2  Applicable layer depth HS =       .53  2
 2222222222222222222222222222222222222222222

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+01   CUNITS=  mg/l
 NTOX   =  1          CMC    = .2200E+02  CCC    =  CSTD
 NSTD   =  1          CSTD   = .2200E+01
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```
 REGMZ =  1
 REGSPC=  1          XREG  =     15.20  WREG  =        .00  AREG  =        .00
 XINT  =    200.00  XMAX  =    200.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the bottom and the diffuser mid-point:
         3.95 m  from the LEFT  bank/shore.
    X-axis points downstream, Y-axis points to left, Z-axis points upward.
NSTEP = 50 display intervals per module
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD201: DIFFUSER DISCHARGE MODULE

 Due to complex near-field motions:  EQUIVALENT SLOT DIFFUSER (2-D) GEOMETRY

 Profile definitions:
   BV = Gaussian 1/e (37%) half-width, in vertical plane normal to trajectory
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
      .00      .00      .15      1.0  .100E+01     .02     1.85
** CMC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below CMC value of  .220E+02
   due to mixing in this control volume.
 The actual extent of the TOXIC DILUTION ZONE will be smaller than control
   volume outflow values predicted below.

** WATER QUALITY STANDARD OR CCC HAS BEEN FOUND **
 The pollutant concentration in the plume falls below water quality standard
   or CCC value of  .220E+01 due to mixing in this control volume.
 The actual extent of the zone at whose boundary the water quality
   standard or the CCC is exceeded will be smaller than the control
   volume outflow values predicted below.


END OF MOD201: DIFFUSER DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

 In this laterally contracting zone the diffuser plume becomes VERTICALLY FULLY
  MIXED over the entire layer depth (HS =      .53m).
   Full mixing is achieved after a plume distance of about five
   layer depths from the diffuser.

 Profile definitions:
   BV = layer depth (vertically mixed)
   BH = top-hat half-width, in horizontal plane normal to trajectory
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X        Y        Z        S        C        BV       BH
      .00      .00      .15      1.0  .100E+01     .02     1.85
      .04      .00      .15      3.8  .263E+00     .03     1.85
      .07      .00      .15      5.0  .202E+00     .04     1.85
      .11      .00      .16      5.8  .171E+00     .04     1.84
      .15      .00      .16      6.6  .152E+00     .05     1.84
      .19      .00      .16      7.3  .138E+00     .06     1.84
```

.22	.00	.16	7.9	.127E+00	.06	1.84
.26	.00	.17	8.4	.119E+00	.07	1.84
.30	.00	.17	8.9	.112E+00	.08	1.83
.33	.00	.17	9.4	.106E+00	.10	1.83
.37	.00	.17	9.9	.101E+00	.11	1.83
.41	.00	.18	10.3	.972E-01	.12	1.83
.44	.00	.18	10.7	.935E-01	.13	1.83
.48	.00	.18	11.1	.901E-01	.14	1.83
.52	.00	.18	11.5	.871E-01	.15	1.82
.56	.00	.18	11.8	.844E-01	.16	1.82
.59	.00	.19	12.2	.820E-01	.17	1.82
.63	.00	.19	12.5	.797E-01	.18	1.82
.67	.00	.19	12.9	.777E-01	.19	1.82
.70	.00	.19	13.2	.757E-01	.20	1.82
.74	.00	.20	13.5	.740E-01	.21	1.82
.78	.00	.20	13.8	.723E-01	.22	1.82
.81	.00	.20	14.1	.708E-01	.23	1.82
.85	.00	.20	14.4	.693E-01	.24	1.82
.89	.00	.21	14.7	.680E-01	.25	1.82
.93	.00	.21	15.0	.667E-01	.26	1.81
.96	.00	.21	15.3	.655E-01	.28	1.81
1.00	.00	.21	15.5	.643E-01	.29	1.81
1.04	.00	.21	15.8	.632E-01	.30	1.81
1.07	.00	.22	16.1	.622E-01	.31	1.81
1.11	.00	.22	16.3	.612E-01	.32	1.81
1.15	.00	.22	16.6	.603E-01	.33	1.81
1.18	.00	.22	16.8	.594E-01	.34	1.81
1.22	.00	.23	17.1	.585E-01	.35	1.81
1.26	.00	.23	17.3	.577E-01	.36	1.81
1.29	.00	.23	17.6	.569E-01	.37	1.81
1.33	.00	.23	17.8	.562E-01	.38	1.81
1.37	.00	.24	18.0	.555E-01	.39	1.81
1.41	.00	.24	18.3	.548E-01	.40	1.81
1.44	.00	.24	18.5	.541E-01	.41	1.81
1.48	.00	.24	18.7	.535E-01	.42	1.81
1.52	.00	.24	18.9	.528E-01	.43	1.81
1.55	.00	.25	19.1	.522E-01	.45	1.81
1.59	.00	.25	19.4	.517E-01	.46	1.81
1.63	.00	.25	19.6	.511E-01	.47	1.81
1.66	.00	.25	19.8	.506E-01	.48	1.81
1.70	.00	.26	20.0	.500E-01	.49	1.81
1.74	.00	.26	20.2	.495E-01	.50	1.81
1.78	.00	.26	20.4	.490E-01	.51	1.81
1.81	.00	.26	20.6	.486E-01	.52	1.81
1.85	.00	.26	20.8	.481E-01	.53	1.81

Cumulative travel time = 3. sec

Plume centerline may exhibit slight discontinuities in transition to subsequent far-field module.

END OF MOD271: ACCELERATION ZONE OF UNIDIRECTIONAL CO-FLOWING DIFFUSER

BEGIN MOD251: DIFFUSER PLUME IN CO-FLOW

Phase 1: Vertically mixed, Phase 2: Re-stratified

Phase 1: The diffuser plume is VERTICALLY FULLY MIXED over the entire layer depth.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

Phase 2: The flow has RESTRATIFIED at the beginning of this zone.

This flow region is INSIGNIFICANT in spatial extent and will be by-passed.

END OF MOD251: DIFFUSER PLUME IN CO-FLOW

** End of NEAR-FIELD REGION (NFR) **

BEGIN MOD241: BUOYANT AMBIENT SPREADING

Discharge is non-buoyant or weakly buoyant.
Therefore BUOYANT SPREADING REGIME is ABSENT.

END OF MOD241: BUOYANT AMBIENT SPREADING

BEGIN MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .113E-01 m^2/s
Horizontal diffusivity (initial value) = .283E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to layer depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
ZU = upper plume boundary (Z-coordinate)
ZL = lower plume boundary (Z-coordinate)
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 1 (not bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
1.85	.00	.53	20.8	.481E-01	.53	1.86	.53	.00

Plume interacts with BOTTOM.
The passive diffusion plume becomes VERTICALLY FULLY MIXED within this prediction interval.

3.56	.00	.53	21.5	.465E-01	.53	1.92	.53	.00
5.27	.00	.53	22.2	.450E-01	.53	1.99	.53	.00
6.97	.00	.53	22.9	.437E-01	.53	2.05	.53	.00
8.68	.00	.53	23.5	.425E-01	.53	2.10	.53	.00
10.39	.00	.53	24.2	.414E-01	.53	2.16	.53	.00
12.10	.00	.53	24.8	.403E-01	.53	2.22	.53	.00
13.81	.00	.53	25.4	.394E-01	.53	2.27	.53	.00

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds the regulatory value = 15.20 m.
This is the extent of the REGULATORY MIXING ZONE.

15.51	.00	.53	26.0	.385E-01	.53	2.32	.53	.00
17.22	.00	.53	26.6	.376E-01	.53	2.38	.53	.00
18.93	.00	.53	27.1	.369E-01	.53	2.43	.53	.00
20.64	.00	.53	27.7	.361E-01	.53	2.48	.53	.00
22.35	.00	.53	28.2	.354E-01	.53	2.52	.53	.00
24.05	.00	.53	28.8	.348E-01	.53	2.57	.53	.00
25.76	.00	.53	29.3	.342E-01	.53	2.62	.53	.00
27.47	.00	.53	29.8	.336E-01	.53	2.67	.53	.00
29.18	.00	.53	30.3	.330E-01	.53	2.71	.53	.00
30.89	.00	.53	30.8	.325E-01	.53	2.75	.53	.00
32.59	.00	.53	31.3	.320E-01	.53	2.80	.53	.00
34.30	.00	.53	31.8	.315E-01	.53	2.84	.53	.00

36.01	.00	.53	32.2	.310E-01	.53	2.88	.53	.00
37.72	.00	.53	32.7	.306E-01	.53	2.93	.53	.00
39.43	.00	.53	33.2	.301E-01	.53	2.97	.53	.00
41.13	.00	.53	33.6	.297E-01	.53	3.01	.53	.00
42.84	.00	.53	34.1	.293E-01	.53	3.05	.53	.00
44.55	.00	.53	34.5	.290E-01	.53	3.09	.53	.00
46.26	.00	.53	35.0	.286E-01	.53	3.13	.53	.00
47.97	.00	.53	35.4	.283E-01	.53	3.16	.53	.00
49.67	.00	.53	35.8	.279E-01	.53	3.20	.53	.00
51.38	.00	.53	36.2	.276E-01	.53	3.24	.53	.00
53.09	.00	.53	36.6	.273E-01	.53	3.28	.53	.00
54.80	.00	.53	37.1	.270E-01	.53	3.31	.53	.00
56.51	.00	.53	37.5	.267E-01	.53	3.35	.53	.00
58.21	.00	.53	37.9	.264E-01	.53	3.39	.53	.00
59.92	.00	.53	38.3	.261E-01	.53	3.42	.53	.00
61.63	.00	.53	38.7	.259E-01	.53	3.46	.53	.00
63.34	.00	.53	39.1	.256E-01	.53	3.49	.53	.00
65.05	.00	.53	39.4	.254E-01	.53	3.53	.53	.00
66.75	.00	.53	39.8	.251E-01	.53	3.56	.53	.00
68.46	.00	.53	40.2	.249E-01	.53	3.60	.53	.00
70.17	.00	.53	40.6	.246E-01	.53	3.63	.53	.00
71.88	.00	.53	41.0	.244E-01	.53	3.66	.53	.00
73.59	.00	.53	41.3	.242E-01	.53	3.70	.53	.00
75.29	.00	.53	41.7	.240E-01	.53	3.73	.53	.00
77.00	.00	.53	42.1	.238E-01	.53	3.76	.53	.00
78.71	.00	.53	42.4	.236E-01	.53	3.79	.53	.00
80.42	.00	.53	42.8	.234E-01	.53	3.83	.53	.00
82.13	.00	.53	43.1	.232E-01	.53	3.86	.53	.00
83.83	.00	.53	43.5	.230E-01	.53	3.89	.53	.00
85.54	.00	.53	43.8	.228E-01	.53	3.92	.53	.00
87.25	.00	.53	44.2	.226E-01	.53	3.95	.53	.00

Cumulative travel time = 139. sec

Plume Stage 2 (bank attached):

X	Y	Z	S	C	BV	BH	ZU	ZL
87.25	3.95	.53	44.2	.226E-01	.53	7.90	.53	.00
89.50	3.95	.53	44.3	.226E-01	.53	7.92	.53	.00
91.76	3.95	.53	44.4	.225E-01	.53	7.94	.53	.00
94.01	3.95	.53	44.5	.225E-01	.53	7.96	.53	.00
96.27	3.95	.53	44.6	.224E-01	.53	7.98	.53	.00
98.52	3.95	.53	44.7	.224E-01	.53	8.00	.53	.00
100.78	3.95	.53	44.8	.223E-01	.53	8.02	.53	.00
103.03	3.95	.53	45.0	.222E-01	.53	8.04	.53	.00
105.29	3.95	.53	45.1	.222E-01	.53	8.06	.53	.00
107.54	3.95	.53	45.2	.221E-01	.53	8.08	.53	.00
109.80	3.95	.53	45.3	.221E-01	.53	8.10	.53	.00
112.05	3.95	.53	45.4	.220E-01	.53	8.12	.53	.00
114.31	3.95	.53	45.5	.220E-01	.53	8.14	.53	.00
116.56	3.95	.53	45.6	.219E-01	.53	8.16	.53	.00
118.82	3.95	.53	45.7	.219E-01	.53	8.18	.53	.00
121.07	3.95	.53	45.8	.218E-01	.53	8.20	.53	.00
123.33	3.95	.53	45.9	.218E-01	.53	8.22	.53	.00
125.58	3.95	.53	46.1	.217E-01	.53	8.24	.53	.00
127.84	3.95	.53	46.2	.217E-01	.53	8.26	.53	.00
130.09	3.95	.53	46.3	.216E-01	.53	8.28	.53	.00
132.35	3.95	.53	46.4	.216E-01	.53	8.30	.53	.00
134.60	3.95	.53	46.5	.215E-01	.53	8.32	.53	.00
136.86	3.95	.53	46.6	.215E-01	.53	8.33	.53	.00
139.12	3.95	.53	46.7	.214E-01	.53	8.35	.53	.00

141.37	3.95	.53	46.8	.214E-01	.53	8.37	.53	.00
143.63	3.95	.53	46.9	.213E-01	.53	8.39	.53	.00
145.88	3.95	.53	47.0	.213E-01	.53	8.41	.53	.00
148.14	3.95	.53	47.1	.212E-01	.53	8.43	.53	.00
150.39	3.95	.53	47.2	.212E-01	.53	8.45	.53	.00
152.65	3.95	.53	47.3	.211E-01	.53	8.47	.53	.00
154.90	3.95	.53	47.4	.211E-01	.53	8.49	.53	.00
157.16	3.95	.53	47.6	.210E-01	.53	8.51	.53	.00
159.41	3.95	.53	47.7	.210E-01	.53	8.52	.53	.00
161.67	3.95	.53	47.8	.209E-01	.53	8.54	.53	.00
163.92	3.95	.53	47.9	.209E-01	.53	8.56	.53	.00
166.18	3.95	.53	48.0	.208E-01	.53	8.58	.53	.00
168.43	3.95	.53	48.1	.208E-01	.53	8.60	.53	.00
170.69	3.95	.53	48.2	.208E-01	.53	8.62	.53	.00
172.94	3.95	.53	48.3	.207E-01	.53	8.64	.53	.00
175.20	3.95	.53	48.4	.207E-01	.53	8.66	.53	.00
177.45	3.95	.53	48.5	.206E-01	.53	8.67	.53	.00
179.71	3.95	.53	48.6	.206E-01	.53	8.69	.53	.00
181.96	3.95	.53	48.7	.205E-01	.53	8.71	.53	.00
184.22	3.95	.53	48.8	.205E-01	.53	8.73	.53	.00
186.47	3.95	.53	48.9	.204E-01	.53	8.75	.53	.00
188.73	3.95	.53	49.0	.204E-01	.53	8.77	.53	.00
190.98	3.95	.53	49.1	.204E-01	.53	8.78	.53	.00
193.24	3.95	.53	49.2	.203E-01	.53	8.80	.53	.00
195.49	3.95	.53	49.3	.203E-01	.53	8.82	.53	.00
197.75	3.95	.53	49.4	.202E-01	.53	8.84	.53	.00
200.00	3.95	.53	49.5	.202E-01	.53	8.86	.53	.00

Cumulative travel time = 320. sec

Simulation limit based on maximum specified distance = 200.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD261: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Because of the fairly LARGE SPACING between adjacent risers/nozzles/ports, the above results may be unreliable in the immediate near-field of the diffuser.
A SUBSEQUENT APPLICATION OF CORMIX1 IS RECOMMENDED to provide more detail for one of the individual jets/plumes in the initial region before merging.

CORMIX2: Submerged Multiport Diffuser Discharges End of Prediction File
2222222222222222222222222222222222222222222222222222222222222222222222222222

ATTACHMENT 2

DO ANALYSIS – STREETER-PHELPS EQUATION CALCULATIONS

CLIENT: Molalla | PROJECT: Effluent/Irrigation PS, FM, outfall

SUBJECT: Water Quality Analysis - DO

Analysis based on STREETER-PHELPS equation described in Metcalf and Eddy, 1991, pp. 1218-1220

Conditions evaluated for November, December, and June as also used for the mixing zone analysis

① Determine stream DO prior to discharge

Assumed data (worst case conditions for DO are 95% sat)

Condition	Temp	DO at 100% sat	DO at 95% sat
Nov	14°C	10.23 mg/L	(10.23)(0.95) = 9.72 mg/L
Dec	12°C	10.69 mg/L	10.16 mg/L
June	18°C	9.79 mg/L	9.30 mg/L

② Determine temp, DO, and BOD of river/effluent mix

Assumed data (November)

Parameter	Effluent	River	Basis for River Data
Flow	1.84 mgd	33 mgd	November 7Q10 Low
Temp	18°C	14°C	November max
DO	6.0 mg/L	9.72 mg/L	From ① above
BOD	10.0 mg/L	1.3 mg/L	Oct max from STORET

T_{mix} (Nov) = [(14)(33) + (18)(1.84)]/34.84 = 14.2°C

DO_{mix} (Nov) = [(9.72)(33) + (6.0)(1.84)]/34.84 = 9.52 mg/L

$BOD_{5,mix}$ (Nov) = [(1.3)(33) + (10.0)(1.84)]/34.84 = 1.76 mg/L

$BOD_{ult,mix}$ (Nov) = $(1.76)/(1-e^{-(0.14)(5)})$ = 3.50 mg/L

Assumed data (December)

Parameter	Effluent	River	Basis for River Data
Flow	1.86 mgd	113 mgd	December 7Q10 Low
Temp	15°C	12°C	December Max
DO	6.0 mg/L	10.16 mg/L	From ① above
BOD	10.0 mg/L	1.3 mg/L	Oct max from STORET

CLIENT: Molalla | PROJECT:

SUBJECT: Water Quality Analysis - DO, cont.

② cont.

T_{mix} (Dec) = [(12)(113) + (15)(1.86)]/114.86 = 12.0°C

DO_{mix} (Dec) = [(10.16)(113) + (6.0)(1.86)]/114.86 = 10.09 mg/L

$BOD_{5,mix}$ (Dec) = [(1.3)(113) + (10.0)(1.86)]/114.86 = 1.44 mg/L

$BOD_{ult,mix}$ (Dec) = (1.44)/(1 - $e^{-(0.11)(5)}$) = 3.40 mg/L

Assumed data (June):

Parameter	Effluent	River	Basis for River Data
Flow	1.17 mgd	70 mgd	June 7Q10 Low
Temp	19°C	16°C	June max
DO	6.0 mg/L	9.30 mg/L	From ① above
BOD	10.0 mg/L	1.3 mg/L	Oct max from STORET

T_{mix} (Jun) = [(16)(70) + (19)(1.17)]/71.17 = 16.0°C

DO_{mix} (Jun) = [(9.30)(70) + (6.0)(1.17)]/71.17 = 9.25 mg/L

BOD_{mix} (Jun) = [(1.3)(70) + (10.0)(1.17)]/71.17 = 1.44 mg/L

$BOD_{ult,mix}$ (Jun) = (1.44)/(1 - $e^{-(0.18)(5)}$) = 2.43 mg/L

③ Determine rate constants for T_{mix}

$K_{1,18}$ = 0.23 ⇒ $K_{1,14.2}$ = (0.23)(1.135)$^{14.2-18}$ = 0.14 (Nov)

$K_{1,12}$ = (0.23)(1.135)$^{12-18}$ = 0.11 (Dec)

$K_{1,16}$ = (0.23)(1.135)$^{16-18}$ = 0.18 (Jun)

$K_{2,18}$ = 0.67 ⇒ $K_{2,14.2}$ = (0.67)(1.024)$^{14.2-18}$ = 0.61 (Nov)

$K_{2,12}$ = (0.67)(1.024)$^{12-18}$ = 0.58 (Dec)

$K_{2,16}$ = (0.67)(1.024)$^{16-18}$ = 0.64 (Jun)

④ Determine critical time t_c and critical distance x_c

$DO_{sat,14.2}$ = 10.18 mg/L (Nov)

D_o (Nov) = 10.18 mg/L - 9.52 mg/L = 0.66 mg/L

$$t_c \text{ (Nov)} = \frac{1}{0.61 - 0.14} \ln \frac{0.61}{0.14}\left[1 - \frac{(0.66)(0.61 - 0.14)}{(0.14)(3.50)}\right] = 1.0 \text{ days}$$

x_c (Nov) = (1.5 fps)(1.0 days) = 130,000 ft = 24.5 mi

KCM	2930103	KJJ	02/08/00	2 OF 3
	JOB NO.	BY	DATE	SHEET NO.

CLIENT: Molalla | PROJECT:

SUBJECT: Water Quality Analysis - DO; cont.

④ cont.

$DO_{sat,-12.0} = 10.69\ mg/l$ (Dec)

D_o (Dec) $= 10.69\ mg/l - 10.09\ mg/l = 0.60\ mg/l$

$$t_c(Dec) = \frac{1}{0.58-0.11} \ln \frac{0.58}{0.11}\left[1 - \frac{0.60(0.58-0.11)}{(0.11)(3.40)}\right] = 0.55\ days$$

x_c (Dec) $= (2.5\ fps)(0.55\ days) = 120{,}000\ ft = 22.6\ mi$

$DO_{sat,16.0} = 9.79\ mg/l$ (Jun)

D_o (Jun) $= 9.79\ mg/l - 9.25\ mg/l = 0.54\ mg/l$

$$t_c(Jun) = \frac{1}{0.64-0.18} \ln \frac{0.64}{0.18}\left[1 - \frac{0.54(0.64-0.18)}{(0.18)(2.43)}\right] = 0.93\ days$$

x_c (Jun) $= (2.0\ fps)(0.93\ days) = 161{,}000\ ft = 30.6\ mi$

⑤ Determine critical DO deficit and critical DO

D_c (Nov) $= (0.14/0.61)(3.50)(e^{-(0.14)(1.0)}) = 0.70\ mg/l$

DO_c (Nov) $= 10.18\ mg/l - 0.70\ mg/l = 9.48\ mg/l$

DO_{drop} (Nov) $= 9.72\ mg/l - 9.48\ mg/l = \underline{0.24\ mg/l > 0.1\ mg/l}$

D_c (Dec) $= (0.11/0.58)(3.40)(e^{-(0.11)(0.55)}) = 0.61\ mg/l$

DO_c (Dec) $= 10.69\ mg/l - 0.61\ mg/l = 10.08\ mg/l$

DO_{drop} (Dec) $= 10.16\ mg/l - 10.08\ mg/l = \underline{0.08\ mg/l < 0.1\ mg/l}$

D_c (Jun) $= (0.18/0.64)(2.43)(e^{-(0.18)(0.93)}) = 0.58\ mg/l$

DO_c (Jun) $= 9.79\ mg/l - 0.58\ mg/l = 9.21\ mg/l$

DO_{drop} (Jun) $= 9.30\ mg/l - 9.21\ mg/l = \underline{0.09\ mg/l < 0.1\ mg/l}$

KCM	2930103	KSJ	02/08/00	3 of 3
	JOB NO.	BY	DATE	SHEET NO.

ATTACHMENT 3
TEMPERATURE AND PH ANALYSIS CALCULATIONS

CLIENT: Molalla

PROJECT: Effluent / Irrigation PS, FM, outfall

SUBJECT: Water Quality Analysis - Temperature Dilution Calcs

November 7Q10 Low Conditions

Dilution at edge of RMZ = 12.6 = 7.9 %

Temp River = 14°C

Temp Effluent = 18°C

Temp at edge of RMZ = (14)(1-0.079) + (18)(0.079)

= 14.32°C ⇒ 0.32°C increase

December 7Q10 Low Conditions

Dilution at edge of RMZ = 27.5 = 3.6 %

Temp River = 12°C

Temp Effluent = 15°C

Temp at edge of RMZ = (12)(1-0.036) + (15)(0.036)

= 12.11°C ⇒ 0.11°C increase

June 7Q10 Low Conditions

Dilution at edge of RMZ = 25.9 = 3.9 %

Temp River = 16°C

Temp Effluent = 19°C

Temp at edge of RMZ = (16)(1-0.039) + (19)(0.039)

= 16.12°C ⇒ 0.12°C increase

KCM	2930103	KJS	02/02/00	1 OF 1
	JOB NO.	BY	DATE	SHEET NO.

CLIENT: Molalla

PROJECT: Effluent/Irrigation PS, FM, Outfall

SUBJECT: Water Quality Analysis - pH Dilution Calcs

November 7Q10 Low Conditions

Dilution at edge of RMZ = 12.6 = 7.9 %

pH River = 6.8

pH Effluent = 6.0

pH at edge of RMZ = (6.8)(1 - 0.079) + (6.0)(0.079)

= 6.74

KCM	2930103	KSJ	02/02/00	1 of 1
	JOB NO.	BY	DATE	SHEET NO.

PERMIT APPLICATION INFORMATION FOR DISCHARGE TO THE MOLALLA RIVER

The Department of Environmental Quality (DEQ) regulates the National Pollution Discharge Elimination System (NPDES) permit program in the State of Oregon. The procedures and guidelines for the disposal of wastewater treatment plant (WWTP) effluent to a surface water is governed by Oregon Administrative Rules Chapter 340, Division 41 (OAR 340-41), *State-Wide Water Quality Management Plan; Beneficial Uses, Policies, Standards, and Treatment Criteria for Oregon*. In order to obtain any changes to existing permit limits, including a new surface water discharge, the changes must be approved by the DEQ and the Environmental Quality Commission (EQC). This appendix documents the findings from this plan with respect to mass load and water quality requirements and is provided for informational purposes only; actual mass load limit revisions must be negotiated with DEQ.

BASIS FOR NEW SURFACE WATER DISCHARGE

The Molalla WWTP currently maximizes the use of the Bear Creek surface water discharge mass loads established in the current discharge permit. The increase in wastewater flows and loads to the plant projected in this plan can not be accommodated within the existing mass load limits. As discussed in Chapter 8, the plant would have to produce 1.5 milligrams per liter (mg/l) BOD or better effluent to achieve the current mass load limit at the Phase 2 MMWWF.

Since the mass load limits for Bear Creek are not likely to be increased, it was determined that the City must develop another means of winter effluent disposal. The discussion of the use of the Molalla River for surface water discharge is included in Chapter 8. This appendix outlines the suggestions for adding a Molalla River discharge to the plant's current NPDES permit. It is assumed that the current limits for Bear Creek would remain unchanged and that the City would continue to use this discharge.

SUGGESTED PERMIT LIMITS

Table 1 shows the suggested permit limits for the Molalla River surface water discharge. These limits are similar to the current Bear Creek limits except for the mass loads. The concentration limits are equal to the more stringent summer-time basin standards for the Willamette Basin. The basis for the mass load limits are these concentration limits and the 2019 MMWWF that must be discharged to the Molalla River.

TABLE 1.
MOLALLA RIVER PERMIT LIMITS

	Max. Conc. (mg/l)		Max. Mass Load (ppd), (1)		
November 1—April 30 (Winter)					
	Monthly Average	Weekly Average	Monthly Average	Weekly Average	Daily
Biochemical Oxygen Demand (BOD_5)	10	15	384	576	768
Total Suspended Solids (TSS)	10	15	384	576	768
May 1—June 30 (Early Summer)					
	Max. Conc. (mg/l)		Max. Mass Load (ppd), (2)		
	Monthly Average	Weekly Average	Monthly Average	Weekly Average	Daily
Biochemical Oxygen Demand (BOD_5)	10	15	175	263	350
Total Suspended Solids (TSS)	10	15	175	263	350
July 1—October 31 (Mid and Late Summer) – No discharge allowed.					
Dilution	Effluent BOD_5 concentration in mg/l, divided by the dilution factor (ratio of streamflow to effluent flow) shall not exceed 1.				
pH	Shall be within the range of 6.0 – 9.0 standard units.				
Escherichia coli (E. coli)	30-day log mean of 126 organisms per 100 ml and no single sample shall exceed 406 organisms per 100 ml.				
BOD5 and TSS percent removal	Efficiency shall not be less than 85 percent monthly average.				
Mixing Zone	Notwithstanding the effluent limitations established by the permit, except as provided for in OAR 340-45-080, no wastes shall be discharged and no activities shall be conducted which will violate water quality standards as adopted in OAR 340-41-485 except in the defined mixing zone: The mixing zone shall not exceed a fifty-foot radius from the point of discharge.				

1) Based on the Phase 2 MMWWF of 5.4 mgd minus the Bear Creek discharge of 0.8 mgd shown in the water balance evaluation for a total of 4.6 mgd discharged to the Molalla River.
2) Based on the Phase 2 ADWF of 2.1 mgd.

REQUIREMENTS FOR NEW DISCHARGE

In order have any increase in mass load limits approved by the DEQ, the City will have to provide the information required by the policies presented in Section 26 of the OAR 340-41. These policies are outlined below with suggested responses to each item. It must be noted that the suggested responses are intended for use as guidelines only and do not constitute the submittal that will be required by DEQ for a new surface water discharge. These responses are provided only as documentation of the findings from this plan with respect to general water quality requirements.

3(a)(A) *The new or increased load would not cause water quality standards to be violated.*—The City requests 384 ppd BOD_5 and 384 ppd TSS mass loads on an average monthly basis during the winter. The City also requests 175 ppd BOD_5 and 175 ppd TSS mass loads on an average monthly basis during May and June. The standard 150 percent weekly and 200 percent daily multipliers are also requested. The winter mass loading would not violate water quality basin standards for the receiving stream. The winter discharge request, in fact, holds to the more strict summer requirements of the basin standards. The mass loading for the early summer months would also not violate water quality basin standards for the receiving stream. Water quality modeling was performed and is included in Appendix G.

3(a)(B) *The new or increased discharge load would not unacceptably threaten or impair any recognized beneficial uses.*—The proposed mass load limits would not impair any recognized beneficial uses such as recreation, fishing, and transportation. The proposed facilities include dechlorination, effectively removing the concern for chlorine toxicity associated with a chlorinated discharge. Additionally, the proposed facilities include reaeration of the dechlorinated effluent to ensure that the impacts of the effluent discharge on the dissolved oxygen levels in the river are minimized.

3(a)(C)(i) *The new or increased discharge load shall not be granted if the receiving stream is classified as being water quality limited, unless the pollutant parameters associated with the proposed discharge are unrelated, either directly or indirectly, to the parameters causing the receiving stream to be water quality limited.*—As outlined in Chapter 4, the Molalla River is currently listed by DEQ to be water quality limited. The water quality parameters of concern are reported to be high bacteria levels in the fall, winter, and spring and high temperatures in the summer. For the proposed discharge during the winter months, only the high bacteria levels are of concern. The OAR 340 basin standards for bacteria are the same as what is on the proposed discharge permit. Meeting the discharge permit limits will therefore ensure that basin standards for bacteria are not violated. As a result, the impact of the discharge on the river with regard to bacteria levels is within regulatory requirements. For the proposed discharge during the summer months, temperature levels are of concern. The water quality analysis includes a temperature analysis that indicates temperature standards will not be violated as a result of the proposed discharge during May and June.

3(a)(J) *The activity, expansion, or growth necessitating a new or increased discharge load is consistent with the acknowledged local land use plans as evidenced by a statement of land use compatibility from the appropriate local planning agency.*—The Molalla Comprehensive Plan was most recently published in 1987 and the population projections in the plan are somewhat outdated. The growth rate used in this plan to project population, and subsequent wastewater flows and loads, are consistent with those used in the Molalla Water System Master Plan, EAS, 1996 that was approved by the City. The City's Comprehensive Plan will be updated to more closely reflect the growth rates used in this plan and the 1996 Water System Master Plan. The periodic review process for the Comprehensive Plan will obtain the approvals of Clackamas County and Department of Land Conservation and Development (DLCD).

3(b)(B)(i) *The assimilative capacity of Oregon streams are finite, but the potential uses of this capacity are virtually unlimited. Thus, it is important that priority be given to those beneficial uses that promise the greatest return (beneficial use) relative to the unused assimilative capacity that might be utilized. In-stream uses that will benefit from reserve assimilative capacity, as well as potential future beneficial use, will be weighed against the economic benefit associated with increased loading.*—The proposed discharge to the Molalla River is for wet weather months only when the assimilative capacity of the river is at its highest. Also, the evaluation of the Molalla River discharge assumed a dilution requirement, shown in Table 1, that limits the impact of the proposed discharge on the assimilative capacity of the river when the streamflow is low. Additionally, the concentration limits assumed for the permit adhere to the more stringent summer-time basin standards even though the discharge will only occur during the winter and early wet weather summer months.

3(b)(B)(ii) *The cost of improved treatment technology, non-discharge, and limited discharge shall be evaluated.*—Improved treatment technology will not eliminate the need for a new surface water discharge since, as stated above, the plant would have to produce 1.5 mg/l BOD or better effluent to achieve the current mass load limit at the Phase 2 MMWWF. No economically feasible technology is available to meet this quality requirement on a continuous basis.

The proposed improvements do include non-discharge during the dry summer months when the City can land apply the effluent and the assimilative capacity of the river is at its lowest. Non-discharging alternatives for winter months, such as holding, are not economically feasible.

Another means by which to limit the mass load requirements of the proposed discharge is to expand the advanced treatment system of the plant to continue to consistently produce an effluent BOD concentration of 5 mg/l or better. The recommended facilities include a 4.4 mgd total capacity for the advanced treatment facilities. To achieve a 5 mg/l BOD concentration in the effluent, it would be most effective to bypass the facultative lagoons (to minimize algae growth) and the dissolved air flotation (DAF) units and size the filters for the PDAF of 10.9 mgd, or 2.5 times the currently recommended capacity. Addition of this filter capacity has a total project cost of $3,100,000 with construction contingencies and engineering, administration, and legal fees. Assuming this total project cost is financed with a 20-year loan at a 6 percent interest rate, these facilities translate to a monthly cost of $22,500. The operation and maintenance cost of the system would be an additional cost of approximately $5,000 per month. Assuming 1,700 existing equivalent residential units (ERUs), the required rate increase would be approximately $16 per month per ERU. This is a substantial rate increase that would be in addition to the rate increase required to accommodate the recommended facilities. As a result, it is recommended that the City negotiate with DEQ for the mass loads shown in Table 1.

MAPS AND PHOTOS OF MOLALLA RIVER FOR DISCHARGE TO THE MOLALLA RIVER

The following pages include maps and photos of the Molalla River in the vicinity of the proposed discharge.

LOOKING UPSTREAM FROM PROPOSED DISCHARGE

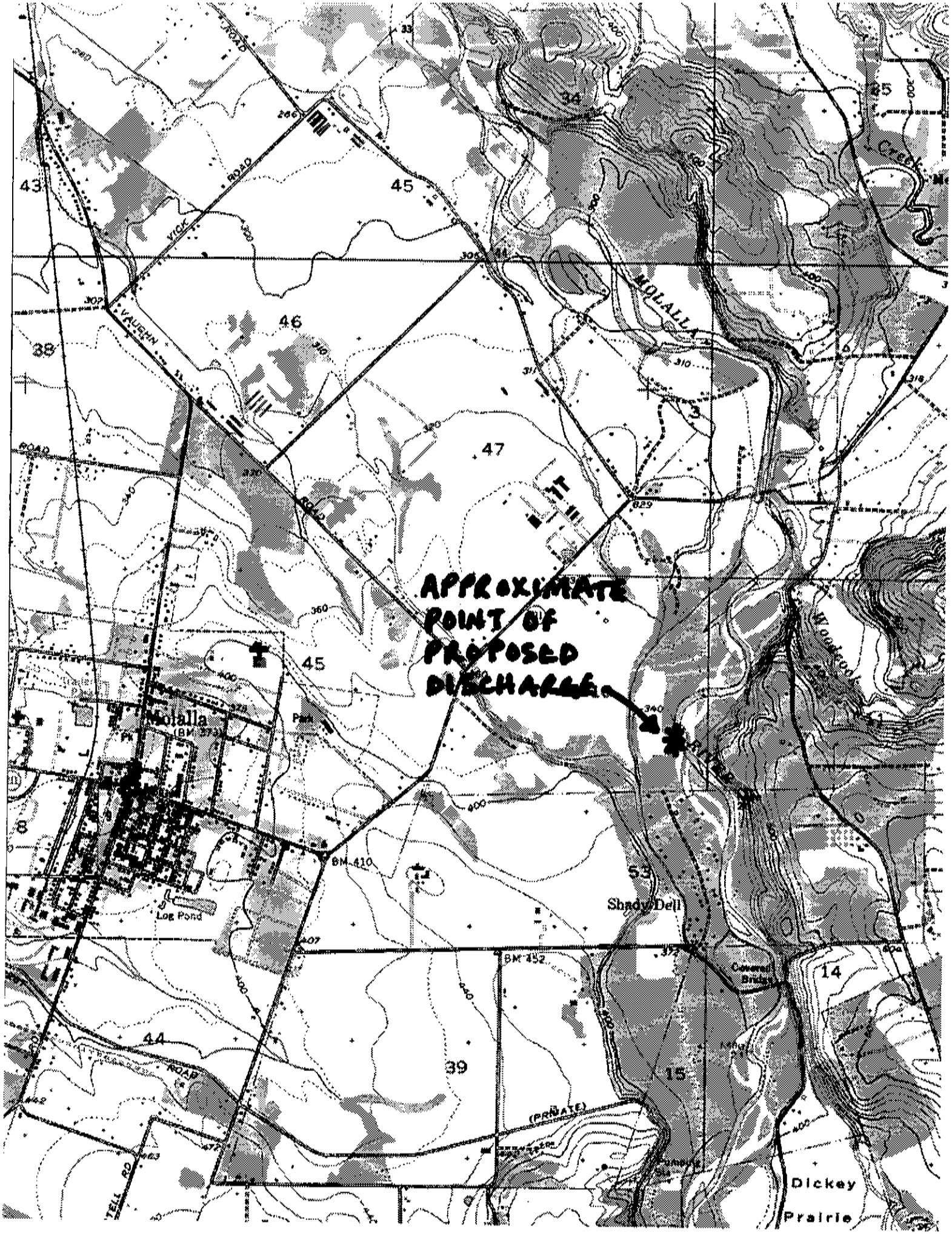
LOOKING DOWNSTREAM FROM PROPOSED DISCHARGE

APPROXIMATE LOCATION OF PROPOSED DISCHARGE TO MOLALLA RIVER

LOOKING OVER RIVER (EAST) FROM PROPOSED DISCHARGE

LOOKING TOWARD RIVER (EAST) AT PROPOSED DISCHARGE

APPROXIMATE LOCATION OF PROPOSED DISCHARGE TO MOLALLA RIVER


APPROXIMATE POINT OF PROPOSED DISCHARGE
MOLALLA
Molalla
Log Pond
Shady Dell
Covered Bridge
Dickey Prairie
VICK ROAD
VAUGHN
(PRIVATE)
BM 410
BM 452
43
45
46
47
38
34
35
44
39
53
14
15
8

PUBLIC INVOLVEMENT

The following pages present the agendas and the minutes from the two public involvement meetings held in conjunction with the development of this plan.

Vaughn Brown, Jerry Minor
Gene, Dean
Gary Duyle
Phil - Pioneer

City of Molalla

Draft Wastewater Facilities Plan

Open House
November 17, 1999
Molalla Senior Center
7:00 – 8:30 PM

Agenda

7:00	**Register** Get some refreshments, Preview information materials, Chat informally with staff	
7:15	**Draft Wastewater Facilities Plan Overview** Welcome and Introductions Wastewater Study Purpose and Need Study Findings and Recommendations	 Vaughn Brown Gene Green Jerry Minor Kay Swartwood
7:45	**Public Input** Questions, Suggestions, Concerns	 Group Discussion
8:25	**Summary & Next Steps**	Vaughn Brown
8:30	**Adjourn**	

City of Molalla
Fact Sheet
Draft Wastewater Facilities Plan

The City of Molalla is in the final stages of wastewater facilities planning. A Draft Wastewater Facilities Plan has been prepared, which includes the following:

- A description of existing conditions
- A description of existing wastewater facility problems
- Recommendations for system improvements and upgrades
- Estimated upgrade costs

Existing Facility Assessment

Collection System – The collection system condition is generally fair, however more than half of the system is over 40 years old. Relatively high levels of leakage into the system cause system surcharging during significant winter storms. Although the leakage is widespread, it is more cost-effective to transport and treat it than to remove it, though a maintenance program is required to keep leakage from getting even worse.

Treatment Plant – A significant number of components of the treatment plant are undersized and near the end of their design life. Upgrades are required to meet current treatment requirements.

Discharge – The City discharges treated wastewater to the intermittent Bear Creek in wet weather and stores and irrigates on agriculture land during dry weather. Treated wastewater that discharges to Bear Creek exceeds permitted limits during high rainfall periods. Dry weather irrigation presently exceeds crop needs.

Current Circumstances

Molalla's population is projected to grow from its current size of approximately 5,800 to approximately 10,000 by 2009 and nearly 13,500 by 2019. The existing wastewater treatment system is barely adequate to serve the current population and certainly will not be able to meet increasing future service demands.

This year, the Department of Environmental Quality issued the City a Notice of Noncompliance due to discharges exceeding permit limits. It is highly unlikely that the City will be allowed to discharge greater volumes into Bear Creek during the wet season. Summer application will also require significantly more land to accommodate existing and future larger volumes of wastewater.

The City must address the following immediate issues:

- Determine how to deal with increasing wet season discharge volumes
- Increase the amount of agricultural land for dry season irrigation of wastewater
- Undertake the upgrades and improvements necessary to meet current and future system needs
- Develop a funding plan to pay for the estimated $25-$30 million facilities improvement cost

Molalla Wastewater Facility Plan
Open House – Nov. 17
Flip Chart Notes

Discharge Information

- Treated effluent from Molalla plant is well above water quality standards – BOD, TSS and bacteria (even without chlorination)

- Dilution rate for discharge into Molalla River would be much greater than current rates for Bear Creek

- Reusing effluent for drinking water not feasible right now

System Improvement Ideas and Information Needs – to be discussed at next meeting

- The storage option allows for incremental expansion of storage capacity as City grows. Some costs can be delayed.

- Investigate possibility of discharging at some other point on Bear Creek to remove dilution as the limiting factor. May be able to discharge the mass load with more than one discharge point.

- What is the capital cost comparison between storing and pressure pipe installation over the twenty-year period. Spread costs over the years they will be incurred.

- Is it possible to discharge the full year's flow into the Molalla in a six-month wet season? Will DEQ allow it?

- What is the possibility of irrigating Arrowhead?

WASTEWATER FACILITIES PLAN PRESENTATION 11/16/99
7:00 p.m. at the Molalla Adult Center, 315 Kennel Avenue, Molalla, OR.

In Attendance: Vaughn Brown, Jerry Minor, Kae Swartwood, Gene Green, Dean Madison, Gary Tuyls, Paul Puffer, Don Kemp, Phil (Molalla Pioneer), Melanie Helmig, Dennis St. Clair, April Maier, Kern Buckner.

Vaughn Brown, Public Involvement Consultant, opened the presentation and advised that the meeting would consist of explaining the plan and to take public testimony and comments.

Gene Green, City Manager advised that the City encourages public input, and that the City Staff has the best interest of the City in consideration, and at the same time to be able to properly treat city wastewater. He also advised that there would probably be another meeting in approximately two weeks.

Jerry Minor, Project Manager, that the city's current structure was constructed in the 1950's. He advised that the collection system is structurally sound from an environmental standpoint. On the treatment side, he advised that plant was constructed in 1980, and is 20 years old. He advised that the City of Molalla's system is unique and that it may be the only system like it in the State of Oregon. He explained how the system functions. He advised that in winter months, the city has a permit to discharge into Bear Creek.

Kae Swartwood, showed with a chart the population increase, which also gave a projection of population. She advised that the current facility is not really capable of handling the future projection of population. She advised that the existing flow is at capacity, but that the plant is doing what it is designed to do, but that it has reached a 20-year age capacity.

Kae Swartwood also used a chart to show the treatment facilities under draft recommendation, which were: New headworks, new transfer station, new aeration basins, new clarifiers, new sludge lagoon, upgraded tertiary treatment, chlorination, and new dechlorination. She explained the proposed treatment process.

Gary Tuyls, asked how the process works with the current treatment process. Expressing concerns of sludge.

Jerry Minor explained the difference and advised that the sludge was of no concern, and that it had good fertilizer value.

Kae Swartwood also explained existing disposal and the future options.
EXISTING: Wet weather flows: discharge to Bear Creek.
Dry weather flows: holding and irrigation.

FUTURE ALTERNATIVES: Wet Weather Flows: Discharge to Bear Creek; holding or storage for irrigation in summer (would take approximately 200 acres for a big lagoon); discharge to a new receiving water.
Dry Weather Flow Alternative: Irrigation on additional land.

Gary Tuyls, asked if there was any way to increase the flow of Bear Creek. He also stated that approximately 35 years ago there was salmon fishing in Bear Creek.

Dennis St. Clair explained Bear Creek and it's flow capacity and seasonal capacities.

Don Kemp asked about the poundage of Bear Creek and asked about another flow output.

Jerry Minor felt that storage for summer irrigation would not be a feasible option. He further advised that you could only use so much water for irrigation, DEQ only allows up to a certain percentage, not interfering with the environment.

Kae also showed and explained the draft recommendation, whereas: Disposal Under Draft Recommendation: Wet Weather Flows: Discharge to Bear Creek and Discharge to Molalla River. Dry Weather Flows: Irrigate Existing Site and Irrigate Coleman Ranch. She showed where the Coleman Ranch was and where lines would run to that ranch and to the Molalla River, advising that there would not be an adverse effect to the Molalla River.

Dean Madison advised that the acreage of the Coleman Ranch would help the Molalla River, meaning the more irrigation to the Ranch, the less water he would draw from the River.

Jerry Minor explained that discharge to the Molalla River would be from November 1 to April 30th, that would be the only time DEQ would issue a

permit for that, and that they would make sure that requirements would be met. He further explained that DEQ would take weather conditions into consideration before allowing discharge, so that water quality would not be a concern.

Jerry Minor also advised that the City of Molalla has very clean wastewater, and explained the methods Molalla's Plant uses to achieve this. He further advised that the plant is very well operated. He advised that most cities in Oregon chlorinate, but that Molalla's is so clean that it is not necessary, and far exceeds standards. The treated effluent is well above required standards BOD, TSS and bacteria and that the treated water quality is very high.

Gary Tuyls expressed concern of Bear Creek flow becoming less if another pipe is installed. He further asked of the possibility of reusing effluent water as drinking water, which Mr. Minor advised was not feasible right now for monetary and technological reasons.

Paul Puffer asked about completion time estimates. Mr. Minor advised that if permission is received to proceed it could be complete by next summer, with the exception of the Molalla River pipe from the Coleman Ranch, which could be done by next fall.

Don Kemp expressed concerns of Molalla River accepting the amount of flow that is being discussed. He also asked if a different route (along Hwy. 213) was considered. Mr. Minor advised that a landowner was willing to allow the irrigation on his property, and although this route is slightly longer, the availability is there. He asked about a discharge line and acreage allowance and the cost difference between storage and the pipeline proposal. (200 acres of storage). Mr. Minor also advised that this would take 200-300 acres out of agriculture or residential use just for pond storage, and did not think that DEQ would allow the variables of this idea.

Mr. Kemp advised he would like to see a cost analysis between the two ideas.

DPW Madison advised that the analysis could be available in the December meeting. He also advised that another advantage on the chosen route is that the City owns a considerable amount of the logging road (route of the pipeline), so that would avoid going on public property.

Dennis St. Clair explained the irrigation needs of the Ranch/Farm and the different needs for irrigation depending on the crops, etc. He further advised

that these crops couldn't be used for human consumption, but that it must be used for feeding cattle, etc.

Paul Puffer asked how long the contract would be with Mr. Coleman, DPW Madison advised 25 years and then renewable after that, but that the city would still have the "right-a-way".

DPW Madison also advised that this would prevent building (i.e. subdivision, developments, etc.) on the Coleman property for at least 25 years.

There was discussion of the possibility of using Arrowhead Golf Course, rather than the Coleman property.

Jerry Minor advised that a separate rate and SDC analysis, which would fund the expenses, and that it would have a significant impact on Molalla's rates.

DPW Madison advised that rate projections might be brought to the December meeting, which would be close approximations of rate increases.

Vaughn Brown, advised that another meeting would be planned for 2-3 weeks from now. City Manager Green suggested December 1st as the next meeting. There was consensus to meet again on December 1, 1999 at 7:00 p.m. at the Adult Center, 315 Kennel Avenue.

Jerry Minor did not feel that rates and cost estimates would not be available for the next meeting.

DPW Madison felt that good data would be better than estimates or generalities.

Vaughn Brown adjourned the meeting at 8:40 p.m.

Minutes by: Melanie Helmig

Melanie Helmig, City Clerk/City Recorder

CITY OF MOLALLA
DRAFT Wastewater Facilities Plan

Open House #2
Meeting Plan Ideas

December 8, 1999

AGENDA

Meeting Purpose

1. Respond to Community Issues – Provide factual responses to community concerns
2. Inform Public – Improve community understanding of wastewater system need
3. Build Partnership – Initiate understanding of an dsupport for permitting and rate increase

Meeting Format – Group Discussion after Project Update Report

11:45 **Registration**

Distribute handouts. Early arrivers mingle with staff and consultants. Set up aerial photo(s), system schematic, study findings summary, etc.

12:00 **Presentation**

Jerry	Welcome, introductions and meeting overview.
Jerry/Kay	Short project review and update, response to information requests (see below).

12:30 **Discussion**

Roundtable discussion.

1:00 **Adjourn**

WASTEWATER FACILITIES PLAN PRESENTATION
12/08/99
12:00 P.M. at the Molalla Adult Center, 315 Kennel Avenue, Molalla, OR

In attendance: Jerry Minor, Kae Swartwood, Melanie Helmig, Don & Shirley Edwards, Minnow Seitoff, Clarence Cunningham, Dewayne Parker, Alma Bilyeu, Gary Tuyls, Marlene Seitoff, Phil (Molalla Pioneer). (City Manager Green, arriving late due to prior engagement, 12:30)

Mr. Minor introduced himself, explaining that KCM has presented a facilities plan draft to the City, which will soon be ready for final presentation.

He explained the existing system, which the majority of the system was constructed in the 1950's and is leaking. He further advised that these systems built at those dates are typically leaking today.

He explained the current treatment facility, showing by way of poster, the numerous lagoons and how the treatment works (headworks grinding as the preliminary treatment; aeration basis, stabilization bonds as secondary treatment, dissolved air flotation as a tertiary treatment filtration, chlorination also as secondary treatment).

Questions and concerns were asked regarding the allowance of dumping into Bear Creek, and especially during dry winter months. Mr. Minor advised that in the case of a dry winter month that the city would not be able to dispense into Bear Creek, creating a hardship to the plant.

Mr. Minor advised that the City has over discharged at times, which DEQ has been aware of.

Kae Swartwood, KCM, showed a draft chart of the City's population (current and projection). She advised that 13,000 people is the projection in the 20-year plan. She advised that the recommendation of the facility plan is: new headworks; new transfer station; new aeration basins; new clarifiers; new sludge lagoon; upgraded tertiary treatment; chlorination; new dechlorination. She explained the working methods of each, and highlighted the better efficiency of the recommended treatment plan.

Ms. Swartwood also explained the differences between existing disposal and future options.

Existing: Wet Weather Flows: discharge to Bear Creek.
Dry Weather Flows: holding and irrigation.

Alternatives for the future:
Wet Weather Flow Alternatives: discharge to Bear Creek; Holding or storage for irrigation in Summer; Discharge to a new receiving water.
Dry Weather Flow Alternative: Irrigation on additional land.

She advised that the disposal under the draft recommendation:
Wet Weather Flows: Discharge to Bear Creek; Discharge to Molalla River.
Dry Weather Flows: Irrigate existing site; irrigate Coleman Ranch.

She explained the process and showed the proposed pipe and its route, with its expandable line route.

Minnow Seitoff advised that the land to the south of the orange marked area on the Coleman Ranch there would be a potential problem of Creamery Creek and Bear Creek meeting could cause excessive standing water. He showed an area to the North of the orange marking that would handle excessive water without a problem. He further explained that 20 years will go by quickly and we will again be facing a similar problem.

Mr. Minor advised that the city has limits of irrigation from DEQ, which would summarize that there would be no over irrigation, which could cause excessive standing water.

Mrs. Seitoff expressed concern over the grazing of cattle from the treated water. Mr. Tuyls expressed similar concerns, and asked KCM to take another look at the proposal which could eliminate any large mistakes on the environment and later outcome to the future.

There were also expressed concerns of Mr. Coleman selling the property and then the City could be in danger of loosing the pipelines.

Mr. Minor advised that the City has recently signed a 20-year contract with Mr. Coleman, which protects the City for at least 20 years, and that it can be extended.

There was some concern of wetlands.

There were some concerns of pollution by the discharge being poured into the creek and rivers.

Ms. Swartwood advised that this is treated water, with nearly natural treatment rather than extensive chemicals, with the exception of some chlorine. But that there are many more things causing much more damage that the proposed treated water.

Mr. Minor advised that irrigation is planned for July, August and September months. Ms. Bilyeu advised that there is no drainage behind the new Methodist Church on the Coleman property and could not absorb a lot of heavy irrigation.

Mr. Cunningham advised that Molalla is too wet and has a drainage problem, and will only hold water and felt that it always would unless the citizens take action and stand together to find a solution to the problem to get rid of the excessive surface water.

Mr. Minnow asked if Academia could assist us in any way.

Mr. Tuyls felt that he understood what Mr. Cunningham was explaining, he concurred that the citizens need to gather together and find a permanent solution.

City Manager Green advised that nearly every community is facing similar situations, but because of monetary factors, solutions are limited somewhat. He advised that the City of Molalla is currently trying to find the best solutions with the city's best interest at heart. He advised that there is now a storm development charge to developers to help the City. He further explained that the developers and "wetland" areas have several complicated issues, and that those "wetland" area issues must be addressed.

There were several concerns of the city becoming more and more wet and soggy with more development coming in, which will create less & less drainage capability.

Manager Green advised that the City would have other options if the Coleman property would not be available in 25 years. He felt that ultimately the city may have to head to the river, he further advised that the western area may be a possibility. He advised that in California, there are golf courses, etc., that pay for the wastewater to irrigate with. He also advised that many cities plant "poplar trees" because of their known ability of absorption. He further explained that the city can't just decide a moratorium just to prevent growth, but explained that it's quite a process.

Mr. Minor explained the urban growth boundary regulations and development.

Mrs. Bilyeu expressed concern of drinking the water from the Molalla River after discharging human waste in it, and how will they get water if there is no snow pack, etc.

City Manager Green advised that the city stores water in the storage tanks.

Mr. Seitoff and City Manager Green spoke of the consideration now will help the youth of today. He further advised that the Molalla River Watch was also holding the youths future.

There was an emphasis on creek and river cleanup and getting the message to citizens and it's youth on the importance of clean water and the environment.

There was discussion of educating the youth in the schools on environment issues within our community, with permission from teachers in the classroom.

Meeting adjourned at 1:15

Melanie Helmig, City Recorder

RESPONSES TO DEQ REVIEW

The following pages present the review letter of the draft plan from the Department of Environmental Quality (DEQ) and the responses to the DEQ comments including information on how the draft plan was revised to address the DEQ review.

RESPONSES TO DEQ REVIEW

We have updated the draft plan, where appropriate, based on DEQ's review comments. The following is a detailed discussion of how we responded to the review comments. This discussion follows the review comments paragraph by paragraph.

Section 2.3, Population

The concern was raised about the long-term impacts that may occur if growth occurs at less than that projected. Though we recognize this concern, it is also not prudent to plan and budget for a projected population that may be less than the actual future population. There are several factors, detailed in subsection 2.3.2, that support the relatively aggressive population growth rates used in the plan. Additionally, the City directed the use of these population growth rates to reflect the potential for growth in the City and to be consistent with the population projections used in the planning of other City facilities.

Though we agree that unusually high growth may be controversial, recent growth rates indicate that the City must be prepared for the scenario of continued strong population growth. Though it is certainly not good practice to overestimate the population projections of a community, underestimating future populations could have serious consequences on a community's planning and funding strategies, as well as creating the potential for negative impacts on the environment.

The previous population projections shown in Table 2-2 were provided for information only since each of these projections has limited value with respect to the current population projections. As such, these were not illustrated graphically, but Figure 2-4 was expanded to show the previous 20 years of historical population.

Historical and current populations were obtained from Portland State University (PSU) Population Research and Census Center. Growth rates were calculated from the historical and current populations.

The discussion on page 2-4 is shown in Table 2-2. Since these previous population projections have limited value with respect to the current population projections, they do not warrant graphical illustration beyond what is already provided.

Section 3.1, Wastewater Flow Projections

Dry weather and wet weather months are defined in subsection 3.1.1. A subsection of Appendix A was added and includes a tabulation of the monthly flow data for 1996 to 1998. The per capita average dry weather flow of 159 gpcd is not extremely high compared to other communities west of the Cascades with older collection systems. This is particularly true during the past few years of relatively wet weather.

The somewhat high per capita average dry weather flow is not due to a wet industry, but is likely due to a relatively old collection system and a high groundwater table. The population figures are not likely to be low.

We maintain that the per capita average dry weather flow for the City is not unusual and therefore need not be evaluated in great detail.

As discussed in the plan, the extreme influent flows of February 1996 were not included in the flow projection evaluation. Not only are these flows representative of a storm beyond the required 5-year design storm, but the influent flows exceeded the measurement capability of the current headworks. The 1996 flows, therefore, did not contribute to the 10.2 mgd peak instantaneous flow (PIF) estimation.

The statement was made that unit per capita flows can be applied to flow projections for dry weather flows, but not for wet weather. Though we agree that the wet weather flows will not increase directly proportional to the population increase, the wet weather flows for this application will increase by more than just the additional domestic component. This is because as the City grows, the collection system will expand and allow for additional sources of I/I.

The future tighter collection system sections will not have the extent of I/I as does the existing older system sections, but will have some measure of I/I. As a result, reduced per capita flows were applied to the future population as discussed in the plan. Another method of calculating flow projections was performed and is included in a subsection of Appendix A, which has similar calculated flow projections.

A discussion of the existing influent BOD/TSS ratio was added to the plan.

As discussed above, another method of calculating the flow projections was performed and is included in a subsection of Appendix A. These calculations give additional credibility to the original flow projections.

Chapter 4, Regulations

The City's current NPDES permit was added to Appendix F. A map showing the location of the City relative to the Pudding River and the Molalla River was added in Figure 4-1.

The plan states that since the Molalla River and the Pudding River are listed by DEQ as water quality limited, more stringent regulation than the basin standards may apply.

A water quality analysis of the impact of the proposed discharge on the Molalla River was added to Appendix G.

The water quality analysis added to Appendix G includes a dilution analysis and a temperature analysis. Maps and pictures of the river are also included.

As stated above, a water quality analysis was added.

The plan assumes a permit requirement of 10/10 effluent, as stated in Table 7-2.

Text was added that indicates DEQ will not allow sewage treatment plants to accept septage without having special equipment and tankage design for septage receiving and handling. It was assumed for this plan that the City will not provide such facilities.

Chapter 5, Existing Facilities

This chapter will not be rearranged as suggested; having the existing facilities in a separate chapter helps the organization of the plan and keeps the chapters shorter. It may appear that the collection system discussion is too light, but the relatively short discussion indicates that there are no existing major problems with the system except for the high levels of I/I.

A description of the pump stations was not included in the scope for this plan. These pump stations are relatively small and do not have any existing capacity or maintenance problems. There are no private or non-municipal pump stations discharging to the City sewer.

As stated above, this chapter will not be rearranged.

The information in Table 5-1 was expanded to include most of the items suggested.

As stated above, the information in Table 5-1 was expanded.

As suggested, more information on the existing Bear Creek stream gauge was provided in the plan.

The failure of the existing meter is irrelevant to the need to replace the gauge.

The influent sewer manhole upstream from the headworks does not have a history of overflow. The 5.25 mgd capacity of this influent sewer is stated as the full flow capacity; surcharging in the line would allow for additional capacity and, therefore, higher influent flows.

A flood study has not been performed on the relatively small Bear Creek in the vicinity of the treatment plant. There are no manholes along the existing outfall line to the creek.

The suggestions for treatment plant deficiencies were not added because they are either not true or not considered deficiencies.

The plan has been modified to clearly explain that no disinfection has occurred on flows that were discharged to Bear Creek since bacteria standards for discharge have been met without disinfection.

Chapter 6, Collection System

As stated above, this chapter will not be rearranged.

It was determined that the units used to show the measured flow of gpd/in.dia.-ft. provides values that are easy to compare.

It would be extremely difficult and expensive to determine the relative contributions of inflow and infiltration throughout the system. The goal of the I/I analysis was to determine the areas with the worst I/I and then invest further time and money only on the areas that have the worst I/I in the proposed I/I reduction program.

As discussed above, it is difficult and expensive to determine the relative contributions of inflow and infiltration throughout the system.

As discussed above, it is difficult and expensive to determine the relative contributions of inflow and infiltration throughout the system.

Chapter 7, Treatment

The alternative to add irrigation that was briefly discussed and dismissed was to address an alternative that was included in the 1996 draft plan. This alternative included only the improvements indicated. This alternative was dismissed because it is not adequate to address the improvement requirements for the plant. Adding irrigation as a part of the overall solution was not dismissed, as indicated by the effluent disposal section later in the chapter. To avoid further confusion, the original alternative was deleted from the plan.

Similar to the preceding paragraph, the alternative to add lagoon storage represents an inadequate solution to the improvement requirements for the plant. Adding lagoon storage as a part of the overall solution was not dismissed. To avoid further confusion, this original alternative was also deleted from the plan.

The alternative labeled "expand the existing plant using similar treatment processes" was relabeled to "upgrade the existing plant treatment processes". This alternative is the recommended alternative and a schematic and tabulation of the design basis are included in the plan.

The other treatment alternatives are described in the plan and detailed cost estimates are included in Appendix C. Flow schematics for these alternatives are not included since they are not the recommended alternative.

Though a hydrogen sulfide analysis need not be performed for intermediate plant pump stations, the recommended upsizing of the pump station forcemain warranted this analysis to determine if hydrogen sulfide production should be expected.

Because DEQ is not likely to approve the cyclic feed aeration alternative due to its limited installations in North America, it was deleted from the plan.

The assumed effluent requirements are included in Table 7-2. Each alternative was developed based on these assumptions.

- The oxidation ditch alternative with secondary clarifiers can reliably meet 10/10 effluent without filters.
- The alternative to upgrade the existing plant treatment processes will have two phases. Phase 1 will be an aerated basin that will require advanced treatment of the effluent to meet 10/10. Phase 2 will add secondary clarifiers and RAS/WAS pumping that will convert the system to an extended aeration activated sludge system that can reliably meet 10/10 effluent without filters.
- The SBR would be designed to reliably meet 10/10 effluent.
- As discussed above, the cyclic feed aeration alternative was deleted from the plan.
- As discussed above, the cyclic feed aeration alternative was deleted from the plan.

- Detailed cost estimates are provided for all alternatives. Detailed design criteria is provided only for the recommended alternative, the upgrade of the existing plant treatment processes.

The annual costs were revised; those presented in the draft plan included annual costs for the secondary treatment facilities only and were low as a result. Costs for alum and polymer are included in the maintenance/replacement costs.

The text was revised to indicate that a 4-foot water cap may be required on the facultative sludge lagoon to reduce odors. Since the limiting factor in the sizing of the sludge lagoon was surface area, the depth of the water cap will only impact the sludge storage volume.

Section 7.5, Effluent Disposal

This section was given its own chapter, the updated Chapter 8. The discussion on Bear Creek was not expanded since the capacity of Bear Creek is very limited. Also, at a meeting on September 22, 1998 attended by David Mann and Lyle Christensen (DEQ), Dean Madison and Dennis St. Clair (City), and Jerry Minor (Tt/KCM), DEQ indicated that no additional loading to the creek would be allowed. Because of these reasons, additional discharge to Bear Creek was not considered.

- Flows in the creek are not readily available for graphing.
- Identification of a 7Q10 flow requires a statistical analysis of several years of creek flow. The necessary flow information is not available.
- The potential mass load limitations on the creek (and its receiving stream, the Pudding River) support the assumption that no additional loading to the creek will be allowed.
- The City currently discharges effluent better than 5/5 (see Chapter 5) and cannot meet the discharge required to keep up with existing influent flows.

A figure illustrating the drainage area upstream of the proposed discharge was added to Appendix G. It is extremely unlikely the Army Corps of Engineers would allow the addition of a flow measurement structure in the river in the vicinity of the proposed discharge. It is possible, however, that some other means of approximating flows in the vicinity of the proposed discharge, particularly low river flows, could be accomplished. This possibility is discussed in Appendix G.

The water quality analysis added to Appendix G includes an oxygen-sag analysis for the assumed effluent quality of 10/10 indicated in Table 7-2. A temperature analysis is also included in the water quality analysis.

Plant effluent flows, temperature, solids, and BOD are recorded at the plant. Chlorine residual measurement downstream of dechlorination can be added if necessary. River temperatures are recorded at the water treatment plant intake just upstream of the proposed discharge. River flow measurements at the proposed discharge would require a new gauging station; as stated above, it is not likely that the Army Corps would allow such a station due to the negative impacts on fish.

Table 8-1 was expanded to show the Molalla River flow at of the proposed discharge in mgd.

Figure 8-1 was added to show the Coleman Ranch where effluent irrigation will occur. The storage facilities will likely be located on property adjacent to the plant. Discharge flow metering, sampling, and dechlorination are shown on Figure 8-1. Table 8-3 shows the design criteria for the land application area and storage lagoons including phasing.

Section 7.6, Treatment Recommendation

This section summarizes the recommendations from the rest of the chapter and provides additional design criteria detail. It will remain in this chapter from which it was built.

The control building and office that were remote from the main treatment area in the draft have been removed. Due to budget considerations, the existing building facilities will be used as much as possible and any additional control buildings required will be added at the north end of the site.

For the process schematic in Figure 7-2:

- Showing the collection system manhole on the process schematic will not add any information to the schematic.
- Since this is a schematic only, the view (plan or section) that best represents the process was used.
- The type of fine screen is listed in Table 7-8.
- Future (Phase 2) flow streams additions are clearly marked as dashed lines throughout the schematic.
- Because of the mixing provided by the effluent weir, a chlorine flash mixer is not required.
- An effluent sampler was added upstream of the dechlorination chemical addition. Dechlorination will only occur when flows are discharged to the river.
- A chlorine residual analyzer will be provided on the river discharges downstream of the dechlorination chemical addition.
- Grab samples can be taken downstream of the re-aeration facilities to measure dissolved oxygen and temperature.
- The pump station will be referred to as the transfer pump station.

The fine screen is recommended to be a rotary-type screen with washing and compaction.

A detailed annual cost spreadsheet is included in Appendix C.

The current class for the existing operators has been added in Chapter 5.

Chapter 8, Immediate Improvements (updated Chapter 9)

This chapter was updated to reflect any changes in the previous chapters. This chapter was also modified to reflect projects performed since the draft.

The purpose of the lagoon piping improvements is to allow the operators to pull from multiple levels in the lagoon using a flexible pipe. This would provide even more flexibility than a floating drawoff.

The $14,000 costs for the dechlorination facilities assume reuse of equipment originally provided for chemical feed upstream of the DAF unit. Since alum is now the only chemical added, there is chemical feed equipment that can be used for dechlorination.

Chapters 9 and 10, CIP and Funding (updated Chapters 10 and 11)

The CIP and funding chapters were updated to reflect any changes in the previous chapters. These chapters were also modified to reflect planning and projects initiated since the draft.

Oregon
John A. Kitzhaber, M.D., Governor

Department of Environmental Quality
Northwest Region
2020 SW Fourth Avenue
Suite 400
Portland, OR 97201-4987
(503) 229-5263 Voice
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September 24, 1999

Dean Madison, Public Works Director
City of Molalla
PO Box 248
Molalla, OR 97038

RECEIVED
OCT 0 4 1999
PORTLAND OFFICE
TETRA TECH/KCM, INC.

Re: WQ - Molalla
Review of Draft Facilities Plan
File 57613
Clackamas County

Dear Dean Madison:

Thank you for the opportunity to review the April 1999 draft wastewater facilities plan, which we received May 3 from Kay S. Swartwood PE of KCM, Inc. We appreciate the city's efforts, through commissioning this report, to develop an orderly plan for managing wastewater collection, treatment, and discharge.

We have enclosed detailed comments in the form of an attachment to this letter. Please have KCM make the changes noted and resubmit a final report. To expedite approval, all of the comments should be addressed. If there are any questions about our comments or requested back-up materials, please do not hesitate to call (229-5310).

In connection with our review of the final report and preliminary design work, we will be required to charge a technical activities fee as established in OAR 340-45-075(3). The total amount of these review fees will be based on the scale and number of projects that develop from the planning effort.

We estimate that the review fees will probably total approximately $6000. During your pre-design phase for each project component, we will invoice the city for the correct amount. However, if you have not done so, a $6000 allowance for review fees should be budgeted.

Respectfully,

David S. Mann, P.E.
Senior Environmental Engineer
Northwest Region

Attachment: Review Comments
cc:
Kay S. Swartwood PE, KCM, Inc. 7080 SW Fir Loop, Portland, OR 97223-8022
Richard Santner, SRF Project Officer, NWR
Lyle Christiansen, NWR
Robert P. Baumgartner, NWR
ecc:
DEQ staff engineers

REVIEW COMMENTS ON DRAFT WASTEWATER FACILITIES PLAN
City of Molalla
September 24, 1999

Format:

ADD tabbed dividers for each chapter and each appendix.

Bind copies of any facilities plan being submitted to DEQ so as to provide a spine label. A 3-hole binder format is preferred to facilitate making copies and revisions, and to make the requirement of a spine label easier.

Two copies should be submitted to DEQ. Submittals should be directed to the attention of Molalla's compliance officer, Lyle Christensen.

Chapter 2.3, Population:

We are somewhat concerned about long-term impacts which may come from unrealized growth projections at 7%, and grossly oversized facilities. If the selected growth rate turns out to be unrealistic, we would be concerned that it could impact project phasing, revenues, and sewer rates.

These projections, being the basis for the scale of planning in the rest of the report, must be made easier to visualize and understand. Unusually high growth may be controversial, and must be well defended. Perhaps some ramifications of these projections for funding, permit limits, disposal options, storage volumes, etc. should be listed and discussed.

To this end, add further graphical illustration of the various growth scenarios that are being considered and rejected. Also add graph showing the design population basis compared to past growth. A projection of 20 years ahead should be balanced with at least 20 years historical perspective in the same graph.

Graphically illustrate the various growth scenarios of Table 2-2. Explain how the city determined that its growth is now 9% or 7%. Was there a census? How does the city keep track of current population?

The discussion on page 2-4 also needs to be shown in graphic form. This could perhaps involve extending Figure 2-4 to 11x17, adding 1980 and 1985 data, and showing the various rates from Table 2-2. What would those rates look like?

Chapter 3, Flow Projections

Table 3-1, add note defining dry-weather and wet-weather months. Add reference to an appendix where the monthly flow data for 1996-1998 are tabulated. Add discussion explaining the extremely high per capita dry weather flow of 150 gpcd, compared with a normal flow of about 100 gpcd.

Is this 50% extra flow due to a wet industry? To a sustained high groundwater table? Does it come mainly from the unsealed manhole bases in the deep sewer along the highway? Does the flow remain steady all night? Could the population figures be way low?

One place to start this discussion might be to obtain copies of 1998 plant flow charts and observe what minimum hourly flows were recorded during 3-4am from June through October. Dry weather and significant rains should be noted. Copies of significant and representative charts should be added to an appendix for reference.

Delete 1996 flow data from the determination of current flows here and in Appendix A, particularly Figure A-4. The 1996 data are not representative and result in the exaggerated current estimate for PIF5 of 10.2 MGD. 1998 data better reflect the current status of the collection system and should be used. If the PDAF5 is 5 MGD, from Figure A-2, then PIF5 might be as high as 7 MGD. This is where the 1998 data point in Figure A-4.

Page 3-3, delete last sentence. Unit per capita flows can be applied to flow projections for dry weather flows, but not for wet-weather. Correct the errors introduced into the maximum month and peak design flows through increasing these flows parallel to population.

Only the average flows and the base-flow component of peak flows increase in direct proportion to the number of users. The 5-year and 10-year storm intensities, derived from several decades of data, are constant or appear to be constant in the brief 20-year horizon of facilities plans. Hence the stormflow portions of MMWWF5 and MMDWF10, PDAF5, and PIF5 do not increase with population.

Bottom of page 3-6, is this unusual influent BOD/TSS ratio of .23/.16 = 1.4:1 consistent year round? For many years, or recent? Add a discussion of background, significance and causes. Is there some high-BOD contributor such as a brewery or a dairy? Is this BOD a high-carbohydrate component which might have treatability problems? Does the collection system contain STEP installations? This ratio is too peculiar go unchecked in a facility plan.

Table 3-7, after adjusting down the summary of design flows for 1999 and the future as previously indicated, append sample calculations showing how base flow conversions to peak flow were projected.

Chapter 4, Regulations

Add a copy of the current NPDES permit to an appendix or to this chapter. Add a map or aerial photo showing the location of the plant site relative to Bear Creek, the Pudding, and the North Fork Molalla.

Add further explanation of the ramifications of Table 4-1. For example, add words to the effect that no additional loadings to Bear Creek, and no new loadings to the Molalla, can be permitted without scientifically persuasive evidence that these loadings will not impair beneficial uses or the aquatic environment.

Either here or in the chapter on effluent disposal, also discuss various approaches which the development of such evidence may require, including field work where data may be lacking. If the final facility plan determines that a new winter-only discharge to the Molalla is desirable and economical, then the facilities plan must be expanded to include development of the evidence and documentation of the analysis.

Since the Molalla in winter remains limited with respect to temperature, it will be necessary to analyze for effluent temperature and minimum dilution in the proposed mixing zone that would need to be achieved by any proposed diffuser. The design of the diffuser can be postponed to a predesign report, but the proposed outfall location and mixing zone must be described in the facilities plan. Provide a topographic map, plan view and river crossection, and selected photographs of the proposed outfall location.

With sufficient information documenting the proposed discharge to be safe and harmless to the Molalla river, DEQ could then support permitting the discharge in public hearings and before the Environmental Quality Commission. However, we cannot approve a facilities plan that relies on this solution unless the facilities plan provides the supporting documentation.

As part of this material, the plan must identify the outfall location and determine its adequacy. Winter river flows should be analyzed for dilution availability. If the mixing zone and diffuser can be designed to provide a minimum 20:1 dilution without temperature limitations, then effluent quality in the range of 20/20 might be feasible. Add a determination whether dilution in the Molalla in normal water years at the proposed outfall may warrant effluent quality of 20/20, instead of 5/5 as assumed in Chapter 7 cost estimates.

Section 4.4, Septage: Add that sewage treatment plants are not allowed to accept septage without having special equipment and tankage designed for septage receiving and handling. Explain that plants which may have the capacity to treat septage, but lack approved special facilities for handling it in a manner to reliably prevent plant upsets, cannot accept it.

Chapter 5, Existing Facilities. Change this chapter to "Collection System."

This whole chapter should be devoted to the collection system, bringing in the entire Chapter 6. The collection system description here is too light.

5.2.2, add complete description and field evaluation of both pump stations per DEQ guidelines. Also identify any private or non-municipal sewage pump stations discharging to the city sewer, including industrial parks and manufacturing plants, schools, trailer parks, condominiums, shopping centers, etc.

Move the existing treatment plant description out of Chapter 5 and combine it with the contents of Chapter 7.

On Table 5-1, add design flow and design population. Add influent sampler type, whether flow-paced or on a timer. Add standby power and alarm telemetry. In aeration basin section, add liner type. In pump station section, change "redundant" to "firm." In lagoon sections, add freeboard and minimum depth for lagoons. Note whether the transfer piping is adjustable-depth, fixed open pipe, valved pipe on the floor similar to Figure 5-3, or what? Add outlet pipe diameter.

DAF: add hydraulic thoughput capacity and design TSS effluent quality. Filters, add media type and depth. Disinfection section, add: 150 lb gas, 100 ppd chlorinator, flash mixing - none, contact basin L:W ratio. Outfall, add pipe material and effluent diffuser type and description. Irrigation section, add pump HP. Add irrigation area available on site. Add average irrigation in MGD applied during the summer. Add adjacent irrigation area and gas-driven pump capacity.

Add a stream gauge crosssection, and add diagram of the gauge showing dimensions. Add copy of the gauge curve along with an evaluation of accuracy of the gauge. Does the gauge contain a critical-flow section? A uniform flow section? When was the velocity measured to verify the curve that operators use to estimate the flow in Bear Creek?

Describe what components of the original gauge from 1980 failed and made it inoperable. As this gauge was paid with grant funds, its fate is a point of interest.

Figure 5-2, add the influent sewer manhole upstream from the headworks. Regarding this sewer, what is its overflow history and where is the low point that overflow may be expected?. If this sewer's capacity is limited to 5.25 MGD as stated, and if the current PIF5 is about 7 MGD per Figure A-4, how could the influent sewer not overflow?

Indicate 100-year and normal water elevations. Show any manholes in the outfall line. Is there a

manhole on the bank of the creek? If not, how does the line vent?

5.3.11, Deficiencies: add lack of flow-pacing for influent sampling and effluent chlorination; lack of chlorine flash-mixing for viral disinfection; lack of adjustable drawoff piping, causing solids carry-over and transfer between lagoons, thus overloading the DAF.

5.4.3, Deficiencies: add effluent toxicity due to extreme maximum chlorine residuals. The effluent has likely been toxifying Bear Creek for many years.

Chapter 6, Collection System

This whole chapter should be moved into Chapter 5.

Table 6-1, the unit flows should be given in the standard "inch/diameter/mile", so as to facilitate review. Multiply all gpd data by 5280 and round off.

Section 6.2.2, distinguish the relative contributions of inflow and infiltration. Use selected photocopies of plant influent flow charts that indicate whether there is any significant flow response to the heaviest rainfalls. Add flow charts from heavy rains during September-October, 1997 and 1998. Use the flow charts and rainfall records during heavy dry-season storms to estimate inflow rates. Inflow should be expressed as MGD per inch of rain.

Also add copies of selected charts showing rainfall-flow response when groundwater levels are high during January-February 1997, 1998, and perhaps 1999. Use the charts to estimate the infiltration component and distinguish it from base flow when groundwater is high during January - March. Maximum infiltration should be expressed in the standard gallons per day per inch-diameter-mile.

The conclusion at the bottom of page 6-5 regarding "I/I" may be correct with respect to uncurable infiltration, but it needs better documentation with respect to unremovable inflows.

Chapter 7, Treatment.

Page 7-4, adding irrigation: This alternative was dismissed too fast. The study must identify and evaluate potential irrigation land beyond the current 70-acre site. An effort must be made to fly or drive the surrounding lands, map the farms and crops, and evaluate the costs for acquiring the most feasible locations. As Alternative 1, the study must seriously evaluate the costs of a treatment expansion consistent with a phased program of extensive summer irrigation, together with continued winter discharge to Bear Creek at a 5/5 level of effluent quality. Add a schematic diagram of the process and tabulate the sizing of unit processes, lagoons, and irrigation areas that were used to develop the cost estimate.

Page 7-5, adding lagoon storage: This 2nd feasible alternative was also dismissed without considering expanded irrigation in conjunction more storage. The alternative should evaluate expanding the lagoons for additional winter storage, while also expanding the irrigation area through purchase of farmlands. Add a schematic diagram showing what is being considered. Add tabulation of unit sizes and areas that were used to estimate costs.

Page 7-6, change "similar processes" to the more explicitly descriptive "expand aerated lagoon system", or words to that effect. This 3rd feasible alternative would include new effluent storage and irrigation. Part of this would involve expanded DAF and filters for treatment of lagooned effluents, with winter discharge to the Molalla at perhaps a 10/10 or 20/20 effluent quality, and to Bear Creek at 5/5. Add a schematic diagram showing how the alternative would work. Add tabulation of the unit sizes and areas that formed the basis of the cost estimate.

Effectiveness and effluent criteria must be developed to assess the viability of the other 3 processes: oxidation ditch, SBR, and cyclic feed. Sizing and scale for these processes needs to be presented here or in the appendix with the cost estimates. How each of these processes would work, and how it would fit with storage-irrigation-discharge,must be illustrated with a basic flow schematic.

Page 7-10: at wastewater temperatures that prevail in Oregon, H2S is a consideration for discharge to gravity sewers and long outfall lines to surface waters, but generally not for intermediate plant pump stations. The discussion regarding H2S from this intermediate in-plant pump station is not applicable to the design and should be deleted.

Top of page 7-24, change "Pacific Northwest" to "North America." Until the cyclic feed process is better accepted among engineers in the US and Canada, we regard it as developmental. However, it is incorrect that DEQ requires established treatment processes to be re-proved in Oregon before acceptance. Edit page 7-24 accordingly.

Page 7-25, revise the expected BOD & TSS effluent quality that can reliably be produced from each alternative at MMWWF and edit the text accordingly. The effluents they produce are not comparable or equivalent:

- The simple one-circuit oxidation ditch is reliable for 15/15 to 20/20, depending on the clarifiers. Add inner selector circuit to ensure 10/10 without filters.
- The aerated lagoon alternative, a version of extended aeration, cannot be relied on for better than 20/20 in a consistent manner. Add DAF plus filtration to ensure 5/5.
- Of the alternatives described, only the SBR could reliably produce 10/10 without filters. The SBR would need to be deep and large (18-20' deep with 18-20 hours detention at

MMWWF).

- The Cyclic Feed alternative is interesting but its expected performance is unknown, due to a lack of published information and lack of performance data. To include it in the facilities plan as an option will require a better description. Add citations from the literature. If they are none in English, but results have been published in Spanish or German, add those citations. Provide a copy of flow schematic, profile, and a feasible site layout, along with the unit sizing and quantity takeoff that were used for the cost estimate. Submit manufacturer's descriptive materials and installation list.
- Since Cyclic Feed has not been demonstrated in the US, the design of the process, equipment selection, and construction could be a challenge. The low cost estimate suggests that the design basis for the estimate should be checked, reviewed, and expanded in some detail to see what may have been left out. This should all be documented in an appendix. Otherwise the process cannot be considered a viable alternative for Molalla. If not, it should probably be deleted from the list of alternatives.
- Provide one copy of the design unit sizing criteria and quantity takeoff used for estimating the SBR cost. These data are needed to verify adequate sizing of the facilities being costed out and compared. Also provide copies of the oxidation ditch and aerated lagoon unit sizing and quantity takeoffs, along with a copy of cost estimate calculations.

Based on this information, we can review the cost estimates and comparison of alternatives for accuracy. Meanwhile the operations costs appear to be optimistic and should be revisited. Add costs for alum and polymer with the DAF system. Add costs for operation of the treatment facilities concurrent with irrigation. The labor costs do not appear to include any overhead or training for two certified operators.

Page 7-27, change water cap from 3' to 4'. Where there is great likelihood of H2S being produced, as from anaerobic or raw sludge, a 4' watercap is prudent. An operator might try 3' if desired, but 4' should be reserved for watercap on raw sludge when sizing the FSL.

Page 7-32 Effluent Disposal

This section addresses a variety of complex issues and warrants its own chapter. In it, expand the discussion on Bear Creek. The potential for additional discharge to Bear Creek was dismissed without serious consideration.

- Add graph showing the 1997, 1998, and 1999 winter flows (November - May) in the creek.
- Identify the winter 7Q10.
- Describe potential winter mass load limitations based on creek environment, flows, dilution, and temperature.

- Based on an expanded mass load which would not damage the creek, identify discharge volumes that could be sustained at 5/5 effluent quality, through the use of DAF plus filtration.

Expand the discussion for Molalla River discharge as previously noted in comments concerning Chapter 4, Add a figure illustrating the 64% gauged area upstream from Molalla, in relation to the location of the USGS gauge. Determine whether it is feasible to install a new gauge at the proposed outfall location. Estimate costs for the gauge and signal telemetry.

Perform an oxygen-sag analysis based on 10/10 and 20/20 effluent qualities. Determine probably temperature of lagooned effluents and of the river. Project the likely temperature interactions and identify any problems that could arise with respect to the Molalla River's wintertime temperatures if effluents were discharged into the river.

Describe and cost a feasible monitoring system for measuring and recording effluent flows, temperature, dechlorination, solids, and BOD, along with river flows and temperatures.

Page 7-34, Table 7-9, change middle column showing Molalla River flow upstream of town from units of cfs to cfs/MGD.

Page 7-36, add map showing potential poplar irrigation lands which could be acquired by the city. Show desired locations and sizing for irrigation storage ponds, discharge flow metering, sampling, and dechlorination. Note required areas, based on 32"/year poplar uptake. Address potential for phasing these improvements.

Page 7-38, Treatment Plant

This section amounts to a preliminary design. The level of detail here warrants its own chapter.

The location of the control building and office remote from the main treatment works does not enable optimum operation or maintenance. If surveillance and control of plant access is desired, consider motion sensors, video, a gate opener, etc.

Figure 7-2, Process Schematic:

- Show influent sewer low manhole and emergency overflow for extreme storms exceeding the capacity of the headworks.
- Heavy up main flow lines for legibility. Also for clarity maintain consistent view. Grit basins, flumes, clarifiers, and filters are shown in plan view per standard practice, which best shows the number of units. Convert other views accordingly.

- Identify type of fine screen. Is this a sidehill, climber, rotary, or what?
- For clarity, let this figure show Phase 1 system only. Add a separate flowsheet for the future (Phase 2) system.
- Add chlorination flash-mixer for viral kill.
- Move bisulfite addition below effluent sampler. Wastewater applied to public-contact sites should not be dechlorinated. Normally 1 mg/l chlorine residual needs to remain at the point of application.
- Insert chlorine residual samplers/recorders on the public contact irrigation line and on both river discharges.
- Insert DO and temperature meter/recorder at the end of the reaeration channel.
- Use the same name for influent/transfer pump station or "headworks lift station" on the schematics and the profile.

Table 7-10: define screen type (rotary drum, climber, etc.) Due to maintenance problems, we no longer approve sidehill screens at headworks.

Page 7-43, Table 7-11:

- Add a table 7-12 with breakdown for chemicals, labor, training, electricity, supplies, maintenance, etc. Indicate the assumed operator class on which this estimate is based.
- Note current class of existing operator(s)

Chapter 8, Interim:

- We concur with these interim recommendations, but the section needs to be edited to reflect previous comments.
- Regarding Section 8.3.4, note that Lagoon 2 effluent piping should be fitted with an adjustable floating drawoff. This would be similar to those designed for Mt. Angel, Warrenton, Carlton, and Yamhill. The drawoff pipe should be set about 20" to 24" below the surface to retain algae in the lagoon.
- Regarding Section 8.3.5, the supplier's cost estimate of $14,000 is too low. Something may have been omitted. Add costs for a chlorine residual analyzer/recorder and for electrical. Costs for an approvable dechlorination station are running about $50,000, as a budget estimate. This could be reduced by the value of the city's available equipment and storage.

Chapters 9 & 10

As a general comment, revised flow projections, sizing, and phasing will impact the scale of new facilities, the costs involved, and affordability aspects. Chapters 9 and 10 must be edited to reflect the approved design basis and analysis.