WASTEWATER TREATMENT PLANT
FACILITIES PLAN UPDATE

Prepared for the

City of Newberg, Oregon

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BROWN AND CALDWELL

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AAF	average annual flow
AD	average day
ADWF	average dry weather flow
ASP	aerated static pile
AWWF	average wet weather flow
BFP	belt filter press
BOD	biochemical oxygen demand
C	Celsius
CBOD5	5-day carbonaceous biochemical oxygen demand
CIP	Capital Improvement Program
CMOM	Capacity, Management, Operations and Maintenance
DEQ	Oregon Department of Environmental Quality
DI	ductile iron
DNA	deoxyribonucleic acid
EPA	Environmental Protection Agency
EQC	Oregon Environmental Quality Commission
ETL	excess thermal load
F	Fahrenheit
FTE	full-time equivelant
gcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
gsfd	gallons per square foot per day
HI	Hydraulic Institute
HOCl	hypochlorite
hp	horsepower
IPS	Influent Pump Station
I/I	infiltration/inflow
LUC	land use compatibility
MBR	membrane bioreactor
MCC	motor control center
MD	maximum day
MG	million gallons
mg/kg	milligrams per kilogram
mg/L	milligrams per liter
mgd	million gallons per day
mL	milliliters
MLSS	mixed liquor suspended solids
mm	millimeters
MMDWF	maximum month dry weather flow
MMWWF	maximum month wet weather flow
NPDES	National Pollutant Discharge Elimination System
OAR	Oregon Administrative Rules
O&M	operations and oaintenance
OHD	Oregon Health Division
pcd	pounds per capita per day
PDF	peak day flow
PHF	peak hour flow
PLC	programmable logic controller
POTWs	Publicly Owned Treatment Works
ppd	pounds per day
pph	pounds per hour
psfd	pounds per square foot per day
psi	pounds per square inch
PWF	peak week flow
RAS	return activated sludge
RRE	Repair, Renovation and Expansion
SMPU	Sewerage Master Plan Update
SRT	solids retention time
SSOs	sanitary sewer overflows
SST	sludge storage tank
TKN	total Kjeldahl nitrogen
TMDLs	total maximum daily loads
TPAD	temperature-phased anaerobic digestion
TS	total solids
TSS	total suspended solids
UGB	urban growth boundary
URA	urban reserve area
UV	ultraviolet
VLR	vertical loop reactor
WAS	waste activated sludge
WET	whole effluent toxicity
WLA	waste load allocation
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant
ZID	zone of initial dilution
7Q10	annual flow for seven consecutive days that has a recurrence interval of 10 years

EXECUTIVE SUMMARY

ES.1 OVERVIEW

In 2004, the City of Newberg (City) contracted with Brown and Caldwell to develop a Facilities Plan to provide recommendations to the City's Capital Improvements Program (CIP) for the City's wastewater treatment plant (WWTP). The purpose of the Newberg WWTP Facilities Plan Update is to provide the planning for required modifications to meet projected growth within the urban growth boundary (UGB) and the urban reserve area (URA) to maintain compliance with its National Pollutant Discharge Elimination System (NPDES) permit and potential future regulations.

ES.1.1 Wastewater Service

The City owns and operates a secondary WWTP located at 2301 Wynooski Road, Newberg, Oregon. The City currently provides wastewater collection and treatment services to its residents, commercial establishments, institutional customers, and a number of industries. Sewer service is provided only to customers within the city limits, with the exception of a few residences outside of the city and SP Newsprint Company, which discharges only domestic wastewater to the municipal system.

ES.1.2 WWTP Description

The Newberg WWTP was placed into service in 1987. The facility is a Class IV oxidation-ditch type, activated sludge plant with Class A in-vessel biosolids composting. The treatment train consists of influent pumping, screening and grit removal, oxidation-ditch activated sludge, clarification, solids dewatering, composting, odor control, chlorination, dechlorination, and effluent discharge to the Willamette River. A schematic of the treatment train is shown in Figure ES-1.

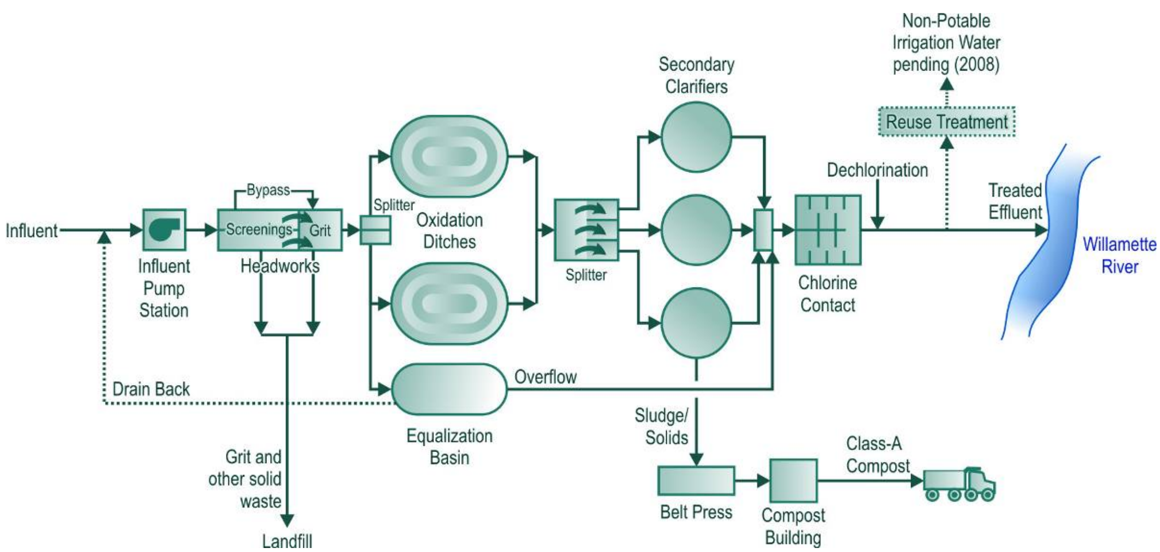


Figure ES-1. Schematic of Newberg WWTP

As shown in Figure ES-2, the two main products that result from the City's wastewater treatment process are water and compost. The WWTP discharges treated water to the Willamette River, and the City is implementing plans to irrigate golf courses with some of the treated product (reuse water) by October 2008. The City sells its NEWGROW compost product to the public throughout the year.


Figure ES-2. Newberg WWTP Products: Water and Compost

The last Facilities Plan was completed 22 years ago as part of the Sewerage Master Plan Update (KCM, 1985), after which the City constructed the existing WWTP on Wynooski Road with federal grants. The purpose of the Newberg WWTP Facilities Plan Update is to provide the planning for required modifications to meet projected growth within the UGB and the urban reserve area to maintain compliance with its NPDES permit and potential future regulations.

ES.1.3 Factors Affecting the WWTP

There are three major factors that impact the wastewater service and the WWTP. These are:

- Ability to treat the City's wastewater to the required quality
- Ability to convey and treat the quantity of wastewater (hydraulic capacity)
- Ability to handle solids, compost, and deliver compost product to the public

Willamette River water quality requirements dictate how well the wastewater needs to be treated. There are minimum technology standards that require secondary biological treatment and disinfection prior to surface water discharge; and receiving water quality standards that protect its beneficial uses. Changes are needed immediately to improve the reliability of the disinfection process.

The WWTP must also be able to accommodate the peak hydraulic flow. Infiltration/inflow (I/I) from rainwater direct connections (inflow) and infiltration from high groundwater is responsible for the peak hydraulic flow at the Newberg WWTP. A schematic of I/I is shown in Figure ES-3. The Newberg Collection System Master Plan being prepared in parallel with the Facilities Plan confirms the I/I contribution to the peak wastewater flows needing to be accommodated.

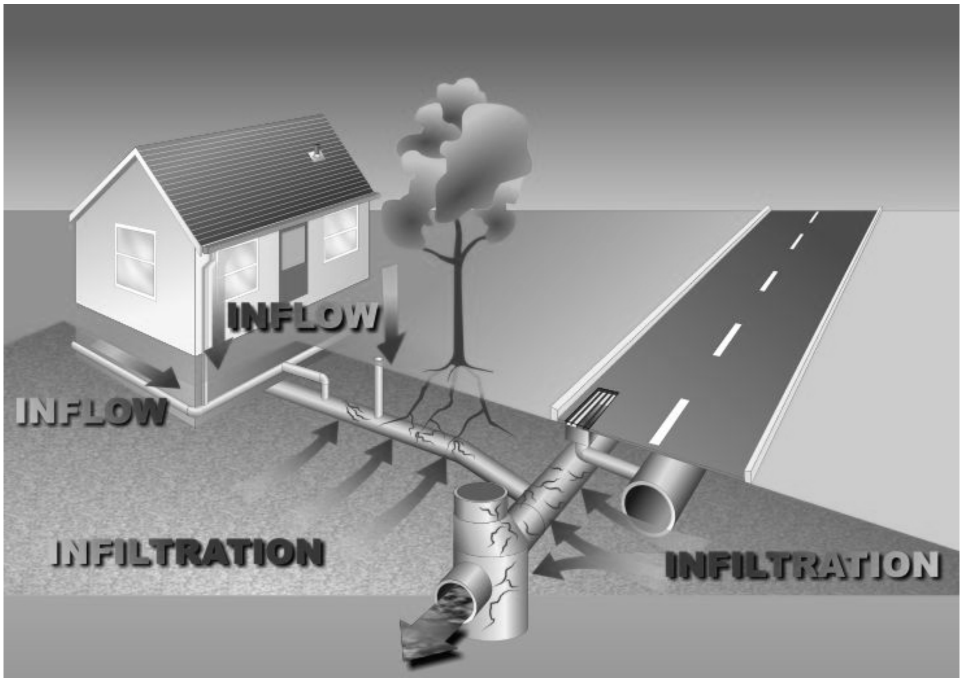


Figure ES-3. Schematic of I/I

I/I entering the collection system has a profound effect on the wastewater quantity that flows to the WWTP influent pumping station (IPS). Currently the IPS cannot convey peak flows with one pump out of service for the design storm event. The Oregon Department of Environmental Quality (DEQ) design standard for wastewater pumping requires that capacity with one pump out of service must convey the peak hour flow for a defined storm event to prevent unauthorized overflows by 2010. All downstream treatment plants must be able to convey the pumped flows.

The impacts of the Newberg I/I elimination program will affect the capacity of the WWTP Repair, Renovation, and Expansion Projects (RRE Projects). However, these impacts will not immediately reduce the first planned RRE Project scope, but will delay, reduce, and/or, postpone future project expansions. The first phase RRE is needed to convey and treat the I/I flows until collection system improvements result in decreased I/I. Reductions in I/I are not expected to occur until after the first phase RRE is implemented.

Solids handling capacity is also a critical component of the WWTP. The City composts the solids and sells the compost to the public. The composting process is currently limited and out of capacity because the moisture content in the sawdust and solids feeding the composter is too high. Recent changes in the sawdust market have resulted in the availability of only high moisture sawdust. New technology is available for drying the sawdust and removing more moisture from the WWTP solids.

ES 1.4 RRE Projects

Figure ES-4 shows the needed improvements to meet the regulatory requirements, guide the future direction of capital improvements projects, and define the land area needed for the City's wastewater treatment that will be phased through 2025.

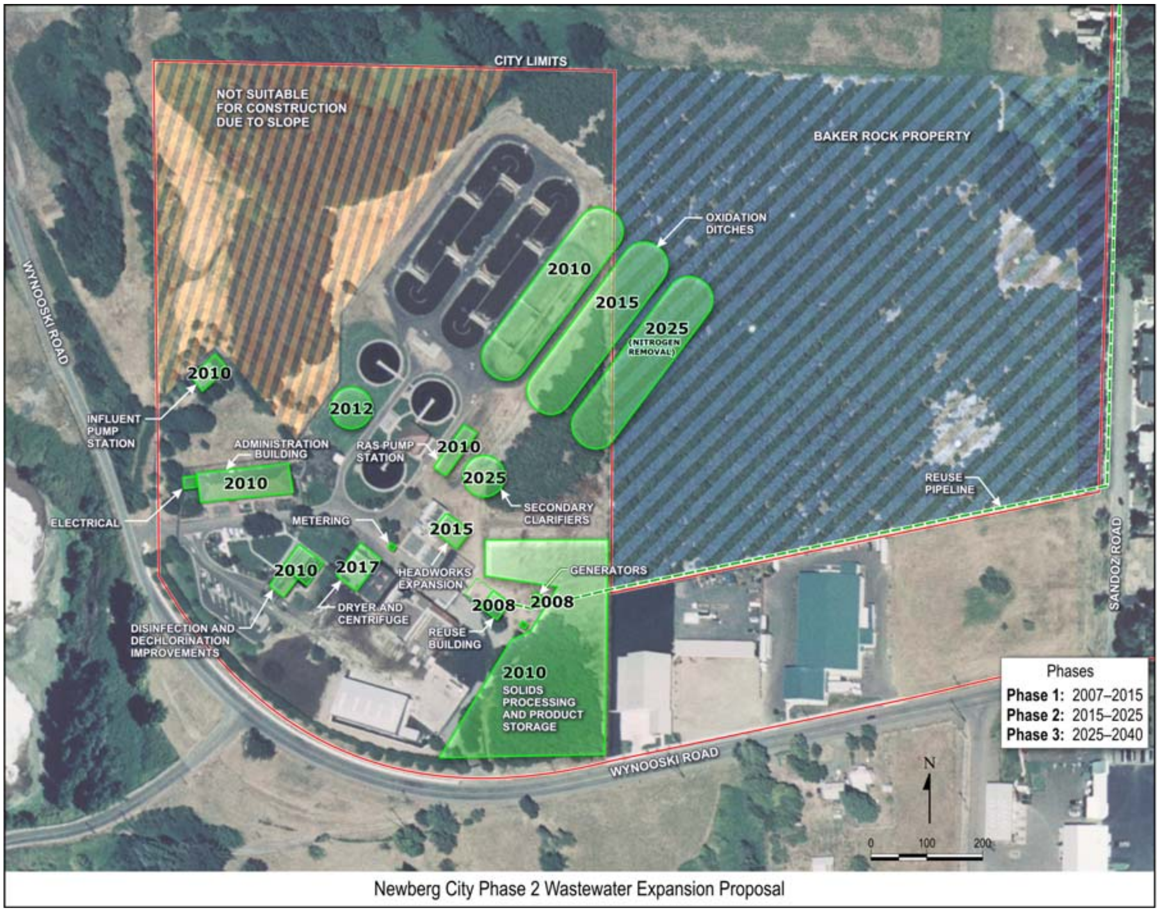


Figure ES-4. Recommended Improvements and Phasing through 2025

The unit processes that need RRE include:

- IPS
- Headworks, including screening and grit removal
- Secondary treatment, including oxidation ditches and secondary clarifiers
- Effluent disinfection and dechlorination
- Effluent conveyance and discharge to the Willamette River (outfall)
- Effluent reuse (currently planned for implementation in October 2008)
- Solids processing and handling systems, including dewatering and composting
- Administration Building
- WWTP support systems

ES.2 PROJECT GOALS

The project goals are listed below.

- Plan for facilities to comply with existing and predictable-potential-future regulations.

- Provide for incremental capacity expansion through 2025 through a CIP and ultimate expansion in 2040.
- Provide for reliability, ease of operations and maintenance (O&M), and safety.
- Plan an efficient Administration Building office space and laboratory.
- Recommend back-up power engine generator requirements.
- Coordinate with the Reuse Water Project- the Reuse Water Project includes the new back-up power engine generator.
- Evaluate staffing requirements for the existing and future WWTP operations.

ES.3 PLANNING PROCESS

The facility planning process includes the following:

- Using existing and future population projections (residential, commercial, and industrial) to estimate significant increases in wastewater flows and loads.
- Identifying capacity needs that ensure that the WWTP will be able to convey and treat the wastewater through service area build-out.
- Evaluating treatment processes with regard to growth and regulatory requirements and identifying needed plant expansions and improvements.
- Developing recommendations to modernize and optimize the system where practicable, and to ensure compliance with local, state, and federal regulations.
- Evaluating and recommending energy efficient alternatives.
- Developing planning-level cost estimates (with 35 percent contingency) for the recommended improvements based on current cost estimating practices and the timing of the improvements.
- Creating a recommended CIP for the facility in increments for the fiscal year period 2007 through 2025.
- Evaluating the Administration Building to meet current and future staff needs (including those of a new water treatment plant (WTP), to be located south of the WWTP across Wynooski Road).

ES.4 PLANNING CRITERIA

The planning criteria used to evaluate the WWTP ability to project the future wastewater quantities, treat the City's wastewater to the required quality, and provide hydraulic capacity are summarized below. The planning is consistent with the current Newberg city boundaries and the UGB as shown in Figure ES-5.

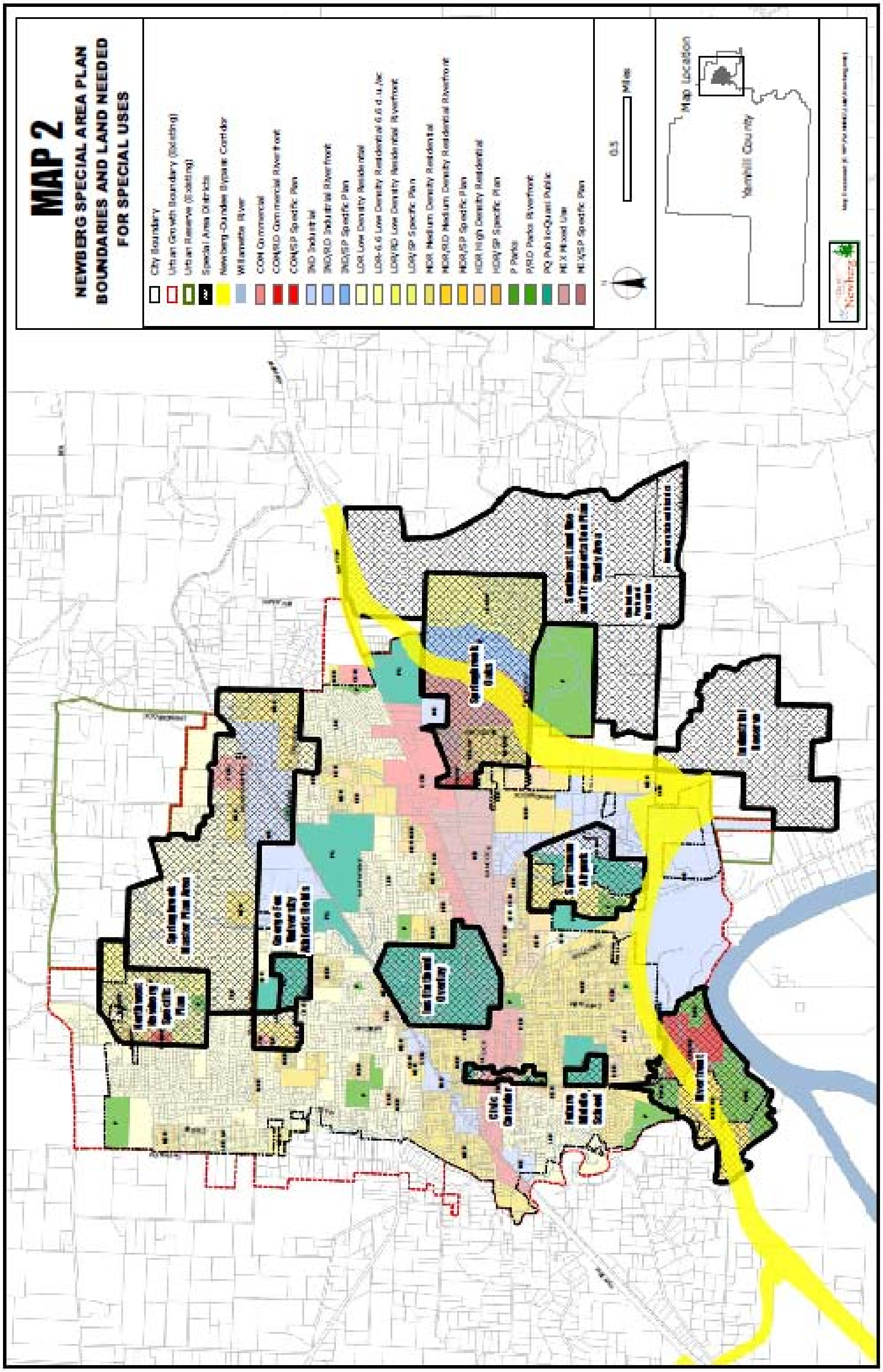


Figure ES-5. Current Newberg City Boundaries and UGB

ES.4.1 Population

Newberg is a fast-growing community in the Willamette Valley. Its current population is just over 20,000 people. Population projections for the service area are integral to projecting sewage flows. The population forecasts for the Newberg UGB are from the Johnson Gardner Report (July 2004). These forecasts are in line with those officially adopted by the City in December 2005. Table ES-1 lists the projected populations for a range of potential growth rates: low growth, median growth, and high growth. The median projected population in 2025, 2030, and at the end of the planning period in 2040 are 38,352, 43,600, and 54,097 respectively. The median growth rate was used for planning the phased CIP. The high growth rate was used to determine land area requirements for ultimate build-out.

Table ES-1. Population Projections

Year	Low growth population	Median growth population	High growth population
2005	20,623	21,132	22,180
2010	23,332	24,497	26,985
2015	26,691	28,712	33,190
2020	30,561	33,683	40,859
2025	33,957	38,352	48,833
2030		43,600[1]	
2040	44,505	54,097	79,701

[1] Population estimate for 2030 based on straight line projection from 2025 to 2040.

ES.4.2 Flows

Planning criteria related to projected flow rates are summarized in Table ES-2. The flow projections were developed by analyzing WWTP operating data from January 1996 to December 2004. The projected average annual flow for 2005 is 3.11 million gallons per day (mgd). For comparison, the actual average annual flow for 2005 was 2.73 mgd. The projected peak hour flow for 2005 is 20.81, the plant has seen higher flows than that after rehabilitating influent pumps. The projected 2005 maximum month wet weather flow (MMWWF) is 7.52 mgd. In December 2006, the maximum month flow was 8.1 mgd, and plant staff had to make significant process modifications to avoid violating the permit. Permit compliance for most water quality parameters is based on a monthly average performance. Therefore, the flow projections for these water quality parameters are based on monthly WWTP data. Peak hour flow projections are used to plan for hydraulic capacity.

Table ES-2. Flow Projections (mgd) Based on Median Growth Projections

Year	2005	2010	2015	2020	2025	2030	2040
Population	21,132	24,497	28,712	33,683	38,352	43,600	54,097
Average dry weather flow (ADWF)	2.07	2.40	2.81	3.30	3.76	4.27	5.30
Maximum month dry weather flow (MMDWF)	3.52	4.08	4.78	5.61	6.39	7.26	9.01
MMWWF	7.52	8.71	10.21	11.98	13.64	21.11	19.24
Peak hour flow (PHF)[1]	20.81	23.65	27.15	31.19	34.77	38.47	45.86

[1] PHF peaking factor (varies) decreases with time (0.2 mgd subtracted for every 5 years of population growth) because peaking factors usually decrease with increasing service area.

The flow projections for low, median, and high growth scenarios for 2040 are listed in Table ES-3. The median growth projections were used to develop the capital improvements projects recommendations. The projections for 2040 were used to determine the potential WWTP land area requirements for ultimate buildout.

Table ES-3. Flow Projections for 2040

	Low growth estimate	Median growth estimate	High growth estimate
Population	44,505	54,097	79,701
ADWF	4.36	5.30	7.81
Average annual flow	6.55	7.97	11.74
Annual wet weather flow	9.12	11.08	16.32
MMDWF	7.41	9.01	13.28
MMWWF	15.83	19.24	28.35
Peak week flow[1]		26.19	
Peak day flow[2]		41.40	
PHF[2, 3]		45.86	

[1] Existing data based on a running 7-day average. Daily minimums and maximums do not apply.
[2] Peak flows are not directly related to population, but rather I/I in the pipes that serve the population. Assumes no I/I removal.
[3] PHF peaking factor (varies) decreases with time (0.2 mgd subtracted for every 5 years of population growth).

The City's Sewerage Master Plan Update (by Brown and Caldwell, June 2007) initial results in April 2007 show that the peak flows that the collection system will currently convey to the WWTP are 17.6 mgd in 2007, 31 mgd in 2025, and 36 mgd in 2040 assuming undersized pipes are replaced. This compares well for 2007. However, the plant is projected to see peak flows of 35 mgd in 2025 using the Facilities Plan methodology rather than by 2040 using the Master Plan methodology. The Facilities Plan projections assume that I/I is not removed, while the SMPU assumes that the new pipes will not have high I/I. To help account for this difference, the recommended hydraulic improvements have been phased for incremental expansion. Should the I/I be removed by 2025, no additional hydraulic improvements are expected to be needed to serve 2040 peak flows.

ES.4.3 Loads

The design loads for 2010 to 2040 based on the median growth rate are summarized in Table ES-4.

Table ES-4. Load Projections from 2005 to 2040 Based on Median Population Growth

Parameter	Year					
	2010	2015	2020	2025	2030	2040
5-day carbonaceous biochemical oxygen demand ($CBOD_5$), pounds per day (ppd)						
Maximum month	5,836	6,840	8,025	9,137	9,508	12,888
Average month	3,318	3,888	4,562	5,194	5,405	7,326
Total suspended solids (TSS), ppd						
Maximum month	8,814	10,330	12,119	13,799	14,359	19,464
Average month	4,423	5,128	6,010	7,050	8,354	11,323

Table ES-5 summarizes the load projections for 2040 for the three growth scenarios.

Table ES-5. Load Projections for 2040

Parameter	Low growth estimate, ppd	Median growth estimate, ppd	High growth estimate, ppd
Average $CBOD_5$	6,027	7,326	10,794
Maximum month $CBOD_5$	10,603	12,888	18,988
Average TSS	9,316	11,323	16,683
Maximum month TSS	16,013	19,464	28,676

The projections for influent ammonia will be an average monthly ammonia concentration of 15.9 milligrams per liter (mg/L) and a maximum monthly concentration of 25.4 mg/L.

ES.5 REGULATORY REQUIREMENTS

The following summarize current and proposed regulations and establish the design criteria to be used in the development of the various treatment and disposal alternatives for the City's wastewater treatment system. The DEQ criteria listed include the Willamette Basin standards, Willamette River discharge criteria, reuse criteria for land application of effluent and biosolids, and U.S. Environmental Protection Agency (EPA) criteria for reliability and redundancy.

ES.5.1 NPDES Discharge Permit—Treatment and Discharge Requirements

The City was issued an NPDES permit from DEQ on June 22, 2004, for its Level III Collection System and Level IV treatment system that discharges to the Willamette River. A copy of the permit is provided in Appendix A. The City currently directs all treated water to the Willamette River. This practice is covered by the NPDES permit. In 2006, the City requested that the DEQ modify the NPDES permit to include a reuse outfall for irrigation on local golf courses, an approved overflow manhole in Hess Creek for the pump station, and acceptance for the revised Biosolids Management Plan. DEQ is currently working on the open permit. The reopened permit is expected to include new discharge requirements to limit temperature discharges to the Willamette River based on the 2006 Willamette River Total Maximum Daily Loads (TMDLs).

The current permit requirements are listed in Tables ES-6 through ES-8. The WWTP has had no permit violations.

Table ES-6. Current Permit Requirements, May 1 – October 31

	Limitation				
	Average concentration, mg/L		Mass load[1]		
Parameter	Monthly	Weekly	Monthly average, ppd	Weekly average, ppd	Daily maximum, pounds
$CBOD_5$[2]	10	15	330	500	660
TSS	10	15	330	500	660

[1] The daily mass load limit is suspended on any day in which the daily flow to the treatment facility exceeds 8 mgd (twice the design ADWF).

[2] The $CBOD_5$ concentration limits are considered equivalent to the minimum design criteria for BOD_5 specified in Oregon Administrative Rules (OAR) 340-041. These limits and $CBOD_5$ mass limits may be adjusted (up or down) by permit action if more accurate information regarding $CBOD_5/BOD_5$ becomes available.

Table ES-7. Current Permit Requirements, November 1 – April 30

	Limitation				
	Average concentration, mg/L		Mass load[1]		
Parameter	Monthly	Weekly	Monthly average, ppd	Weekly average, ppd	Daily maximum, pounds
$CBOD_5$[2]	25	40	1,400	2,000	2,700
TSS	30	45	1,600	2,400	3,200

[1] The daily mass load limit is suspended on any day in which the daily flow to the treatment facility exceeds 8 mgd (twice the design ADWF).

[2] The $CBOD_5$ concentration limits are considered equivalent to the minimum design criteria for BOD_5 specified in OAR 340-041. These limits and $CBOD_5$ mass limits may be adjusted (up or down) by permit action if more accurate information regarding $CBOD_5/BOD_5$ becomes available.

Table ES-8. Current Permit Requirements, Year Round

Parameter	Permit requirement
E. coli	Shall not exceed 126 organisms per 100 milliliters (mL) monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. If a single sample exceeds 406 organisms per 100 mL, then five consecutive resamples may be taken at 4-hour intervals beginning within 28 hours after the original samples were taken. If the log mean of the five resamples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.
pH	6 to 9
$CBOD_5$ and TSS removal efficiency	Shall not be less than 85 percent monthly average for $CBOD_5$ and 85 percent monthly for TSS.
Total residual chlorine	Shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum concentration of 0.05 mg/L. When the total residual chlorine limitation is lower than 0.10 mg/L, DEQ will use 0.10 mg/L as the compliance evaluation level (i.e., daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitation).

ES.5.2 Biosolids Management Plan

The City developed a Biosolids Management Plan in May 2004, included in Appendix B (to be approved as part of the 2007 open permit process) in accordance with the WWTP permit and OAR 340-050, *Land Application of Domestic Wastewater Treatment Facility Biosolids, Biosolids Derived Products, and Domestic Septage* and 40 CFR, §503. All waste sludge must be managed in accordance with the DEQ-approved Biosolids Management Plan to ensure compliance with the federal biosolids regulations (40 CFR, §503) and the state rules (OAR 340-050).

ES.5.3 Pretreatment Program

An Industrial Pretreatment Program has been developed and is being implemented in accordance with federal regulations governing pretreatment programs, as required by EPA's Code of Federal Regulations Title 40 (40 CFR, §403) and approved by DEQ. The City's Industrial Pretreatment Program is designed to protect its Publicly-Owned Treatment Works; the Newberg WWTP, solids quality, the City's collection system, the Willamette River, and worker health and safety. The Pretreatment Programs have undergone informal review by DEQ and are approvable by DEQ.

ES.5.4 Regulatory Criteria

The important regulatory criteria that could influence the direction of the Facilities Plan are summarized below:

- Mass limits
- I/I removal
- SSO elimination
- 85 percent removal
- Nutrients
- TMDLs for temperature
- Mixing zone
- Redundancy and reliability
- Class A biosolids

Mass Limits. Allowable mass limits should reflect design flows at effluent concentrations of 10 mg/L $CBOD_5$ and 10 mg/L TSS (10/10). Oregon Environmental Quality Commission (EQC) approval is required to increase mass loads due to increased flows. The Facilities Plan should preserve maximum flexibility by using an incremental approach for phased expansion to accommodate the need for more or less stringent requirements, triggered by water quality requirements.

If necessary, the City may request new loads of the EQC. Environmental, economic decision-guiding criteria and existing water quality management policies need to be addressed prior to requesting EQC approval. If the plant continues to perform at its current level, mass load increases would not be needed.

I/I Removal. Cost-effective I/I removal should be performed along with continued flow equalization. The City must demonstrate that I/I will be reduced to the cost-effective point based on EPA requirements. No plant capacity will be provided for excess I/I. In addition, the following Newberg NPDES permit language for emergency overflow outfalls applies:

> *"No wastes shall be discharged from these outfalls, unless the cause of the discharge is due to storm events as allowed under OAR 340-041-0120 (13) or (14) as follows: Raw sewage discharges are prohibited to waters of the State from November 1 through May 21, except during a storm event greater than the one-in-five-year, 24-hour duration storm, and from May 22 through October 31, except during a storm event greater than the one-in-ten-year, 24-hour duration storm. If an overflow occurs between May 22 and June 1, and if the permittee demonstrates to the Department's satisfaction that no increase in risk to beneficial uses occurred because of the overflow, no violation shall be triggered if the storm associated with the overflow was greater than the one-in-five-year, 24-hour duration storm."*

SSO Elimination. The City is planning to remove SSOs by December 31, 2010.

85 Percent Removal. The WWTP can meet the 85 percent removal requirement. The plant should not be required to achieve 85 percent removal of $CBOD_5$ and TSS at all times, as long as permitted effluent concentration limits are being met and I/I is being removed to the extent that is cost-effective.

Nutrient Removal. The need for nutrient removal should be driven by water quality. Nutrient removal will probably not be required initially, but the Facilities Plan describes the approach to be taken if nitrogen and/or phosphorus limits are imposed. The EPA is currently reviewing the need for nutrient removal requirements from WWTPs to protect the nation's waters. This is generally the first step in establishing standards for criteria in the future. Should the EPA promulgate nutrient removal requirements, DEQ would allow Oregon treatment facilities time to comply by incorporating compliance schedules into the next permit renewal following promulgation.

TMDLs for Temperature. The temperature TMDL for the Willamette River was adopted September 21, 2006. DEQ will insert temperature Waste Load Allocations (WLAs) into the currently open permit. The City plans to implement additional effluent reuse to address thermal loads discharge limitations.

Mixing Zone. Mixing zones are allowed by regulatory agencies. DEQ is studying the impacts if mixing zones on water quality. With the current mixing zone, Newberg meets current water quality regulations. Anew mixing zone study is currently underway that follows the DEQ Regulatory Mixing Zone Internal Management Directive (July 2008).

Redundancy and Reliability. Class I reliability is appropriate for the Newberg WWTP.

Class A Biosolids. The City is planning to continue to produce Class A biosolids since a market has developed for the Class A product.

ES.6 BASIS FOR COST ESTIMATES

The following paragraphs cover the basis and limitations for cost estimates.

ES.6.1 Precision of Cost Estimates

The precision of a cost estimate is a function of the detail to which alternatives are developed and the techniques used in preparing the actual estimate. The American Association of Cost Engineers defines the order-of magnitude cost estimate as:

> "An estimate is made without detailed engineering data. Techniques such as cost-capacity curves, scale-up or scale-down factors, and ratios are used in developing this type of estimate. This type of estimate is normally accurate within +50 percent or -30 percent. That is, the final cost may be as much as 50 percent more or 30 percent less than the estimated amount. A relatively large contingency is normally included to reduce the probability of underestimating."

The estimates presented in this document are order-of-magnitude estimates because the design has not been developed in sufficient detail for a more precise estimate. A 35 percent contingency is included in the cost estimates. Although the final project cost may vary significantly from these estimates, the estimates are useful in evaluating alternatives because they are fairly accurate relative to each other.

ES.6.2 Basis for Costs over Time

Future changes in the costs of material, labor, and equipment will cause comparable changes in the costs presented in this analysis. However, because the relative economy of the alternatives is expected to change only slightly with overall economic changes, the decisions based on the economic evaluation should remain valid.

The construction costs developed for the recommended phasing for the City's CIP are in March 2007 dollars unless otherwise noted. Phase 1—Immediate Improvements costs are escalated to the midpoint of construction, assuming the midpoint is in 2011. Future phases cannot be realistically escalated and are in 2007 dollars only. The cost of steel and concrete has increased significantly in the last year and this trend is continuing due to the natural disasters that occurred in 2005. The costs are expected to increase 10 percent each year over the next 2 years and cannot be predicted past this point.

ES.7 ALTERNATIVES ANALYSIS SCREENING CRITERIA

The alternatives analysis included using the screening criteria for each individual process to prioritize the alternatives to receive further investigation. Liquids treatment processes that met the regulatory requirements were included in the initial analysis. Only Class A biosolids treatment options were considered since the City has developed a market for Class A product, and the industry trend is moving toward Class A technology.

ES.8 EVALUATION PROCESS

The evaluation process included two Liquid Solids Workshops conducted by Brown and Caldwell. Liquids Solids Workshop No. 1, held on May 23, 2006, consisted of identifying unit process deficiencies and brainstorming technologies to be included in the analysis of wastewater treatment and biosolids alternatives analyses. An initial viability evaluation and screening was used to eliminate alternatives from further consideration. The screening criteria include:

- Relative present worth costs
- Energy conservation
- Regulatory compliance
- Flexibility
- Reliability
- O&M
- Safety
- Viability at the Newberg WWTP

The initial screening used the ratings +, 0, and – for relative scoring. The evaluation was used in Liquids Solids Workshop No. 2, held on December 14, 2006, to verify the ranking of the alternatives in the group setting. If the alternative was not viable at the Newberg WWTP, it was so noted and no scoring was completed. An evaluation of the remaining viable alternatives was conducted by the Brown and Caldwell team.

The alternatives evaluated for the major unit process and the recommendations are summarized below.

ES.9 IPS

The IPS is an essential component of the WWTP. It pumps the wastewater approximately 100 feet between the lowest point in the collection system up to the headworks that provides screenings and grit removal. The pump station is currently under capacity. It cannot convey peak flows when one unit is out of service. Typical high influent flow events could cause permit violations. In addition, there are safety concerns with the existing pump station wet well. The wet well is inefficient and causes frequent problems from rags and debris clogging the pump impellers, which decreases the pumping capacity and requires frequent cleaning. The IPS upgrades and expansion are needed immediately.

The alternatives considered for upgrade and expansion of the IPS to meet current needs and provide for future service include:

Alternative 1: Building additional capacity at the north end of the plant

Alternative 2: Expanding the existing facilities

Alternative 3: Replacing the existing IPS with a new structure next to the existing structure

Alternative 4: Building additional capacity next to the existing and upgrading the existing IPS

Alternative 4 is the recommended expansion alternative. The recommended improvements to the IPS, for safety and capacity reasons, include building additional capacity next to the existing IPS for base flows and upgrading the existing wet well for overflow capacity pumping. The range of flows expected at the IPS is best accommodated by a dual pump station–low and moderate flows would be pumped by a station with a self-cleaning wet well, while higher wet weather flows would be pumped by the overflow pump station with confined inlet pumps. The recommended IPS improvements include modifying the inlet pipe slope, wet well, and related structure for 2040 flow conditions. The pumps selected and installed will be for 2025 flow conditions. The pump station will be able to pump flows in excess of 2015 flows because of the pump sizing constraints that more ideally fit the 2025 phasing. Variable-frequency drives for these pumps are included in the cost estimate. The expansion to Phase 3 will only require modifications or replacement of pumps. The IPS Electrical Room (by others) is sized for future space requirements.

The proposed pump station layout is shown in Figure ES-6.

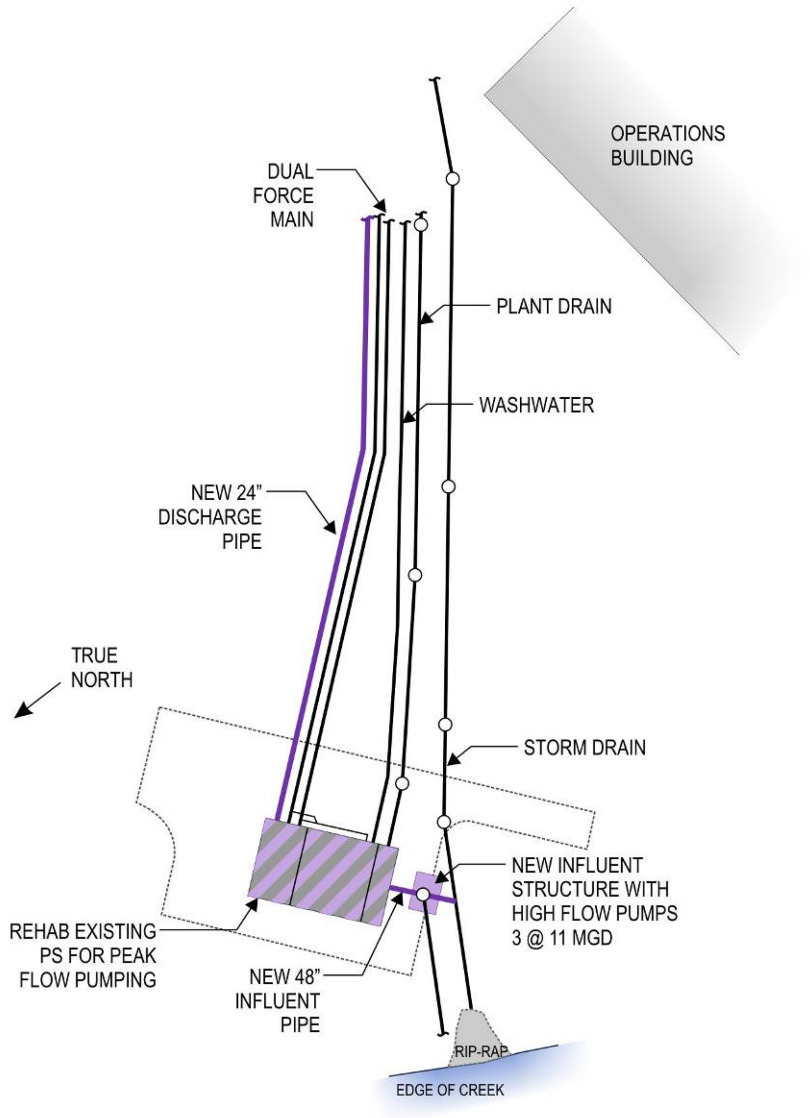


Figure ES-6. IPS Yard Piping Plan

It is also recommended that a section of the influent pipe be elevated sufficiently to remove the slope to the IPS that causes poor influent characteristics and high velocities at peak flows in the IPS. The influent pipe will be a new 60-inch diameter pipe at a slope of 0.0007 foot per foot to limit inflow velocities to less than 4 feet per second. This size pipe is satisfactory for both current and 2040 flow rates so that replacement in the future will not be necessary. When the influent pipe is re-laid, the slope into the wet well will be improved, and the new self-cleaning wet well will be located adjacent to the existing IPS but at a higher elevation.

During the facilities planning process, the Motor Control Center Building location for the IPS was discussed as part of the reuse design process. It was determined that a location to the west of the Administration Building would be optimum. This location avoids the influent piping at the east of the Administration Building, is adequately proximate to the IPS, and avoids the additional costs of construction on a steep slope and where the site is already constrained adjacent to the IPS.

The phased improvements, based on peak hour flow requirements, will provide the incremental IPS capacity, as shown in Figure ES-7.

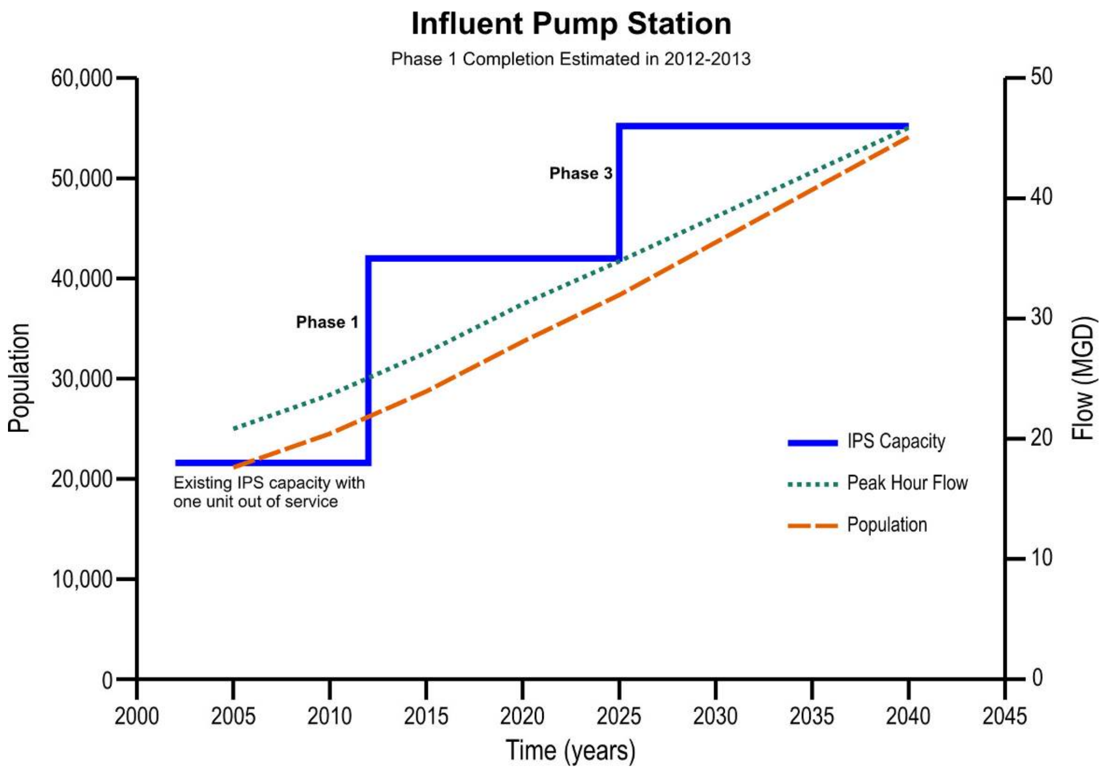


Figure ES-7. Incremental IPS Capacity

ES.10 HEADWORKS

The headworks processes include screening and aerated grit. The screens remove particles greater than 10 millimeters in diameter. The grit is removed with an aerated grit chamber. Although the screens were recently replaced with new, more reliable screens, the existing channel configuration does not allow conveyance and treatment of the total influent flow when one unit is out of service without bypassing around the process.

It is assumed that expansion will include the same type of screens as existing for ease of O&M and because they were determined to be cost-effective in 2002 during the Newberg Dump Station and Headworks Study conducted by Brown and Caldwell. The most cost-effective screen was chosen at that time. Plant staff have had positive experiences with these screens.

The screens will be installed in channels on the east side of the existing headworks, as shown in Figure ES-8. Emergency power should be added to ensure that critical headworks functions can continue in the event of a power outage. Odor control should be provided also as a good neighbor policy and to maintain compliance with OAR 208 that prohibits nuisance conditions such as odors.

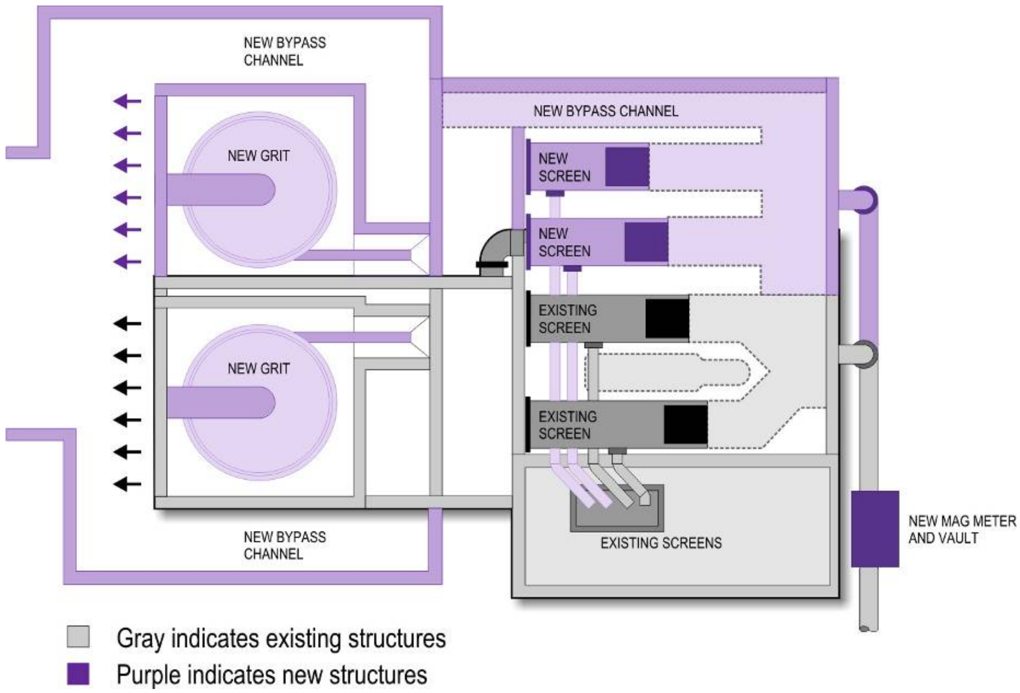


Figure ES-8. Headworks Improvements

The grit removal process is currently undersized, and the recommendation is to provide full grit removal for all flows. Therefore, additional grit removal capacity is needed immediately.

The initial screening analysis for grit improvements includes the following five alternatives:

Alternative 1: Stacked tray separator

Alternative 2: Vortex grit settling with agitation

Alternative 3: Air vortex grit separator

Alternative 4: Free vortex separator

Alternative 5: Expand existing system

The plate gravity settling system that removes grit using a series of stacked plates is the recommended grit removal system to provide the capacity. The stacked plate type grit removal system is shown in Figure ES-9.

Figure ES-9. Photo of Gravity Plate Type Settler for Grit Removal

New flow distribution and flow monitoring will need to be provided. The existing magmeters are not installed for accurate flow measurement. Magmeters will be installed approximately 10 to 20 feet upstream of the headworks to more accurately measure flow. The phased improvements, based on peak hour flow requirements, will provide the incremental headworks capacity as shown in Figure ES-10.

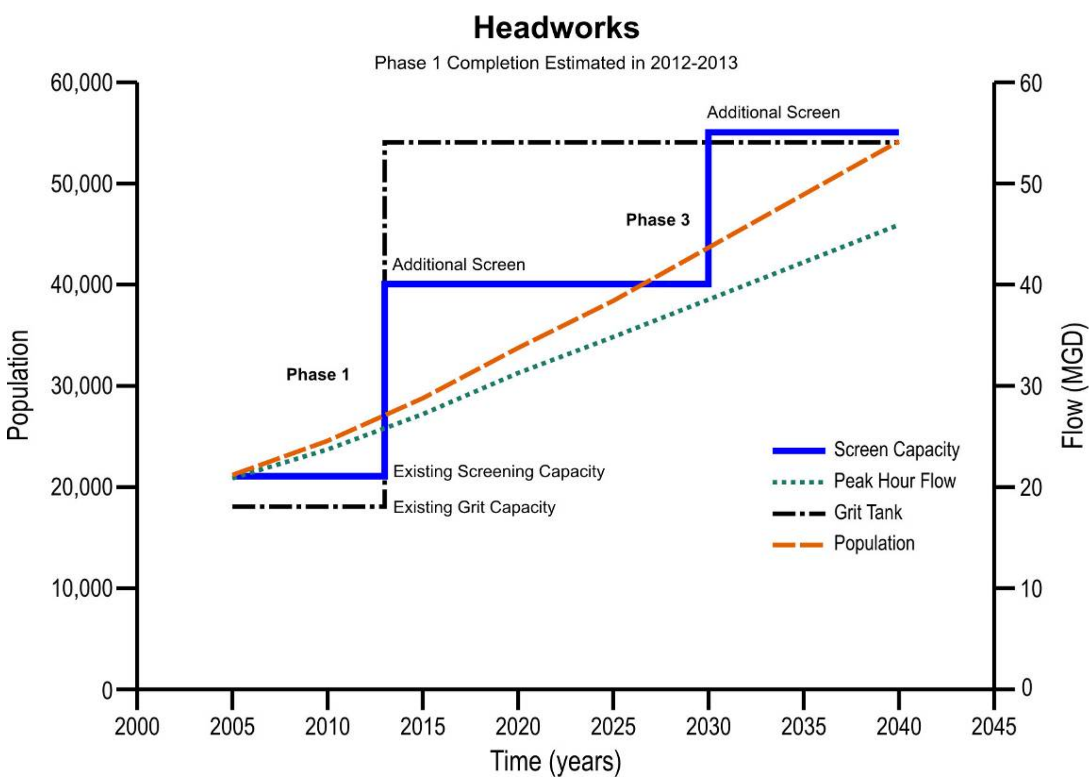


Figure ES-10. Headworks Phased Capacity

ES.11 SECONDARY TREATMENT

The Newberg WWTP currently uses two oxidation ditches for secondary biological treatment to meet regulatory permit requirements. The secondary system is currently undersized for maximum month flow conditions. The analysis for the oxidation ditches and secondary clarifiers were conducted with a static model of both systems as their operations are inter-related in performance capacity.

ES.11.1 Oxidation Ditches

The recommended expansion includes continuing to use the oxidation ditch process because of its low energy and maintenance costs and its ability to treat a wide variation in flows and loads. By 2010, a third oxidation ditch is needed to provide adequate treatment to meet effluent quality requirements. A fourth oxidation ditch is needed by 2015. The City has an interest in acquiring the adjacent Baker Rock property for expansion of the secondary system. However, in the event this land area expansion does not take place, additional processes were considered. The initial screenings evaluation for oxidation ditches included the following alternatives:

Alternative 1: Conventional oxidation ditch

Alternative 2: Vertical loop reactor oxidation ditch

Alternative 3: Cannibal

Alternative 4: Membrane bioreactor

Based on the results of this analysis, a present worth cost comparison, and consensus reached at Liquids Solids Workshop No. 2, expansion with the current oxidation ditch and secondary clarifier processes is the preferred alternative. Should site constraints or significantly more stringent effluent quality become an issue, membrane treatment could be added either in conjunction with the oxidation ditches or by replacing the oxidation ditches and secondary clarifiers with MBRs which would significantly reduce the footprint requirements. The phased capacity expansions for the oxidation ditch process, based on maximum month flow and nitrogen reduction requirements, are shown in Figure ES-11.

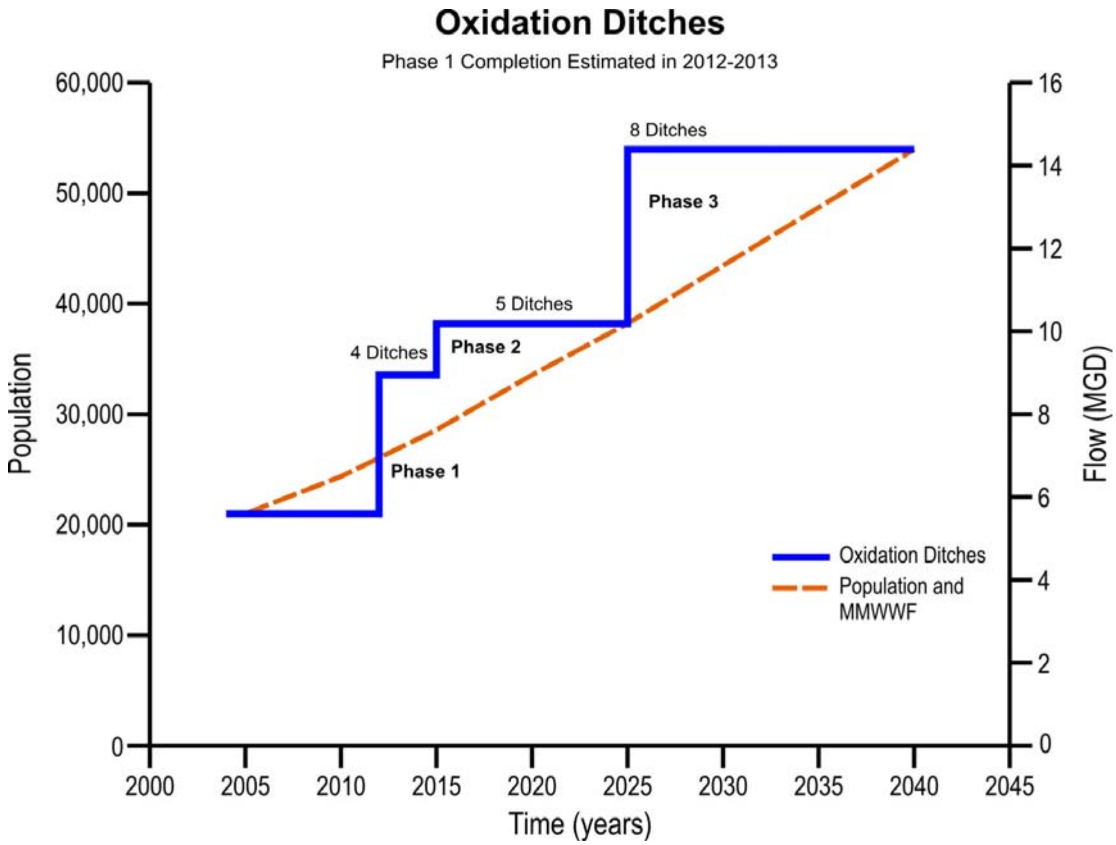


Figure ES-11. Phased Oxidation Ditch Capacity with Nitrogen Reduction Requirements

ES.11.2 Secondary Clarification

Secondary clarifiers separate the biological organisms from the biologically treated wastewater prior to disinfection. The capacity of the secondary clarifiers is related to both hydraulic flow and the mass of biological solids from the oxidation ditches. The secondary system model of both the oxidation ditches and secondary clarifier operation predicted that the secondary clarifier process will need to be expanded with increased population and to match the additional oxidation ditch capacity. By 2012, a fourth secondary clarifier will be needed to meet effluent quality requirements. The phased capacity of the secondary clarifier system, based on maximum month flow requirements, is shown in Figure ES-12.

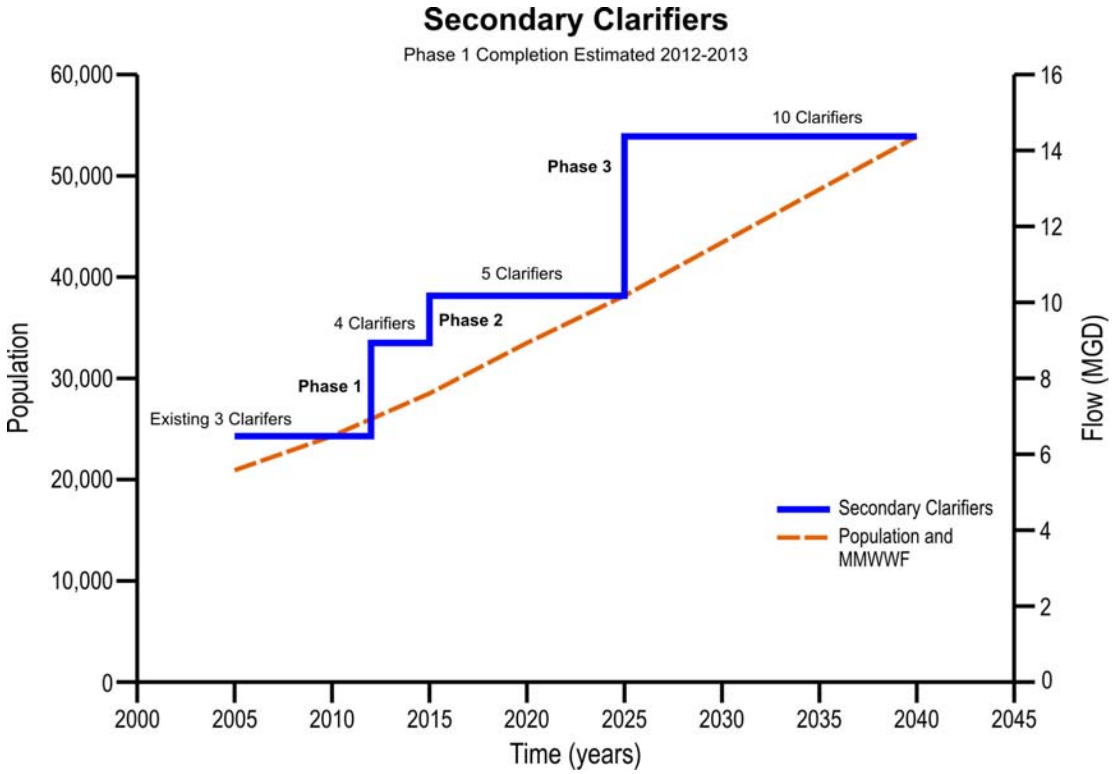


Figure ES-12. Phased Secondary Clarification Capacity with Nitrogen Reduction Requirements

ES.12 DISINFECTION PROCESS

Clarified effluent must be disinfected prior to discharge or reuse. Currently, the disinfection process consists of a chlorination system that uses ton cylinders of chlorine gas. Immediate changes are needed to improve the reliability of the effluent quality to continue to meet disinfection permit requirements. These include chemical induction mixer(s) at the chlorine injection point, scum removal, improved effluent flow monitoring, and automatic disinfection control strategy. Roof drainage needs to be re-routed out of the contact basin. The initial screening of the disinfection alternatives for expansion of the disinfection process includes the following:

Alternative 1: High-rate disinfection

Alternative 2: Additional contact basin

Alternative 3: Additive of onsite generation of sodium hypochlorite

Alternative 4: Ultraviolet (UV) disinfection

The City will continue with gas chlorine for the first 5-year permit cycle as well as the existing contact basins. The City is considering phasing in hypochlorite when the Newberg WTP is constructed in closer proximity to the WWTP. High-rate disinfection can be used to increase the effectiveness of the disinfection to accommodate the limited contact time in the existing contact chamber. The City is also investigating the applicability of phasing in UV treatment at a later date.

UV disinfection may not be feasible at the WWTP since the effluent has iron which can inhibit UV effectiveness.

Disinfected wastewater is currently dechlorinated at the outlet of the chlorine contact basins. The dechlorination system requires complete replacement to be more effective, but currently capacity is limited by the configuration of the equipment. A new 1,050-gallon high-density polyethylene storage tank, two new feed mechanical diaphragm pumps and a new control system is recommended for immediate implementation. The phased disinfection capacity, based on peak hour flow requirements, is shown in Figure ES-13.

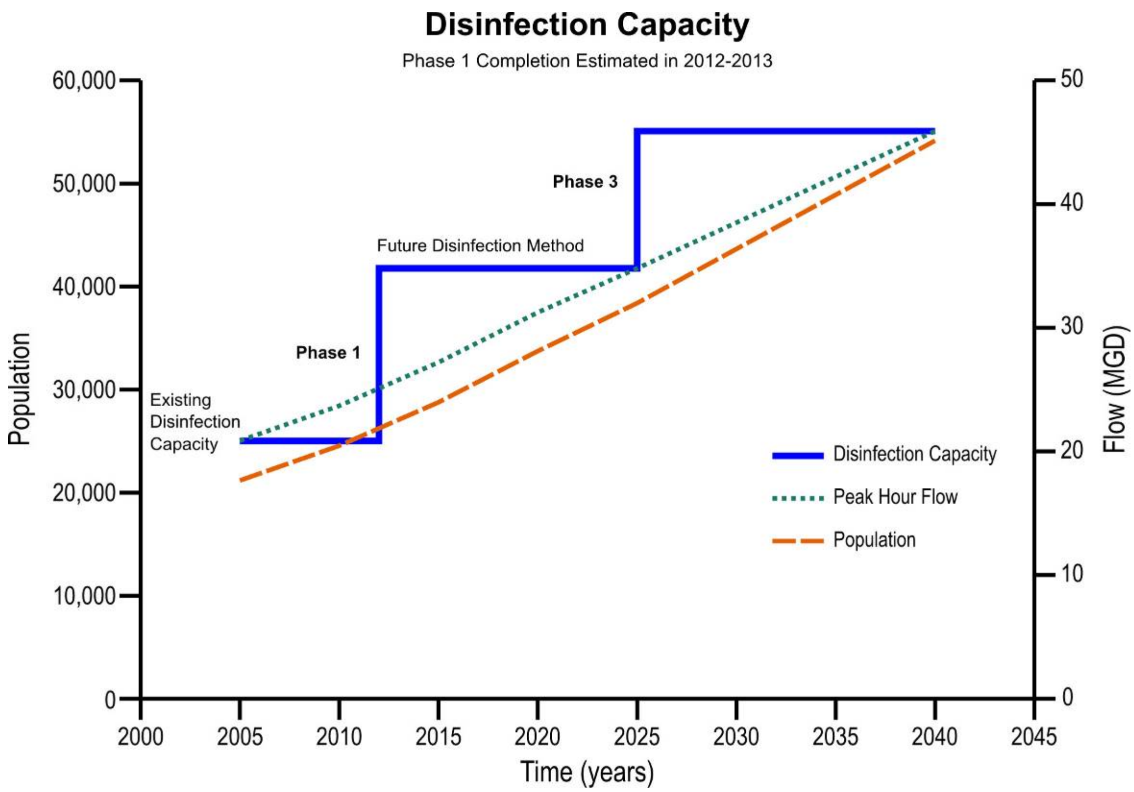


Figure ES-13. Disinfection Capacity

ES.13 OUTFALL

The outfall is primarily a conveyance unit process and the capacity is needed to convey peak flows to the river discharge point. Due to hydraulic conditions caused by air entrainment at high flows that are called a hydraulic cannon, the outfall has experienced structural damage to the uphill manhole. In order to alleviate the hydraulic cannon effects, a parallel outfall down the slope is recommended to be implemented immediately for safety reasons. This will prevent the air entrapment and alleviate the hydraulic effects. The phased outfall capacity increase, based on peak hour flow requirements, is shown in Figure ES-14. The mixing zone study initiated in June 2009 is needed to determine if other outfall modifications are required.

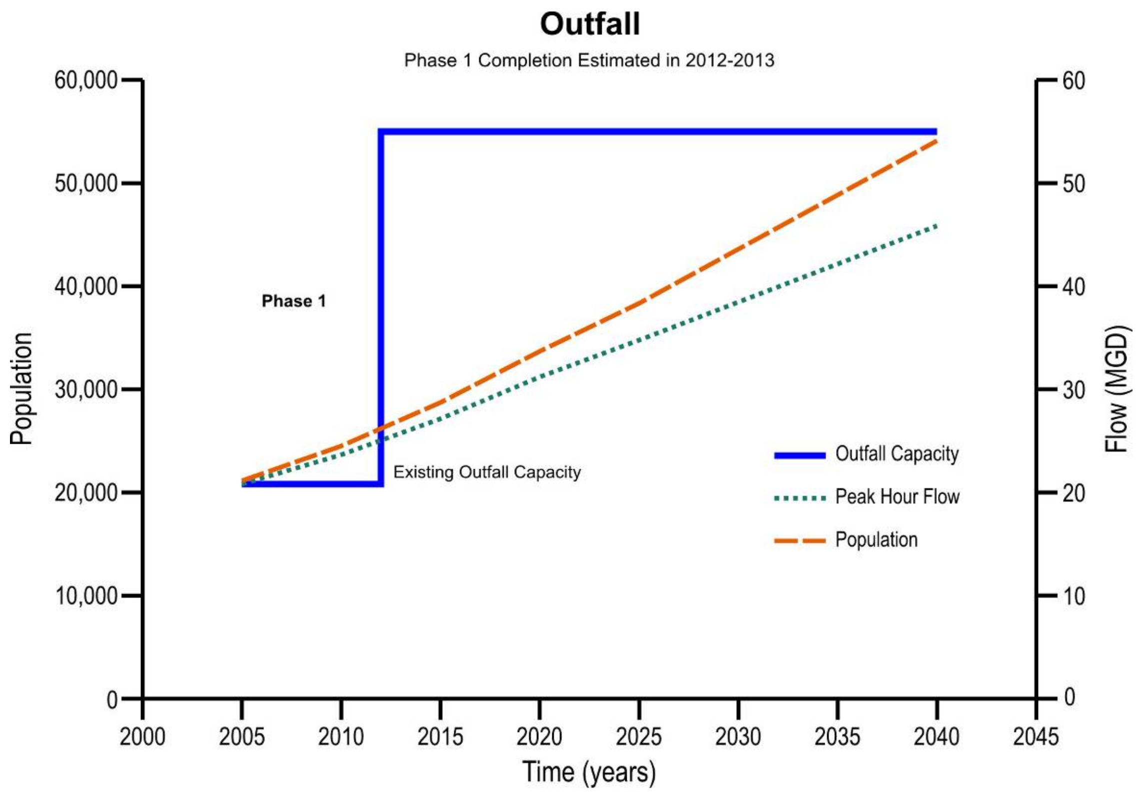


Figure ES-14. Phased Outfall Capacity

ES.14 REUSE

The City is implementing plans to use treated effluent for irrigation of a local golf course.

ES.14.1 Irrigation

Tertiary treatment using membranes has been selected by the City after a predesign evaluation and recommendation documented in the *Reuse Water System Predesign Study* (CH2M Hill, October 2005). Membranes will be assumed to be the preferred technology for future tertiary treatment for reuse at local golf courses (by others). The City is currently planning to provide variable reuse water from April through October, with the lowest demand expected in April. The peak delivery capacity for the hottest summer months is 1 mgd.

ES.14.2 Temperature Compliance

DEQ implemented the Willamette TMDL WLAs in the City's NPDES permit by permit modification. The permit modification is included in Appendix A. The City has the opportunity to track river and effluent temperature and flow on a 7-day running average to comply. Even under the worst case scenario, the City will be able to discharge 3.5 mgd at 23°C, which was the highest 7-day average daily maximum temperature recorded in 2008. (In 2007, the highest single day maximum temperature was 23°C.) In most cases, the City will be able to discharge 4.0 mgd at 23°C or 3.0 mgd and 24°C.. The City has implemented a reuse program to irrigate local golf courses that decreases its effluent discharge by 1 mgd during the irrigation season. As the City grows, the City will meet the WLAs with increased irrigation reuse.

A temperature management plan may be required in the future to show how the City will maintain compliance with the temperature TMDL. The City currently meets the TMDL requirements, so a plan is not included with the temperature allocations in the permit modification. Options to maintain compliance include but are not limited to:

- Increasing reuse and storage for peak flows
- Storing effluent using a combination of night-time discharge when the ambient air and effluent is cooler
- Cooling the effluent prior to discharge through subsurface discharge to the hyporheic zone
- Implementing best management practices at the WWTP to minimize heating across the treatment processes
- Treating effluent using other methods such as wetlands
- Temperature trading

The City plans to add additional reuse to address the temperature WLA in the future. Depending on the final temperature management plan, some storage may need to be provided. The golf course that will use the treated water for irrigation has 3 million gallons of storage capacity.

ES.15 SOLIDS HANDLING AND TREATMENT

The compost process has reached capacity because the compost feed mix has a high moisture content. Compost capacity is based on peak week solids production, solids, and feed sawdust moisture content. Recent market demands for sawdust has resulted in smaller buyers (including the City) receiving wetter sawdust product. This has resulted in an immediate need to provide static compost piles in addition to the mechanized composting operation. Decreasing the moisture in the sawdust with a sawdust dryer would result in a capacity increase and is recommended for immediate implementation. Capital costs are substantially lower than that of mechanical dewatering, and operation can provide the maximum immediate benefit in terms of compost system capacity. An investigation is in progress to determine potential grant funding (up to 40 percent) to reduce required capital investment. Potential energy and labor savings as a result of providing drier compost feedstock are also being evaluated.

After the sawdust dryer capacity is realized, upgrading the dewatering system is recommended for Phase 2 implementation to provide an additional increase in capacity for solids composting, to improve operational performance, and to reduce required bulking agent cost and material (recycle) handling. Centrifuge dewatering is considered a fundamental dewatering system that will benefit existing and future process technologies. Centrifuge dewatering, the only technology that achieves the highest solids content, will improve performance of the existing compost system and increase effective capacity by approximately 30 percent by increasing dewatered solids concentration. The higher solids content that results from centrifuge dewatering requires less bulking agent and reduces materials handling requirements. City staff indicate that the system can handle current solids flow of 3,750 wet tons (600 dry tons at 16 percent solids concentration). The City will experience a

30 percent reduction in solids volume and, effectively, a 30 percent increase in available compost capacity (on a dry ton basis) when the centrifuge is added.

For the Newberg WWTP, the most viable options for accommodating future growth with Class A product are:

Alternative 1: Composting

Alternative 2: Thermal drying

Alternative 3: Offsite energy recovery

For Class A process technologies, composting and thermal drying are nearly equal in cost, while offsite energy recovery is much less. Initial evaluations favor offsite energy recovery. Plant staff have indicated that a backup strategy using a simplified composting technology (aerated static pile) is desired until offsite energy recovery has been implemented locally and has been proven reliable for long-term service.

ES.16 ADMINISTRATION BUILDING

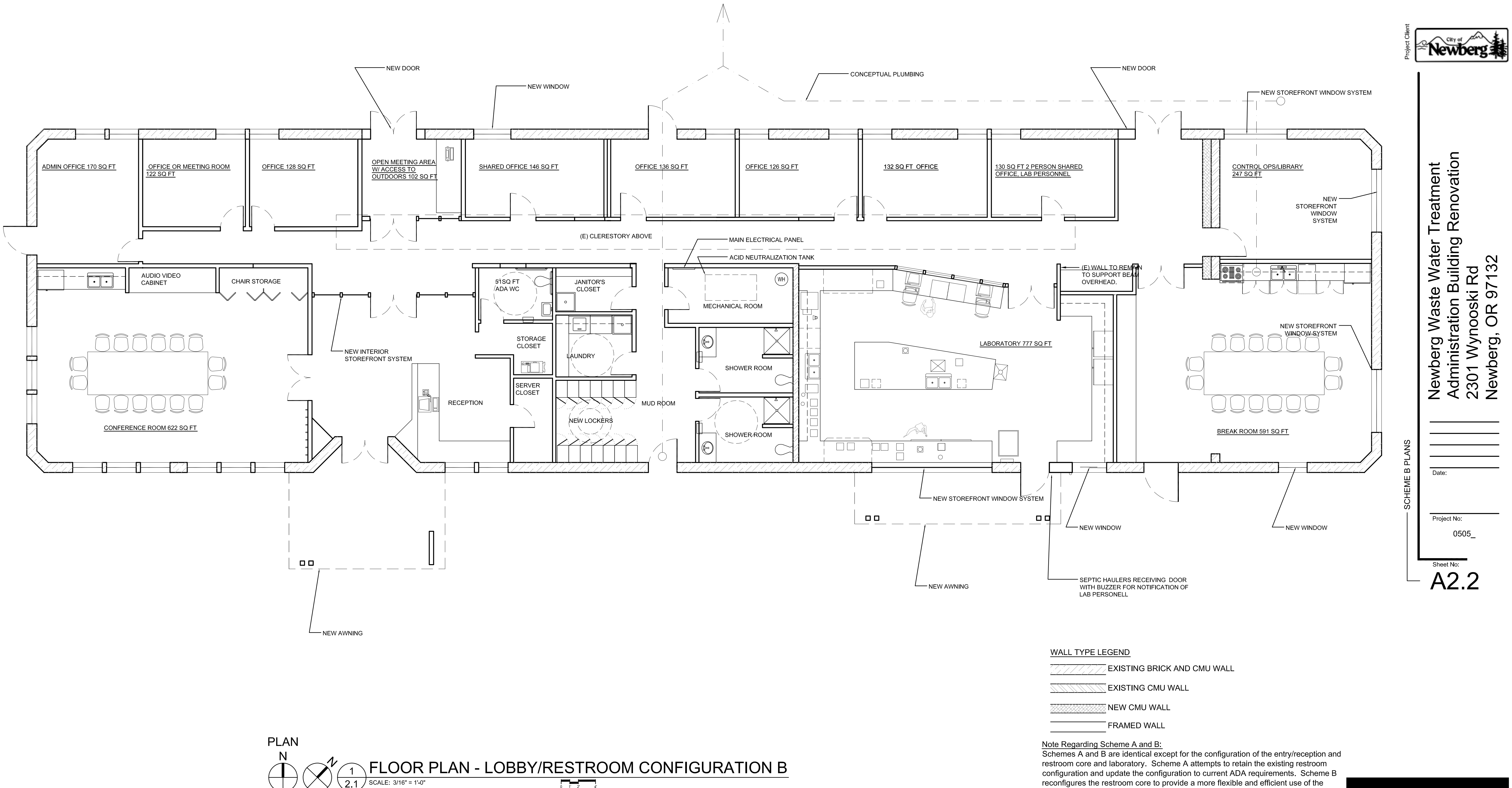
As part of the Facilities Planning process, an evaluation of the Administration Building at the WWTP was conducted. The purpose of the Administration Building evaluation was to develop a concept for a functional, secure, and energy-efficient facility that will improve operations. Built in 1987 and in operation since then, the Administration Building has undergone a number of significant changes in its programmatic functions over the past 20 years. Few design changes and upgrades have occurred over this period leading to a building that is highly inefficient in the use of its available space. For example, major functions such as the maintenance workshop have been moved out of the building into more appropriate locations on the plant site leaving underutilized voids of space; while remaining critical functions, such as the laboratory, administrative areas, and staff support areas, have developed increasing needs for space and technical updating. Most critically, emergency generator exhaust entered the buildings' ventilation system creating worker safety considerations. The recommendation to move the engine generator out of the existing building is already being implemented.

The planning considered the needs for 2025 and the potential to house the City's Water Plant administrative personnel and certain water treatment plant functions (shop, laboratory, etc.). The Administration Building improvements are recommended to be implemented immediately.

The proposed layout for the Administration Building is shown in Figure ES-15. When complete, the remodeled building will be a much improved facility with increased flexibility for growth, greater efficiency, and expanded functionality; and it will be a more productive environment for the WWTP staff and potentially the WTP staff to carry out its mission to the community.

ES.17 WASTEWATER TREATMENT SUPPORT SYSTEMS

Emergency generator needs were established as part of the Administration Building Predesign report. The emergency generator project is being completed by others as part of the reuse improvements.



FLOOR PLAN - LOBBY/RESTROOM CONFIGURATION B

WALL TYPE LEGEND

- EXISTING BRICK AND CMU WALL
- EXISTING CMU WALL
- NEW CMU WALL
- FRAMED WALL

Note Regarding Scheme A and B:
Schemes A and B are identical except for the configuration of the entry/reception and restroom core and laboratory. Scheme A attempts to retain the existing restroom configuration and update the configuration to current ADA requirements. Scheme B reconfigures the restroom core to provide a more flexible and efficient use of the required spaces and meeting current ADA requirements.

Project Client: City of Newberg

Newberg Waste Water Treatment
Administration Building Renovation
2301 Wynooski Rd
Newberg, OR 97132

SCHEME B PLANS

Date:

Project No:
0505_

Sheet No:
A2.2

BROWN AND CALDWELL

Burgee architecture LLC

20% Schematic Design - Draft 11/16/2005

Newberg Waste Water Treatment
Administration Building Renovation
2301 Wynooski Rd
Newberg, OR 97132

Figure ES-15. Proposed Layout for the Administration Building

Based on several meetings and a site walk-through on September 29, 2006, miscellaneous improvements were recommended to the following buildings:

- Chlorine Building, chlorine scrubber, and duct
- Secondary Return Activated Sludge/Waste Activated Sludge (WAS) Pump Building
- Solids Building
- Compost Building
- Compost Building doors

Stormwater generated onsite is conveyed by gravity to the IPS along with recycle streams. In-plant stormwater handling alternatives were studied and documented in a previous report entitled *Final Report for the Recommended Plan City of Newberg Dump Station/Headwork Studies* (*Final Dump Station/Headworks Studies* report) (Brown and Caldwell, June, 2002). The pump station will be located in the vicinity of Stormwater Manhole No. 1 and will be connected to the stormwater and recycle water systems through new gravity sewers.

The current reclaimed water system filters for in-plant use are inadequate and the screening size is too large to be effective. A looped plant water system is recommended that includes adding a source of plant water at the headworks and providing more hose bibs for cleaning at the aeration basins.

New septage receiving facilities are recommended. Septage receiving was studied in *Final Dump Station/Headworks Studies* report. The recommended improvements include modifications to the road southeast of the headworks (including a trench drain and catch basin), a Lakeside 31SAP-type septage receiving station, a buried septage receiving tank, duplex pumps in the septage receiving tank, piping to transfer the septage to the screening channel of the headworks, and a new access road around the north side of the headworks.

ES.18 PHASING OF RECOMMENDED IMPROVEMENTS

The phasing of the recommended improvements is shown in Figure ES-16. Phasing is planned in three increments. Phase 1 is to be completed as soon as funding is available. Figure ES-16 also shows the estimated population projections for low, median, and high growth scenarios, year of the estimated growth is expected to occur and the planned capacity phasing. As mentioned previously, an effective I/I elimination program could potentially postpone the Phase 2 construction numerous years.

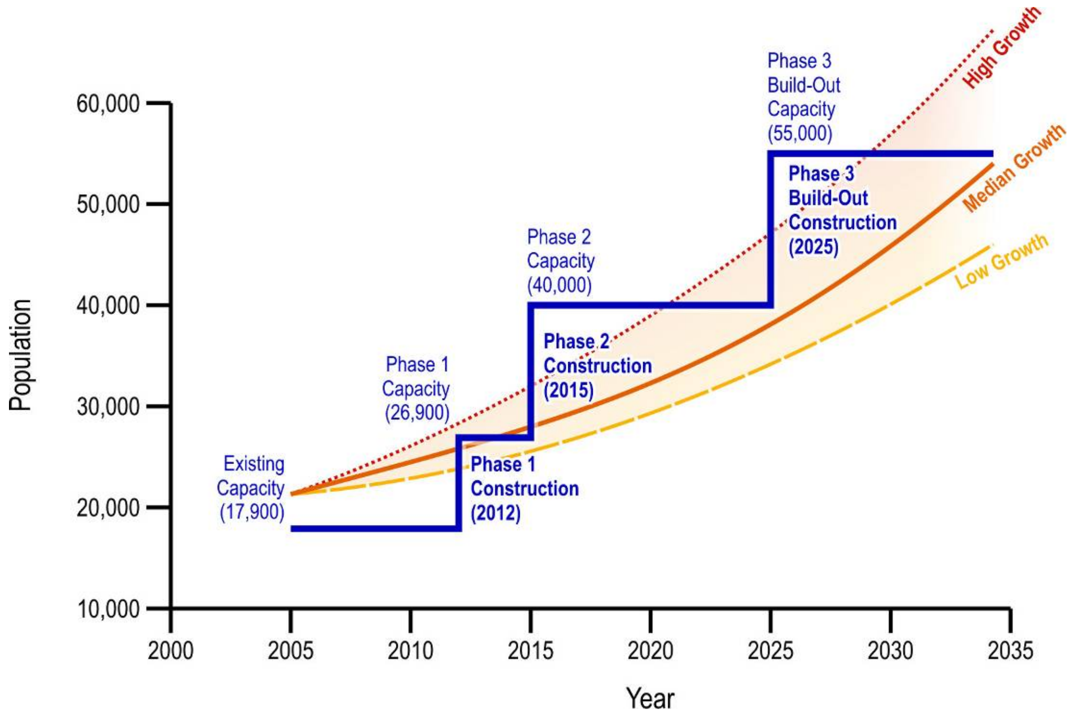
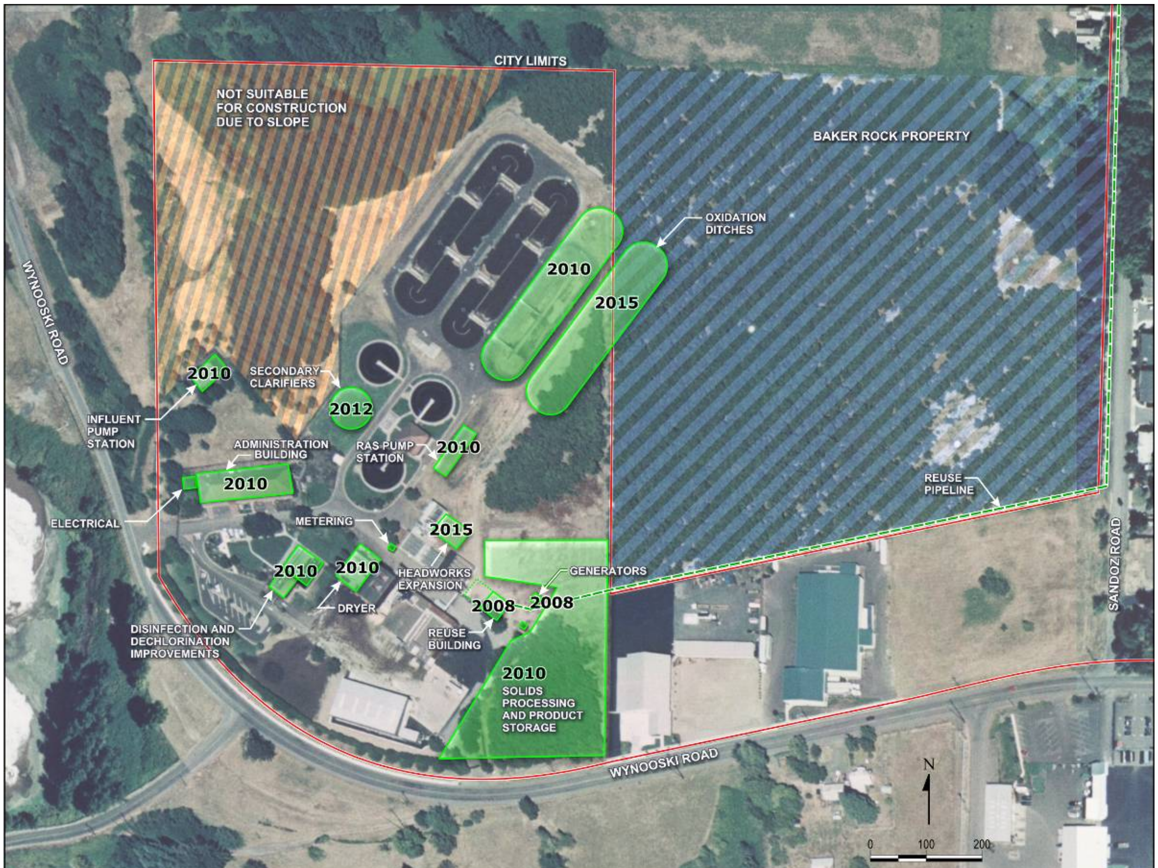


Figure ES-16. Planned Phased Construction Assuming no I/I Removal

ES.18.1 Phase 1, 2007 to 2015

The RRE projects that need to be completed immediately to provide service through 2015 are shown in Figure ES-17 and the order-of-magnitude costs are summarized in Table ES-9.



Newberg City Phase 1 Wastewater Expansion Proposal

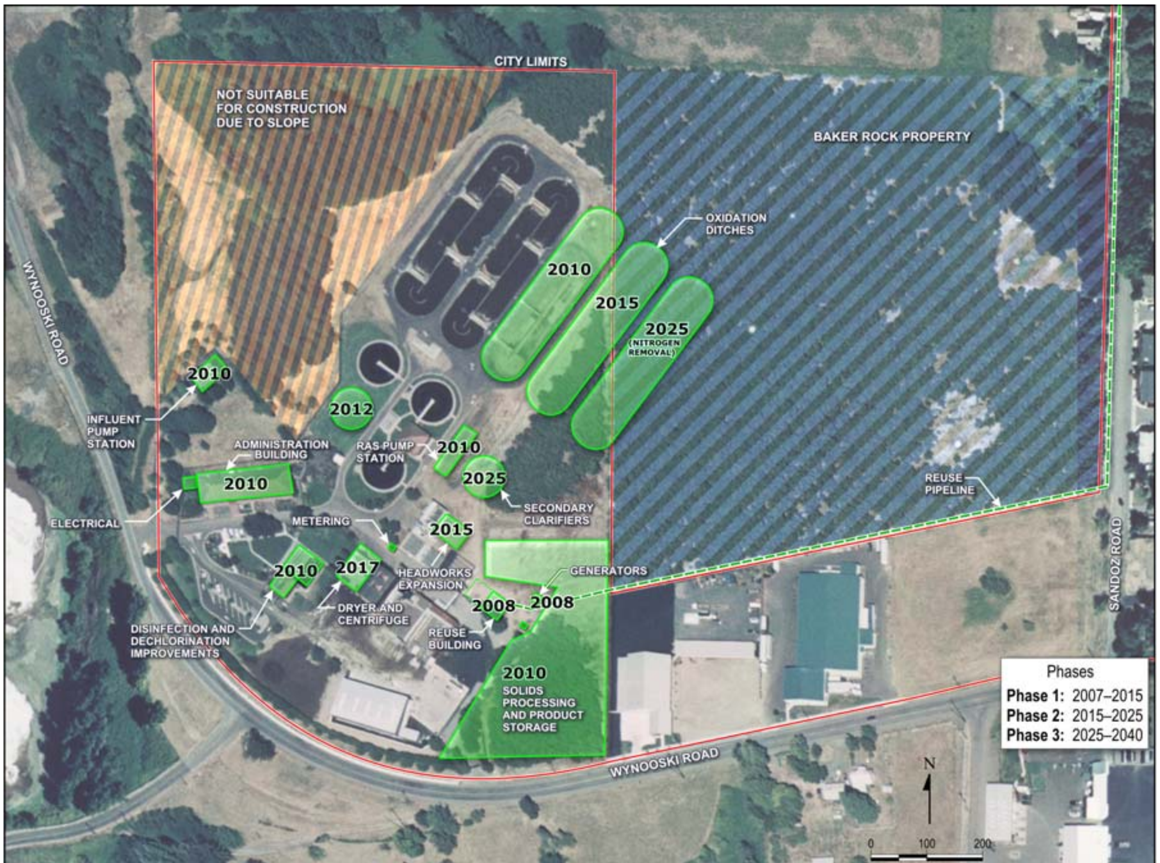
Figure ES-17. Recommended Improvements for Phase 1

Table ES-9. Capital Costs for Phase 1 Improvements 2007 to 2015

WWTP improvements	Cost, dollars	Comments
IPS and parallel discharge pipe	3,124,700	Needed immediately
Influent pipeline improvements	287,000	Needed with IPS.
Influent metering	250,000	
Headworks improvements	4,145,700	
Headworks odor control first phase	70,000	
Septage receiving	395,500	
Third and fourth oxidation ditch	6,565,000	
Existing oxidation ditch repairs	573,500	
Fourth secondary clarifier	3,251,200	
Splitter structures	650,000	
Disinfection	425,300	Needed immediately
Dechlorination	339,000	Needed immediately
Outfall	367,900	Needed immediately
In-plant reuse water	85,000	
In-plant stormwater pump station	474,300	
Building upgrades		
Chlorine Building	77,200	
Chlorine scrubber and duct	833,300	
Secondary Building	346,300	
Solids Handling Building	348,100	
Compost Building	468,400	
Sawdust dryer	533,000	2007 dollars; Needed immediately; Energy funding available
Level IV reuse facilities (by others) and storage	Not included	Provided by others
Administration Building	1,496,100	
Subtotal, construction cost	25,106,500	
Administration/engineering costs at 25 percent	6,276,600	
Total capital cost	**31,383,100**	**Escalated to 2011 mid-point of construction except as noted**

ES.18.2 Phase 2, RRE Projects for 2015 to 2025

The RRE projects that need to be completed to meet the Phase 2 needs from 2015 to 2025 are shown in Figure ES-18 and the order-of-magnitude costs are summarized in Table ES-10. The fifth oxidation ditch will be required in 2025 if nitrogen reduction becomes a regulatory requirement. If nitrogen reduction is not required, the fifth oxidation ditch will not be needed until 2040. For the purposes of cost estimating, it was assumed that nitrogen reduction was required to meet NPDES discharge requirements. This is a conservative estimate. Actual Phase 2 implementation will be based on the NPDES requirements at that time.



Newberg City Phase 2 Wastewater Expansion Proposal

Figure ES-18. Newberg WWTP Phase 2 Improvements 2015 to 2025

Table ES-10. Capital Costs for Phase 2 Improvements 2015 to 2025

WWTP improvements	Cost, dollars	Comments
IPS and parallel discharge pipe	N/A	
Influent pipeline improvements	N/A	
Influent metering	N/A	
Headworks (screenings and grit) improvements	N/A	
Headworks odor control first phase	300,000	Potential for more odor control
Septage receiving	N/A	
Fifth oxidation ditch	4,363,000	Assumes nitrogen reduction requirements
Fifth secondary clarifier	3,251,000	
Splitter structure	600,000	
Electrical building	500,000	
Disinfection	3,065,000	Assumes conversion to UV
Dechlorination	N/A	
Outfall	N/A	
In-plant reuse water	N/A	
In-plant stormwater pump station	N/A	
Building upgrades	N/A	
Composting expansion	3,283,500	
Centrifuge Dewatering	3,508,500	
Level IV reuse facilities and storage	N/A	Level IV reuse by others
Administration Building	N/A	
Subtotal, construction cost	18,871,000	
Administration/engineering costs at 25 percent	4,717,800	
Total capital cost	23,588,800	**(in March 2007 dollars)**

ES.18.3 Phase 3, RRE Projects for 2025 to 2040

The RRE projects to meet the ultimate buildout needs are shown in Figure ES-19. These were identified to define the potential land requirements to serve the population at ultimate build-out of the urban reserve area. Ultimate build-out is assumed to occur by 2040. The scenarios shown on Figure ES-16 include improvements needed to serve median growth estimates as well as improvements needed if high population growth estimates are realized.

Costs for the RRE projects for ultimate buildout are not included in the planning effort. The ultimate buildout was included to define the potential land requirements only.

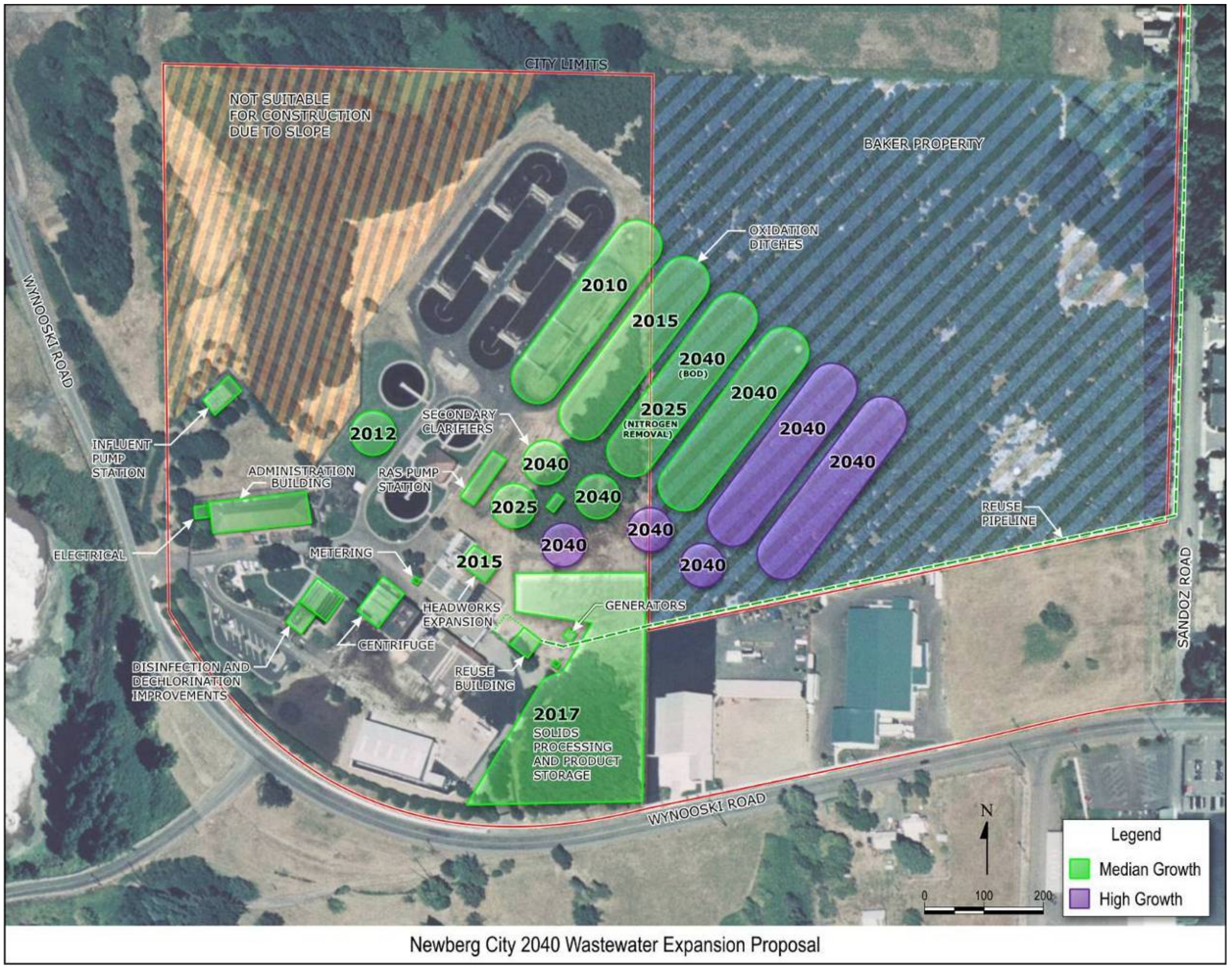


Figure ES-19. Newberg WWTP Ultimate Buildout Improvements

ES.19 MANPOWER REQUIREMENTS

A review of the current staffing level was conducted as part of the Facilities Planning process, and recommendations were developed for three future time periods relating to modifications and additions to the wastewater treatment plant (WWTP). Baseline information for current staffing is presented, as well as estimated staffing needs for the periods of 2007 to 2015, 2015 to 2025 and 2025 to 2040.

It is recognized that many factors will have an impact on staffing recommendations, and they will need to be updated as more of the elements of the plan become firm. Table 1 summarizes recommended staff additions over the course of the three phases of plant modifications and additions.

Table ES-11. Recommended Additional Personnel by Title

Position Title	No.	Responsibilities
Phase 1. 2007 to 2015		
Senior Laboratory Technician/Environmental Specialist	1	Perform water quality testing for the water treatment plant (WTP), WWTP, industrial pretreatment, and receiving stream
Operator II	1	Perform operation duties predominantly in the wastewater treatment plant.
Utility Worker	1	Perform multiple semi-skilled tasks in both the wastewater and water utilities
Phase 2. 2015 to 2025		
Plant Mechanic	1	Perform general mechanical work at the WWTP and assist maintenance workers on water system projects as needed
Senior Environmental Technician	1	Administer pretreatment program and drinking water program
Operator II	1	Perform operation duties in both the wastewater and water utilities
Phase 3. 2025 to 2040		
Operator I	1	Perform operation duties in the WWTP
Utility Worker	1	Perform multiple semi-skilled tasks in both the wastewater and water utilities
Environmental Technician	1	Perform routine sampling in both the wastewater and water utilities

ES.20 PUBLIC INVOLVEMENT

The City held an open house for the public on October 17, 2006, to provide outreach on public works projects, including the Newberg WWTP Facilities Plan Update. The handout explaining the WWTP Facilities Plan Update and Collection System Master Plan is included in Appendix C.

CHAPTER 1
STUDY AREA CHARACTERISTICS AND BASIS OF PLANNING

The City of Newberg (City) owns and operates a secondary wastewater treatment facility located in Newberg, Oregon. The Newberg Wastewater Treatment Plant (WWTP) is located at 2301 Wynooski Road, and was placed into service in 1987. The facility is a Class IV oxidation-ditch type, activated sludge plant with Class A in-vessel biosolids composting. The treatment train consists of influent pumping, screening and grit removal, oxidation ditch activated sludge, clarification, solids dewatering, composting, odor control, chlorination, dechlorination, and effluent discharge to the Willamette River. Key improvements include the 1997 Instrumentation and Control Improvements, the 2004 Headworks project, and the 2004 Composter Odor Control project. The City is implementing a reuse system for irrigation of golf courses to be operational by 2008.

The City currently provides wastewater collection and treatment services to its residents, commercial establishments, institutional customers, and a number of industries. Sewer service is provided only to customers within the city limits, with the exception of a few residences outside of the city and SP Newsprint Company, which discharges only domestic wastewater to the municipal system. The last Facilities Plan was completed as part of the Sewerage Master Plan Update (KCM, 1985), after which the City constructed the existing WWTP on Wynooski Road with federal grants. The purpose of the Newberg WWTP Facilities Plan Update is to provide the planning for required modifications to meet projected growth within the urban growth boundary (UGB) and the urban reserve area (URA) to maintain compliance with its National Pollutant Discharge Elimination System (NPDES) permit and potential future regulations.

In this chapter, the study area is defined and its characteristics are summarized. Recommended improvements will be phased over a 20-year period (to 2025) to address both immediate needs, as well as to provide long-term planning for the year 2040, to guide the future direction of capital improvements projects and define the land area needed for the WWTP.

The project goals are listed below.

- Plan for facilities to comply with existing and predictable-potential-future regulations.
- Provide for incremental capacity expansion through 2025 through a Capital Improvement Program (CIP) program and ultimate expansion in 2040.
- Provide for reliability, ease of operations and maintenance (O&M), and safety.
- Plan an efficient Administration Building office space and laboratory.
- Recommend back-up power engine generator requirements.
- Coordinate with the Reuse Water Project. The Reuse Water Project includes the new back-up power engine generator.
- Evaluate staffing requirements for the existing and future WWTP operations.

The facilities planning process includes the following:

- Using existing and future population projections (residential, commercial, and industrial) to estimate significant increases in wastewater flows and loads.
- Identifying capacity needs to ensure that the WWTP will be able to convey and treat the wastewater through service area build-out.
- Evaluating treatment processes with regard to growth and regulatory requirements and identifying needed plant expansions and improvements.
- Developing recommendations to modernize and optimize the system where practicable, and to ensure compliance with local, state, and federal regulations.
- Evaluating and recommending energy efficient alternatives.
- Developing planning-level cost estimates (with 35 percent contingency) for the recommended improvements based on current cost estimating practices and the timing of the improvements.
- Creating a recommended CIP for the facility in increments for the fiscal year period 2007 through 2025.
- Evaluating the Administration Building to meet current and future staff needs (including those of a new water treatment plant (WTP), to be located south of the WWTP across Wynooski Road).

1.1 PROJECT BACKGROUND

The Newberg WWTP is a biological activated sludge plant that uses the oxidation-ditch process, as depicted in Figure 1-1. Treated effluent and biosolids are the two main products that treatment plants produce. Treatment plant effluent is discharged to the Willamette River or beneficially reused by irrigating at the plant site. Beneficial reuse of effluent is seasonal because crops uptake water from about May through September. Biosolids are composted in-vessel to Class A compost and sold to the public.

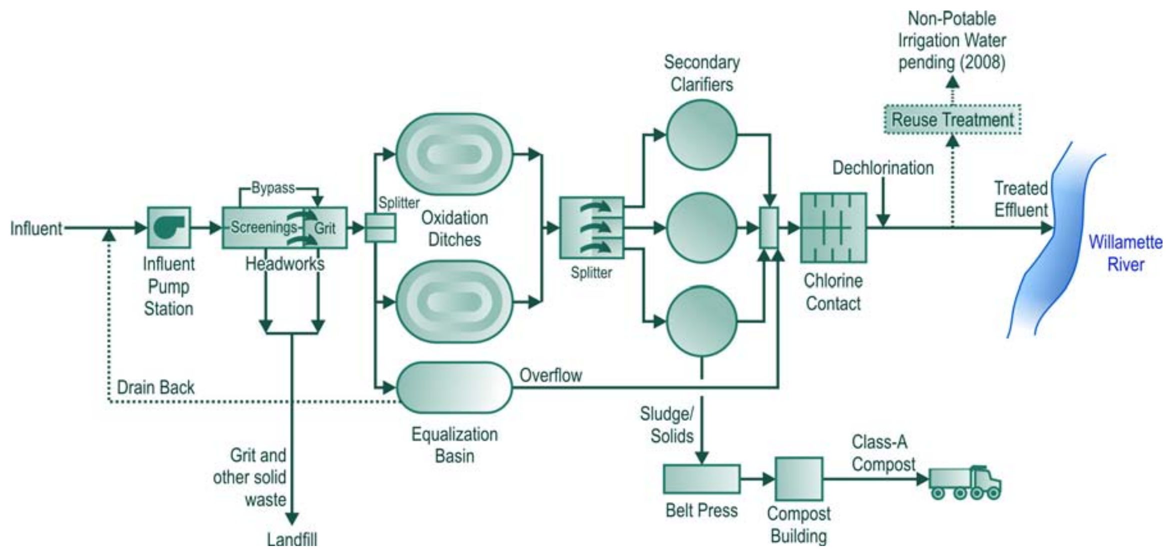


Figure 1-1. Schematic of Newberg WWTP

The treatment facility serves the entire city. Very little service is currently provided outside the city limits and it is provided to only a few hardship cases approved by the City Council. The facilities consist of a raw influent pump station, headworks, activated sludge oxidation ditches, secondary clarifiers, chlorine disinfection, dechlorination, effluent outfall, and biosolids composting. The plant accepts septage from local septic pumpers on a regular basis.

The raw sewage pump station contains four centrifugal pumps that convey all wastewater to the headworks. The headworks currently consist of two influent channels with mechanically cleaned perforated screens with washer/compactors and one bypass pipe. The facility has an aerated grit removal system with two cyclone grit classifiers.

Secondary treatment is provided by two oxidation ditches and three circular 80-foot-diameter, center-feed secondary clarifiers. An equalization basin is designed to take influent flows above 18 million gallons per day (mgd) for peak flow storage. However, depending on the influent pumping and number of oxidation ditches in service, flows below 18 mgd are sometimes diverted. Flows from the equalization basin are overflowed to disinfection or drained back to the influent pump station.

Disinfection is performed with chlorine gas. Treated and disinfected effluent is dechlorinated with sodium bisulfite prior to flow measurement and discharge. The outfall is a single port diffuser in the Willamette River at River Mile 49.7.

The facility is unmanned at night, but it has 24-hour monitoring of alarms through telemetry. The facility also has redundant power and some standby electric generation at the treatment plant. Some lift stations have standby power generators while others can be served by mobile generators.

Waste activated sludge from the secondary clarifiers (and solids dredged from the oxidation ditches when necessary) is pumped to the solid treatment processes. Sludge is stored in two 80,000-gallon sludge storage tanks or directly thickened with two 2-meter belt filter presses. Dewatered sludge is mixed with sawdust and recycled compost, and enters one of two compost reactor vessels to produce Class A biosolids. Each reactor tunnel is 18 feet wide, 12 feet high, and 66 feet long. At maximum loading, corresponding to seasonal variations in feed moisture content, each tunnel has a detention time of approximately 14 days.

Modifications have been made to the facility since 1987. Dechlorination of the effluent was implemented toward the end of 1998. Headworks modifications and compost odor control were added in 2004. With the new headworks, peak instantaneous hydraulic capacity has been rated at 27 mgd.

Population, land use patterns, and economic growth within the UGB determine wastewater treatment system demands and design capacities. For this study, the planning period is 20 years, from 2005 through 2025, and long-term in the year 2040 within the UGB. This section presents population projections based on historical data for the City.

There is a possibility that the City's WWTP may accept sewage from the City of Dundee at some time in the future. When Dundee requests connection to the Newberg system, the City will look at the flow and load impacts on the existing and future capacity of the WWTP.

1.2 PHYSICAL AND SOCIOECONOMIC FACTORS (POPULATION)

Newberg is located in Yamhill County along U.S. Highway 99W, 23 miles southwest of Portland and 12 miles northeast of McMinnville, the county seat of Yamhill County. The study area is shown on Figure 1-2, including the URAs. The boundary of the study area is the City's UGB, as defined by the most recently adopted Comprehensive Plan (revised November 2004). The study boundary generally extends to the expected limits of urban development. The City recently completed a citizen study of the URA and UGB and has recommended changes (expansions) which will likely to be implemented. The recommendations are depicted in Figure 1-3.

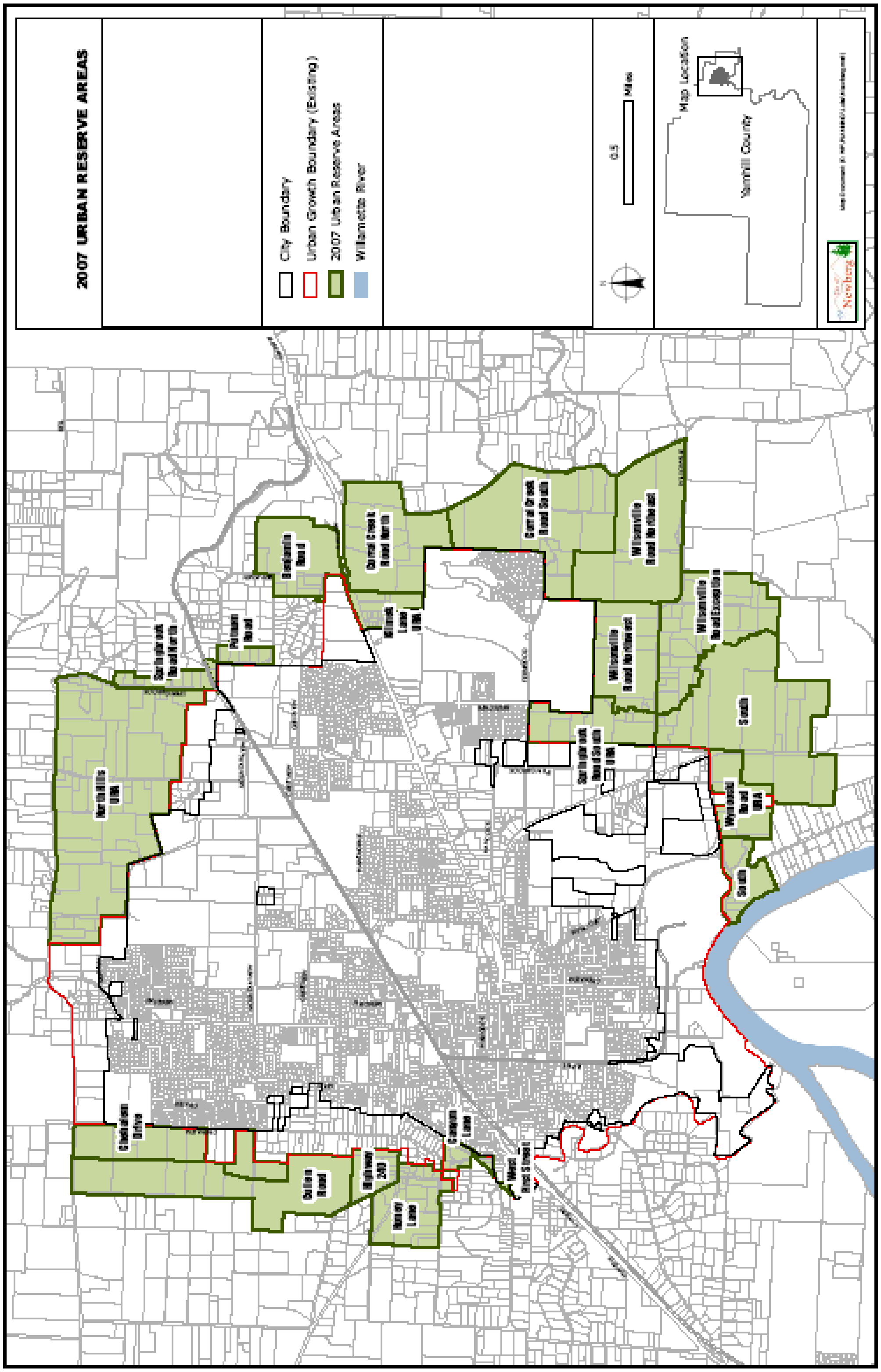


Figure 1-2. URAs

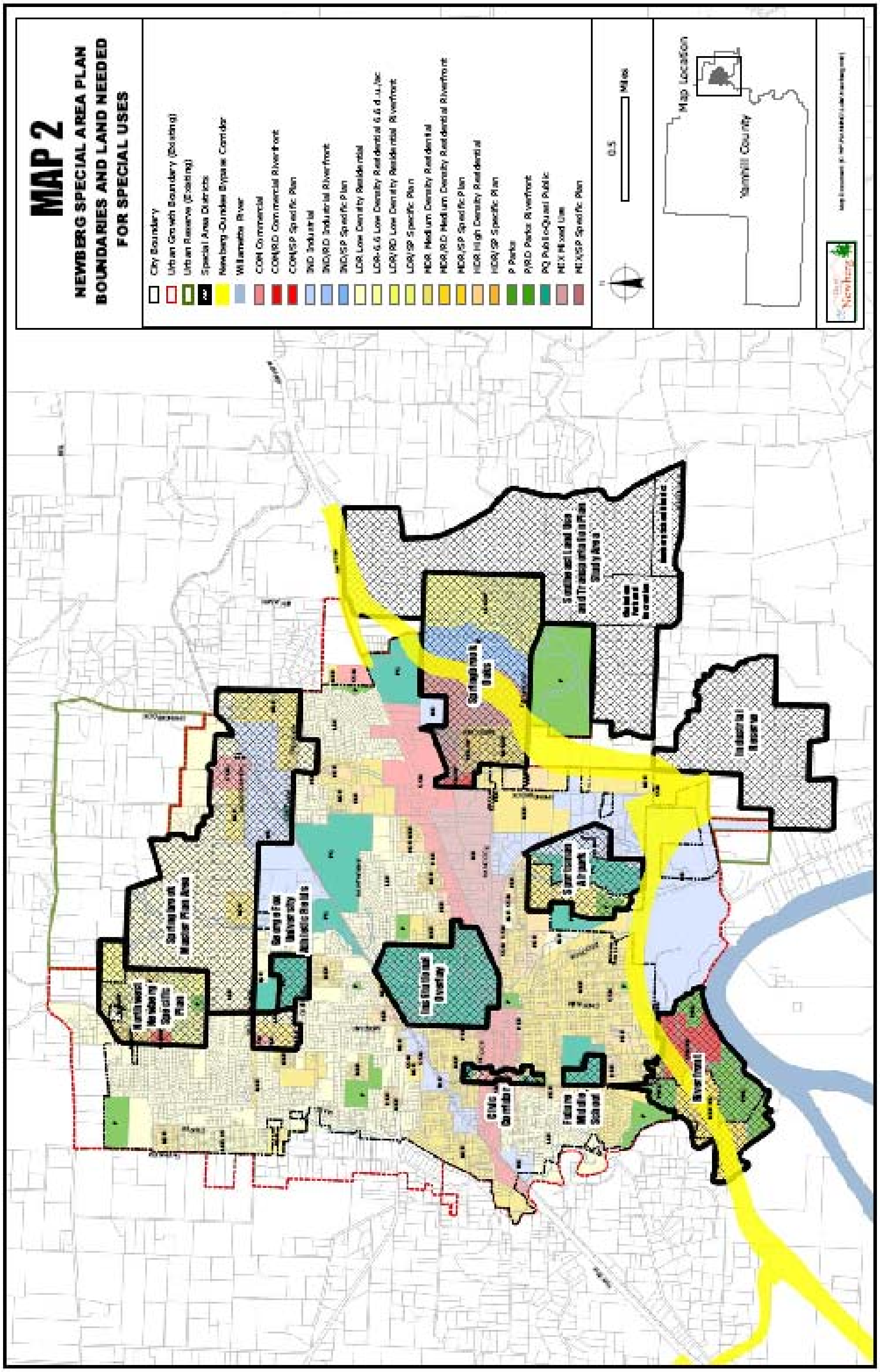


Figure 1-3. URA and UGB Expansions

1.2.1 Physical Environment

The physical environment includes the topography, geology, soils, climate, and water resources of the region.

Topography. Newberg is situated on an elevated terrace just north of the Willamette River at the confluence of the Chehalem Creek and Willamette River valleys. The terrace lies at an elevation of about 160 feet mean sea level and is quite level, with slopes ranging between 0 and 3 percent. This level terrain facilitates development for residential, commercial, and industrial uses. Surrounding the terrace on three sides are hills: to the north is the Chehalem Mountain group; to the east is Parrett Mountain; and to the southwest are the Red Hills of Dundee. To the south is the Willamette River. These topographic features form an envelope around the terrace within which the City can reasonably be expected to develop. In the north of the city, the land slopes increase at the foot of Chehalem Mountain to a degree that would inhibit development to urban densities and limit the economical extension of utility services such as water and sewer.

Geology. Newberg lies in the Willamette River Basin, a very fertile agricultural area. The upper terrace on which Newberg is sited is underlain by Willamette silts. Lying approximately 60 feet below the surface is the Troutdale Formation. Characteristically, the Willamette silts are well drained, with moderate permeability. Agriculturally, Willamette silts are used for grapes, orchards, vegetables, berries, and small grain crops. Some pasture use and hay production also occur.

Climate. With an elevation of only 160 feet above sea level and a location relatively close to the Pacific Ocean, Newberg enjoys a very moderate climate. The annual air temperature is 53 degrees Fahrenheit (F). The average high temperature is 65 degrees F (16 degrees Celsius[C]) and the average low temperature is 39 degrees F (7 degrees C). The annual average rainfall is 42 inches. Rainfall occurs predominantly during the fall, winter, and spring. The summers are warm and dry, often approaching drought conditions for 60 to 90 days during July, August, and September. Water demand peaks during this period. The local growing season is approximately 174 days.

Water Resources. The Newberg water supply system includes two types of water sources: springs and wells. Four springs have provided up to 420,000 gallons per day to the Newberg system. Snider, Skelton, Reynolds, and Oliver Springs are located to the north of town at the foot of Chehalem Mountain. The Reynolds Spring is no longer being used. Water from the springs flows directly into the distribution system. The fifth spring system, Otis, is located to the northeast of Newberg at the foot of Rex Hill, with a production capability approximately 0.35 mgd of non-potable irrigation water. Otis Springs has been determined to be "surface water influenced" by the Oregon Department of Environmental Quality (DEQ) and no longer is connected to the City's potable water system. The four previously mentioned springs operate by gravity. All of these are constant, year-round sources of water that need no treatment other than chlorination.

The City placed new wells in operation in 2002 and 2006, and the well water is treated and distributed. The City is looking at augmenting water resources with reuse water for irrigating local golf courses, cemeteries, parks, etc.

1.2.2 Land Use

The UGB is shown in Figure 1-2. Land use and development are largely governed by the local topography and the intersecting U.S. Highway 99W.

The commercial district extends along U.S. Highway 99W and is concentrated in downtown. A hospital and a dental equipment manufacturer are in the service area. Commercial establishments are largely service-oriented. The rural areas are mostly farms and vineyards.

The Newberg-Dundee Transportation Improvement Project is planning a transportation corridor that will be adjacent to the northeast corner of the WWTP, as shown in Figure 1-4. The corridor is not expected to impact the plant's footprint, as only approximately two-thirds of the area shown is needed for the roadway.

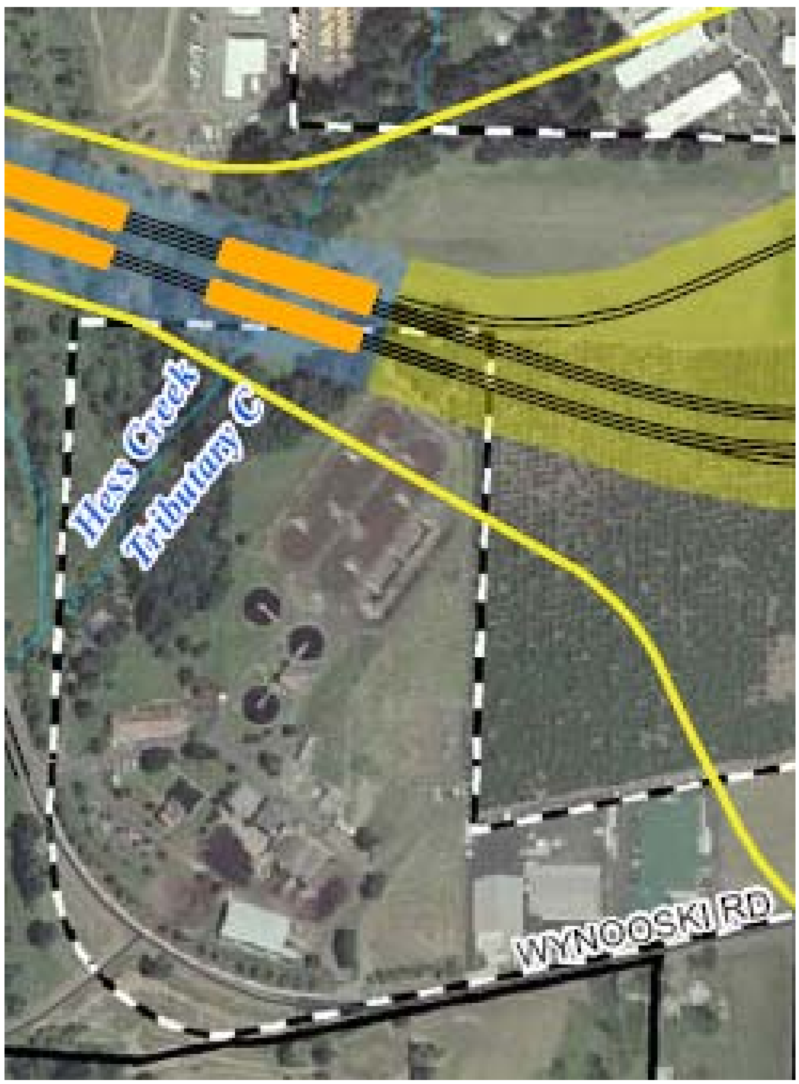


Figure 1-4. Highlight of Newberg-Dundee Transportation Improvement Project Corridor Adjacent to the Newberg WWTP

This footprint of the transportation interchange and the approved bypass corridor shows impacts on the existing oxidation basins, and could potentially restrict the use of the Baker Rock property. This issue will need to be resolved with the Oregon Department of Transportation in order to properly plan for expansion of the WWTP.

In addition, the City is considering relocating the local animal shelter on land adjacent to the WWTP. Siting in this area could limit the expansion capability of the WWTP to the east.

The City intends to site its future potable WTP near the site of the existing WWTP. Other City facilities are also being considered for relocation to adjacent properties to the northeast and east (known as the Baker Rock Property). This Facilities Plan will define the WWTP site requirements for 2040 so that adequate land can be made available for future expansion.

1.2.3 Population within the UGB

Newberg is a fast-growing community in the Willamette Valley. Its current population is just over 21,000 people. Population projections for the service area are integral to projecting sewage flows. The population forecasts for the Newberg UGB are from the Johnson Gardner Report (July 2004). These forecasts are in line with those officially adopted by the City in December 2005. Table 1-1 and Figure 1-5 list the projected populations for a range of potential growth rates: low growth, median growth, and high growth. The median projected population in 2025, and at the end of the planning period in 2040, are 38,352 and 54,097, respectively. The median growth rate was used for planning the phased CIP. The high growth rate was used to determine land area requirements for ultimate build-out.

Table 1-1. Population Projections

Year	Low growth population	Median growth population	High growth population
2005	20,623	21,132	22,180
2010	23,332	24,497	26,985
2015	26,691	28,712	33,190
2020	30,561	33,683	40,859
2025	33,957	38,352	48,833
2030		43,600[1]	
2040	44,505	54,097	79,701

[1] Population estimate for 2030 based on straight line projection from 2025 to 2040.

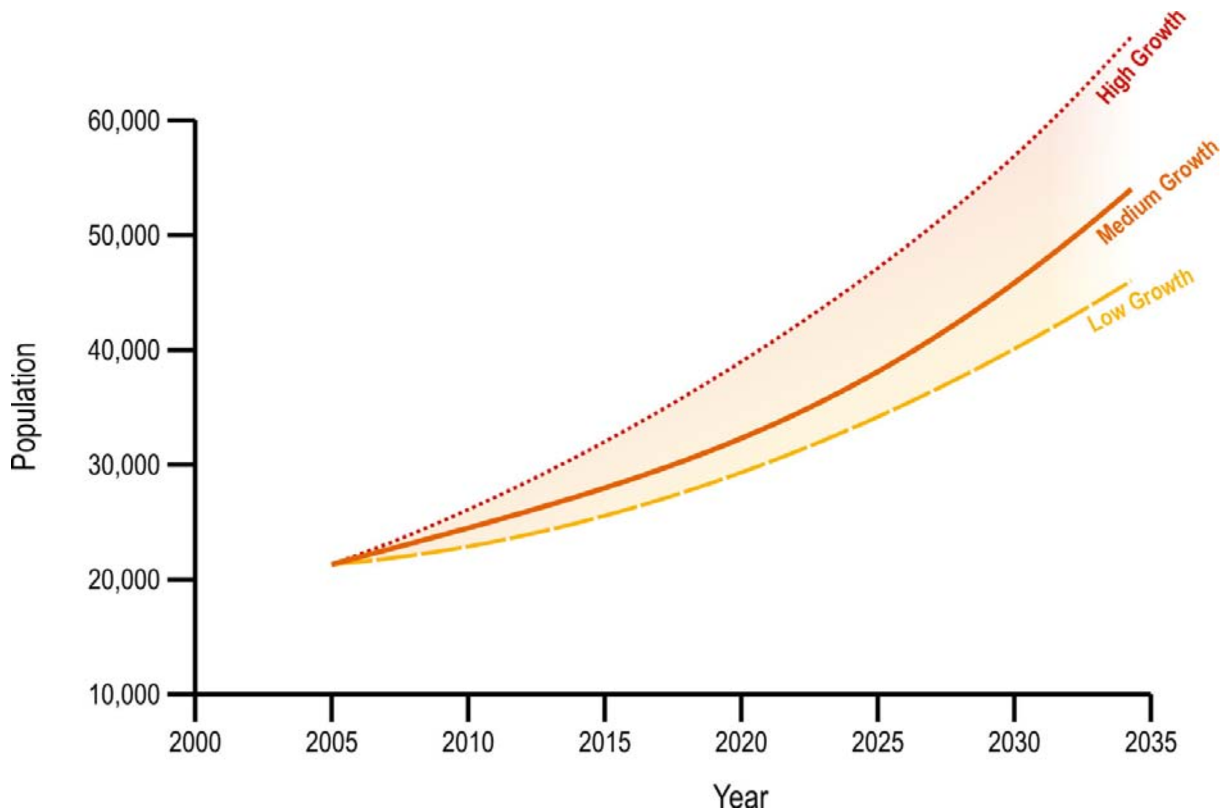


Figure 1-5. Newberg Population Projections, 2004

1.3 PERMIT REQUIREMENTS

The City operates the WWTP that discharges to the Willamette River, as shown in Figure 1-6, and manages the biosolids as a Class A product. The City is considering the development of a reuse program to irrigate the land adjacent to the WWTP, a local golf course, and other venues. These activities are regulated by an NPDES permit in conjunction with the City's Biosolids Management Plan. Each plan includes management options, monitoring requirements, and analysis of any previously collected data to ensure that groundwater is protected. This section details the regulatory history, monitoring requirements, information about future regulations, and their possible affects on the Newberg WWTP. It also evaluates other code compliance issues.

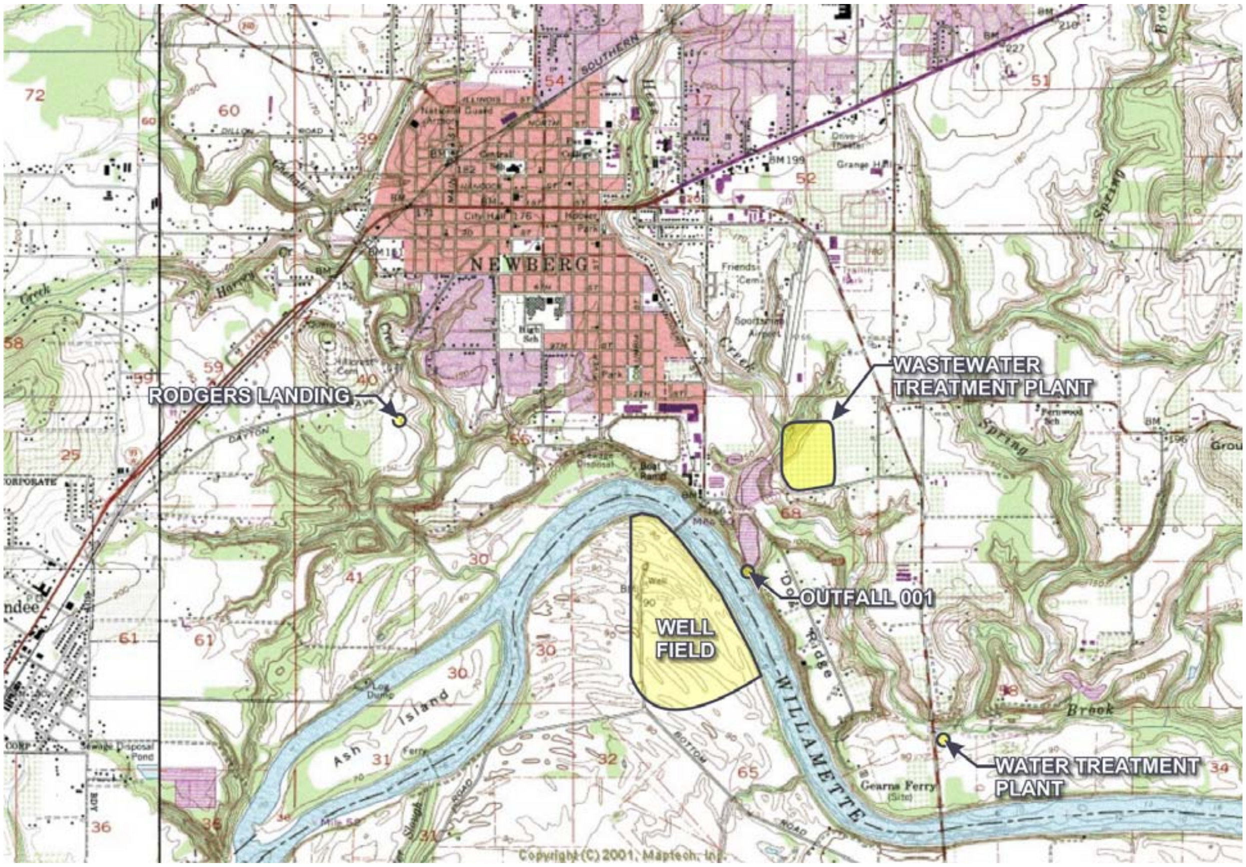


Figure 1-6. Newberg WWTP Location

The following subsections summarize current and proposed regulations and establish the design criteria to be used in the development of the various treatment and disposal alternatives for the City's wastewater treatment system. The criteria listed include the DEQ criteria for the Willamette Basin standards, Willamette River discharge criteria, reuse criteria for land application of effluent and biosolids, and U.S. Environmental Protection Agency (EPA) criteria for reliability and redundancy.

1.3.1 NPDES Permit Requirements

The City was issued an NPDES permit on June 22, 2004, for its Level III Collection System and Level IV treatment system that discharges to the Willamette River. A copy of this permit is included in Appendix A. The City directs all treated water to the Willamette River. This practice is covered by the NPDES permit. The current permit requirements are listed in Tables 1-2 through 1-4.

Table 1-2. Current Permit Requirements, May 1 to October 31

Parameter	Limitation				
	Concentration, milligrams per liter (mg/L)		Mass load[1]		
	Monthly average	Weekly average	Monthly average, pounds per day (ppd)	Weekly average, ppd	Daily maximum, pounds
5-day carbonaceous biochemical oxygen demand ($CBOD_5$)[2]	10	15	330	500	660
Total suspended solids (TSS)	10	15	330	500	660

[1]The daily mass load limit is suspended on any day in which the daily flow to the WWTP exceeds 8 mgd (twice the design average dry weather flow).

[2]The $CBOD_5$ concentration limits are considered equivalent to the minimum design criteria for BOD_5 specified in Oregon Administrative Rules (OAR) 340-041. These limits and $CBOD_5$ mass limits may be adjusted (up or down) by permit action if more accurate information regarding $CBOD_5/BOD_5$ becomes available.

DEQ can and has altered the Table 1-2 requirements to extend for longer periods (into November) based upon prolonged hot dry weather patterns and low river flows.

Table 1-3. Current Permit Requirements, November 1 to April 30

Parameter	Limitation				
	Concentration		Mass load[1]		
	Monthly average, mg/L	Weekly average, mg/L	Monthly average, ppd	Weekly average, ppd	Daily maximum, pounds
$CBOD_5$[2]	25	40	1,400	2,000	2,700
TSS	30	45	1,600	2,400	3,200

[1] The daily mass load limit is suspended on any day in which the daily flow to the WWTP exceeds 8 mgd (twice the design average dry weather flow).

[2] The $CBOD_5$ concentration limits are considered equivalent to the minimum design criteria for BOD_5 specified in OAR 340-041. These limits and $CBOD_5$ mass limits may be adjusted (up or down) by permit action if more accurate information regarding $CBOD_5/BOD_5$ becomes available.

Table 1-4. Current Permit Requirements, Year Round

Parameter	Permit requirement
E. coli	Shall not exceed 126 organisms per 100 milliliters (mL) monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. If a single sample exceeds 406 organisms per 100 mL, then five consecutive resamples may be taken at 4-hour intervals beginning within 28 hours after the original samples were taken. If the log mean of the five resamples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.
pH	6 to 9
$CBOD_5$ and TSS removal efficiency	Shall not be less than 85 percent monthly average for $CBOD_5$ and 85 percent monthly for TSS.
Total residual chlorine	Shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum concentration of 0.05 mg/L. When the total residual chlorine limitation is lower than 0.10 mg/L, DEQ will use 0.10 mg/L as the compliance evaluation level (i.e., daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitation).

The permit contains $CBOD_5$ and TSS removal efficiency limits of 85 percent.

DEQ has developed a statewide permit issuance plan that moves the water quality permits throughout the state on the same permit schedule within a basin. That means that some permits may be renewed prior to their expiration date, and some permits may be administratively extended. The Newberg NPDES permit renewal date as stated on the permit is May 31, 2009. DEQ has scheduled the actual renewal for 2012 to coincide with the basin permitting schedule.

In 2006, the City requested that the permit be re-opened to include a reuse outfall for irrigation on local golf courses, an approved overflow manhole in Hess Creek for the pump station, and acceptance for the revised Biosolids Management Plan. DEQ is currently working on the open permit.

1.3.2 Monitoring Requirements

The City is required to measure plant flow. The current monitoring requirements are listed in Table 1-5. The facility effluent's cyanide, bacteria, pH, chlorine residual grab samples, and all measurements are taken from the Cipolletti weir discharge. Composite and metals samples are taken just before the Cipolletti weir. The composite sampler is located in the reclaimed water pump room.

Table 1-5. Current Monitoring Requirements

Item or parameter	Minimum frequency	Type of sample
Total flow, mgd	Daily	Calculated based on the influent flow and adjusted by measured and/or estimated side stream flows.
Flow meter calibration	Semiannually	Verification
$CBOD_5$	2 per week	Composite
Ammonia (NH_3-N)	2 per week	Composite
TSS	2 per week[1]	Composite
Hardness (mg/L $CaCO_3$)	[1]	Grab
pH	3 per week	Grab
Effluent temperature, daily maximum	Daily	Continuous (Record time between readings; submit annual report; audit continuous monitors in June and December using DEQ guidelines; and visually check monitors monthly to ensure that they are still in place and submerged.)
E. coli	2 per week	Grab (use method number 9213 D, 9222 G, 9223 B in *Standard Methods for the Examination of Water and Wastewater, 19th Edition* or any test procedure that has been authorized by DEQ.)
Quantity chlorine used	Daily	Measurement
Total chlorine residual	Daily	Grab
Pounds discharged, $CBOD_5$ and TSS	2 per week	Calculation
Average percent removed, $CBOD_5$ and TSS	Monthly	Calculation
TKN, NO2+NO3-N, total phosphorus	1 per week	24-hour composite
Metals (Ag, As, Cd, Cr, Cu, Hg, Mo, Ni, Pb, Se, Zn) and cyanide, measured as total in mg/L[1]	Semiannually using three consecutive days between Monday and Friday, inclusive	24-hour daily composite For cyanide samples, at least six discrete grab samples shall be collected over the operating day. Each aliquot shall not be less than 100 mL and shall be collected and composited into a larger container, which has been preserved with sodium hydroxide for cyanide samples to ensure sample integrity. Daily 24-hour composite samples shall be analyzed and reported separately. Toxic monitoring results and toxics removal efficiency calculations shall be tabulated and submitted with the Pretreatment Program Annual Report as required in Schedule E of the NPDES permit.
Iron	Monthly	24-hour daily composite During the first year after permit issuance, monitoring for iron shall be conducted on the effluent at the frequency specified. The method detection limit must be lower than 0.3 mg/L. After the first year, iron monitoring of the effluent may be eliminated unless the City is notified otherwise in writing by DEQ. For all tests, the method detection limit shall be reported along with the sample result.

Table 1-5. Current Monitoring Requirements (continued)

Item or parameter	Minimum frequency	Type of sample
Priority Pollutants. The City shall perform all testing required in Part D of EPA Form 2A. The testing includes all metals (total recoverable), cyanide, phenols, hardness, and the 85 pollutants included under volatile organic, acid extractable, and base-neutral compounds. In addition, the City shall monitor for the pesticide pollutants listed in Table II of Appendix D of 40 CFR §122.	Three scans are required during the 4½ years after permit issuance. Two of the three scans must be performed no fewer than 4 months and no more than 8 months apart. The effluent samples shall be 24-hour daily composites, except where sampling volatile compounds. In this case, six discrete samples (not less than 100 mL) collected over the operating day are acceptable.	24-hour daily composite The City shall take special precautions in compositing the individual grab samples for the volatile organics to ensure sample integrity (i.e., no exposure to the outside air). Alternatively, the discrete samples collected for volatiles may be analyzed separately and averaged.
Whole effluent toxicity (WET)	Annually	Acute and chronic Beginning in 2004, the City shall conduct WET testing for a period of 4 years in accordance with the frequency specified. If the WET tests show that the effluent samples were not toxic at the dilutions determined to occur at the zone of initial dilution (ZID) and the mixing zone, no further WET testing will be required during this permit cycle. Note that four WET test results will be required along with the next NPDES permit renewal application.

[1] During the first 2 years after permit issuance, special monitoring for cadmium, copper, lead, mercury, and silver shall be conducted on the effluent during at least one of the 3 consecutive days of monitoring. TSS and hardness shall be monitored simultaneously. The special monitoring for cadmium, copper, lead, and silver shall be conducted using a clean sampling method, and ultra-clean sampling method, EPA Method 1669 or any other test method approved by DEQ. The special monitoring for mercury shall be conducted in accordance with EPA Method 1631. At the City's option, the results of the special monitoring may be used for one or more of the 3 consecutive days of monitoring required on a semiannual basis. After the first 2 years, special monitoring of the effluent for cadmium, copper, lead, mercury, and silver may be eliminated unless otherwise notified in writing by DEQ. For all tests, the method detection limit shall be reported along with the sample result.

1.3.3 Other Permit Requirements

There are other permit requirements with which the WWTP must comply. These are discussed below.

Sanitary Sewer Overflows (SSOs). Collection system overflows can result from catastrophic failure of the treatment plant or pump station or high flows due to storm events. According to the Newberg NPDES permit and OAR 340-041-0009(6) and (7), the City is prohibited from discharging raw sewage during the period of November 1 through May 21, except during a storm event greater than the 1-in-5-year, 24-hour duration storm. In addition, it cannot discharge raw sewage during the period from May 22 through October 31, except during a storm event greater than the 1-in-10-year, 24-hour duration storm. The City has seven emergency overflow outfalls at the City's pump stations, except the influent pump station. During the current permit period, there has been one surcharge to Hess Creek near the influent pump station during the December 14, 2006 storm. This storm event had a return frequency of between 1 in 25 and 1 in 50 years. The City plans to eliminate SSOs by December 31, 2010.

85 Percent Removal. The permit contains $CBOD_5$ and TSS removal efficiency limits of 85 percent.

Groundwater. All wastewater and process-related residuals shall be managed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

Outfall. The NPDES permit allows the treatment facility to discharge treated effluent into the Willamette River approximately one-third mile southwest of the plant at River Mile 49.7. Treated, disinfected, and dechlorinated effluent is discharged through a pipe to an outfall in the Willamette River.

Mixing Zone Analysis. The current permit provides for a mixing zone that consists of the portion of Willamette River contained within a band extending out 75 feet from the west bank of the river and extending from a point 15 feet upstream of the outfall to a point 150 feet downstream from the outfall. The ZID is defined as the portion of the allowable mixing zone located within 15 feet of the point of discharge.

The defined mixing zone overlaps with the defined mixing zone for the SP Newsprint Company's discharge. However, the effluent plumes do not mix until they are outside the defined mixing zones. The two outfall pipes are at different elevations and the plumes are vertically separated. For the WWTP, outfall, and mixing zone as currently configured, the *Combined Mixing Zone Study*, prepared by Parametrix in November 1993, determined that the dilution factor at the edge of the ZID is 2.4 during critical low stream flow conditions, while the dilution factor at the edge of the mixing zone is 28.5 during critical low flow conditions. The City will complete a new mixing zone study as required by their permit modification that will look at the issue of overlapping mixing zones. A copy of the permit modification letter is in Appendix A.

Temperature. Prior to the completion of a Willamette River temperature total maximum daily load (TMDL), requirements each NPDES point source that discharges into a temperature water quality limited water body is allowed a human use allowance. Each point source may cause the temperature of the water body to increase up to 0.3 degree C above the applicable criteria after mixing with either 25 percent of the stream flow or at the edge of the mixing zone, whichever is more restrictive.

Based on the existing discharge and the 7Q10 river flow, which is the annual flow for seven consecutive days that has a recurrence interval of 10 years, DEQ calculated in-stream temperature increases (using the existing facility design flow and maximum effluent temperatures) by two separate methods as required by OAR 340-041-0028(12)(b):

- Based on 25 percent of the 7Q10 stream flow
- Based on the estimated dilution achieved in the mixing zone at 7Q10 stream flow

Because the in-stream temperature increase is significantly smaller than the allowable increase, this facility has no reasonable potential to violate the temperature standard. Therefore, a summer period excess thermal load limit has not been included in the City's current permit.

A similar evaluation was performed for the winter period. There is no reasonable potential for this facility to violate the temperature standard, and therefore no winter period excess thermal load limit is included in this permit.

Since permit issuance, the Willamette TMDL was completed and wasteload allocations have been determined for the Newberg WWTP based on a 7Q10 river flow of 5460 cfs. They are listed in Table 1-6. The new wasteload allocations are expected to be included in the permit.

The composite sampler is located in the reclaimed water pump room.

Table 1-6. Newberg WWTP Temperature TMDL Allocations

January 1 through December 31 salmon and steelhead migration corridor					
Lookup Table waste load allocation (WLA)					
Flow (Q_{River})	T_{RC}	Human use allowance (HUA)	WLA	HUA	WLA
River flow greater than, cfs	River temperature criteria, C	a = 0 Allowed temperature increase, C	a = 0 Thermal load, million kcals/day	a> 0 Allowed temperature increase, C	a > 0 Thermal load, million kcals/day
0	20.0	0.0026	35	0.0024	32
6,289	20.0	0.0023	35	0.0021	32
6,531	20.0	0.0023	37	0.0021	34
7,664	20.0	0.0020	38	0.0018	34
Lookup Table WLA					
9,353	20.0	0.0017	39	0.0016	37
16,454	20.0	0.0012	48	0.0011	44
118,140	20.0	0.0005	145	0.0005	145

Iron Issues. DEQ does not have any information concerning the discharge of iron from this source. The permit requires the City to monitor the effluent for iron monthly for one year after permit issuance. Since the WTP recently started pumping its backwash water to the WWTP, DEQ added a second year of monitoring. This monitoring will enable DEQ to determine if iron in the discharge has a reasonable potential for causing or contributing to water quality standard violations. DEQ may require additional monitoring or reopen the permit to include new limits, conditions, or requirements if it is determined that this discharge causes or contributes to the violations of the in-stream iron criteria.

Other Toxics (PCB, Aldrin, Dieldrin, DDT, DDE). The TMDL does not address these pollutants, but it is not likely this discharge is a significant source of these pollutants. However, DEQ does not have any information concerning the discharge of these pollutants under this permit. Therefore, the permit includes the pesticide fraction in the annual monitoring of priority pollutants in Schedule B.

1.3.4 Groundwater

The treatment plant is constructed entirely of impervious structures. It is not anticipated that the treatment process and discharge to surface waters will cause groundwater impacts.

1.3.5 Biosolids Management Plan

The City developed a Biosolids Management Plan, May 2004 (included as Appendix B) (to be approved as part of the 2007 open permit process) in accordance with the WWTP permit and OAR 340-050, *Land Application of Domestic Wastewater Treatment Facility Biosolids, Biosolids Derived Products, and Domestic Septage* and 40 CFR, §503.

All waste sludge must be managed in accordance with the DEQ-approved Biosolids Management Plan to ensure compliance with the federal biosolids regulations (40 CFR §503) and the state rules (OAR 340-050). The City's Biosolids Management Plan was originally approved June 22, 1989. An updated management plan was approved with the last permit in 2004. The biosolids consistently meet the vector attraction and Class A pathogen reduction requirements and the Class A metal content limit in 40 CFR §503. After treatment necessary to comply with vector attraction and pathogen reduction requirements, the Class A biosolids can be sold, given away, or beneficially land-applied with few additional restrictions. The plan also allows land application of Class B biosolids (should it become necessary or desirable in the future). The site selection criteria for land application include all of Oregon.

At maximum loading, each composting tunnel has a detention time of approximately 14 days. All batches must meet the minimum vector attraction reduction requirement of aerobic treatment of the sludge for at least 14 days at over 40 degrees C, with an average temperature of over 45 degrees C. The City uses Option 5 (composting time and temperature) to demonstrate compliance with vector attraction requirements. The compost pile must be over 40 degrees C for at least 14 days, with the average temperature of over 45 degrees C.

The City uses salmonella monitoring of the compost and Alternative 5 of Processes to Further Reduce Pathogens to demonstrate compliance with Class A pathogen reduction requirements. The compost pile must be at a minimum temperature of 55 degrees C for 3 consecutive days. The compost (Class A biosolids) is sold to interested parties for home and commercial use.

If Class B biosolids are produced, the biosolids are recycled back through the reactor. However, the City no longer has authorized land-application sites. Any future land-application sites must conform to the site selection criteria in the Biosolids Management Plan and must be located in Oregon.

1.3.6 Pretreatment Program

An Industrial Pretreatment Program must be developed and implemented in accordance with federal regulations governing pretreatment programs, as required by EPA's Code of Federal Regulations Title 40 (40 CFR §403) and approved by DEQ. The City's Industrial Pretreatment Program is designed to protect its Publicly Owned Treatment Works (POTWs); the Newberg WWTP, solids quality, the City's collection system, the Willamette River, and workers' health and safety.

The City has a DEQ-approved Industrial Pretreatment Program. Federal and state pretreatment requirements were included in the previous NPDES permit for this facility. However, the City's Industrial Pretreatment Program has been inactive since the last significant industry closed. The requirement to implement an Industrial Pretreatment Program could be deleted from the permit, but the City wishes to retain its program in order to attract new significant or categorical industries.

The City is updating its pretreatment program and developing documentation for implementation and enforcement. The City is submitting the revised Pretreatment Program documentation as a substantial change for DEQ approval.

1.4 STATE AND FEDERAL REGULATIONS

State regulations take precedence over federal regulations, where applicable. In some instances, state regulations may impose more stringent requirements than federal regulations. However, federal regulations apply if no state regulations are declared.

1.4.1 Water Quality Requirements

The standards for river basins in Oregon are established by DEQ through OAR 340-4 1-445. These rules are reviewed on a yearly basis for setting new or modifying existing standards. Oregon's water quality standards for specific reaches of the Willamette River and its tributaries are discussed below.

The City's discharge to the Willamette River is at River Mile 49.7. The discharge is within the Willamette basin and the middle Willamette sub-basin. The designated beneficial uses of the receiving stream are public and private domestic water supply, industrial water supply, irrigation, livestock watering, fish and aquatic life (including salmon and steelhead migration corridor), wildlife and hunting, fishing, boating, water contact recreation, aesthetic quality, and hydropower. The water quality standards for the Willamette Basin (OAR 340-041) were developed to protect the beneficial uses of the basin.

The Willamette River is included on DEQ's List of Water Quality Limited Water Bodies (also called the 303(d) List) as water quality limited for the parameters listed in Table 1-7.

Table 1-7. Willamette River Water Quality Limitations (from Permit Evaluation Sheet (March 2004))

Record	River Mile	Parameter	Season	Criteria
6038	24.8 to 54.8	Fecal coliform	Winter/spring/fall	Geometric mean of 200, no more than 10 percent >400
9220	24.8 to 54.8	PCB	Year-round	Public health advisories
9221	24.8 to 54.8	Aldrin	Year-round	Public health advisories
9223	24.8 to	Dieldrin	Year-round	Public health advisories

	54.8			
9224	24.8 to 54.8	DDT	Year-round	Public health advisories
9225	24.8 to 54.8	DDT metabolite (DDE)	Year-round	Public health advisories
8381	24.8 to 54.8	Iron	Year-round	Table 20
5864	24.8 to 54.8	Temperature	Summer	Rearing: 17.8 C
7087	24.8 to 54.8	Mercury	Year-round	Public health advisories
6125	24.8 to 54.8	Biological criteria		Waters of the state shall be of sufficient quality

Fecal Coliform TMDL. Fecal bacterial levels exceed the standard during fall, winter, and spring, but no specific TMDL will be developed for NPDES point source dischargers. The NPDES permit represents the Bacteria Control Management Plan for the City. As long as the discharge remains in compliance with the permit's bacteria limits, the treated effluent discharge will not have a negative impact on the water quality of the Willamette River with respect to bacteria. The sewage collection system has undergone minimal raw sewage overflows during the last several years. Those overflows may have contributed slightly to the ambient fecal bacteria violations.

Temperature TMDL. In the preliminary analysis of the Willamette River TMDL, the City of Newberg was not shown to have a significant effect on the Willamette River temperature. The Willamette River TMDL addressing temperature was issued September 2006. The TMDL assigns WLAs to point sources. The WLA for the Newberg WWTP is being incorporated into the permit during the City's currently open permit process.

Mercury Issues. DEQ has used an incremental approach for the mercury TMDL. Beginning in 2007, DEQ will require selected municipal and industrial facilities, including the City, to increase monitoring and reporting of mercury for 2 years, and take steps to reduce known sources of mercury by developing mercury minimization plans. DEQ will also work with communities and businesses to reduce soil erosion that can carry mercury to rivers. Best management practices to reduce soil erosion in agricultural, forested, and urban sectors will also serve to keep mercury out of the basin's waterways. By 2011, DEQ will evaluate the effectiveness of this implementation strategy and update the mercury TMDL, as necessary. A mercury minimization plan may be required as part of the TMDL.

Dissolved Oxygen. For water bodies identified by DEQ as providing cool-water aquatic life, the dissolved oxygen may not be less than 6.5 mg/L, at an absolute minimum. At the discretion of DEQ, when the agency determines that adequate information exists, the dissolved oxygen may not fall below 6.5 mg/L as a 30-day mean minimum, 5.0 mg/L as a 7-day mean minimum, and below 4.0 mg/L as an absolute minimum.

pH. pH values in the Willamette River outside the mixing zone shall not fall outside the range of 6.5 to 8.5.

Turbidity. No more than a 10 percent cumulative increase in natural stream turbidities shall be allowed, as measured relative to a control point immediately upstream of the turbidity-causing activity. However, limited duration activities necessary to address an emergency or to accommodate essential dredging, construction, or other legitimate activities that cause the standard to be exceeded may be authorized by DEQ, provided all practicable turbidity control techniques have been applied.

Biocriteria. Waters of the state must be of sufficient quality to support aquatic species without detrimental changes in the resident biological communities.

Total Dissolved Solids. A concentration of 100 mg/L shall not be exceeded in the Willamette River and its tributaries.

Toxic Substances. Toxic substances may not be introduced in the waters of the state in amounts, concentrations, or combinations that may be harmful, may chemically change to harmful forms in the environment, may accumulate in sediments or bioaccumulate in aquatic life or wildlife to levels that adversely affect public health, safety, or welfare; aquatic life; wildlife; or other designated beneficial uses. Toxic substances should be controlled through Industrial Pretreatment Programs.

Mixing Zone. Federal regulations (40 CFR §131.13) allow for the use of mixing zones, also known as allocated impact zones. When using mixing zones, acute toxicity to drifting organisms must be prevented and the integrity of the water body as a whole may not be impaired. Mixing zones allow the initial mixing of waste and receiving water, but they are not designed to allow for treatment. EPA does not have specific regulations pertaining to mixing zones. Each state must adopt its own mixing zone regulations that are subject to review and approval by EPA. In states that lack approved mixing zone regulations, ambient water quality standards must be met at the end of the pipe.

DEQ has adopted the aquatic life criterion and developed mixing zone regulations. The regulations are primarily narrative and essentially require the permit writer to use the City's best professional judgment in establishing the size of the mixing zone. Based on EPA guidance and DEQ's mixing zone regulations, two mixing zones may be developed for each discharge that reflect acute and chronic effects: 1.) the acute mixing zone, also known as the ZID; and 2.) the chronic mixing zone, usually referred to as the mixing zone. The ZID is designed to prevent lethality to organisms passing through it. The chronic mixing zone is designed to protect the integrity of the entire water body as a whole. The allowable size of the mixing zone should be based upon the relative size of the discharge to the receiving stream, beneficial uses of the receiving stream, location of other discharges to the same water body, location of drinking water intakes, and other considerations. More specific guidance is available from EPA regarding criteria used in appropriately sizing a ZID. Primarily, the ZID must be designed to prevent lethality to drifting organisms.

DEQ's mixing zone regulations state that the mixing zone must be less than the total stream width as necessary to allow passage of fish and other aquatic organisms. Early recommendations regarding the size of the zone of passage originated in a document from the Department of the Interior (1968). That document recommended a zone of passage of 75 percent of the cross-sectional area and/or volume of flow of the receiving stream. Based on this recommendation, DEQ's standard practice is to allow no more than 25 percent of the stream flow for mixing zones.

DEQ has revised its mixing zone guidance and developed a Mixing Zone Internal Management Directive (IMD), effective July 1, 2008, that outlines how to implement mixing zones assessments and permitting.

Temperature Thermal Plume Limitations. Temperature mixing zones and effluent limits authorized under OAR 340-041-0028(12)(b) will be established to prevent or minimize adverse effects to salmonids inside the mixing zone.

Turbidity. Turbidity is measured by nephelometric turbidity units (NTU). The current Oregon water quality standard of an increase of 10 percent turbidity in the receiving stream (see OAR 340-41-205(2) (ç and related standards) is summarized below. No more than a 10 percent cumulative increase in natural stream turbidities shall be allowed, as measured relative to a control point immediately upstream of the turbidity-causing activity. However, limited-duration activities necessary to address an emergency or to accommodate essential dredging or construction, or other legitimate activities that cause the standard to be exceeded may be authorized by DEQ, provided all practicable turbidity control techniques have been applied. This standard is not enforced and will be revised during the next triennial review period.

1.4.2 Wastewater Effluent Reuse Criteria

Reuse is the practice of using treated effluent from a sewage treatment system which, as a result of treatment, is suitable for a direct beneficial purpose. Reuse options include irrigation of agricultural crops, irrigation of parks, irrigation of golf courses, toilet flushing, industrial process water use (i.e., cooling water), landscape irrigation, fire protection, or constructed wetlands.

The WWTP reclaimed water use is within the fence line of the WWTP. Potable water is used to irrigate outside the fence line. Reuse plans are currently being implemented for offsite reuse on a local golf course.

Rules regarding the use of reclaimed water from treatment plants are designed "to protect the environment and public health in Oregon" (OAR 350-055-0005). Through OAR 340-55, DEQ has established treatment and monitoring requirements for potential agricultural and nonagricultural uses of the treated effluent. DEQ has classified reclaimed water into four categories and assigned a minimum degree of treatment required:

Level I: Less than biological treatment or biological treatment without disinfection

Level II: Biological treatment plus disinfection

Level III: Biological treatment plus disinfection with more stringent effluent requirements than Level II

Level IV: Biological treatment, clarification, coagulation, and filtration treatment plus disinfection

The current regulatory environment surrounding reuse water is in transition. Level IV is currently the highest quality reuse water. However, two additional levels of higher quality are being discussed, and standards within each quality classification are likely to change.

Limits for total coliform (organisms per 100 mL) and turbidity (NTU) have been established for the four categories. These standards serve as a general guideline for defining the anticipated water quality required for the various uses. In addition to the water quality limits, DEQ has provided standards for the minimum monitoring required for total coliform and turbidity based on the four categories. Table 1-8 summarizes the treatment and monitoring requirements for the four reuse categories. DEQ may include additional permit effluent limitations and/or permit conditions other than those listed in Table 1-8 if it has reason to believe that the reclaimed water may contain physical or chemical contaminants that would pose potential health hazards to the public or environment.

Table 1-8. Treatment, Water Quality Limits, and Monitoring Requirements for Agricultural Use of Reclaimed Water

Regulations Pertaining to the use of reclaimed water from wastewater treatment facilities				
Category	Level I	Level II	Level III	Level IV
Biological treatment	X	X	X	X
Disinfection		X	X	X
Clarification				X
Coagulation				X
Filtration				X
Total coliform (organisms/100 mL):				
Two consecutive samples	No limit	240	Not applicable	Not applicable
7-day median	No limit	23	2.2	2.2
Maximum	No limit	Not applicable	23	23
Sampling frequency	Not required	Once per week	Three times per week	Daily
Turbidity (NTU);				
24-hour mean	No limit	No limit	No limit	2
5 percent of the time during a 24-hour period	No limit	No limit	No limit	5
Sampling frequency				Hourly

Note: OAR 340-55.

Oregon regulations encourage the use of reclaimed water for beneficial purposes, using methods that protect the health of Oregonians and the environment. The standards for effluent reuse are established by DEQ through OAR 340-055. Reuse activities:

- Are authorized by an NPDES or WWTP permit.
- Must meet all requirements for groundwater protection established in OAR 340-040.
- Require development of a Reclaimed Water Use Plan for offsite reuse.

Application Rates. The goal of reuse on agricultural land is to beneficially reuse treated effluent by applying at rates to meet the crop's gross irrigation and nutrient requirements, which are commonly referred to as agronomic rates.

The nutrient requirement is the amount of fertilizer, such as available nitrogen, phosphorus, and potassium that is needed to obtain an optimum crop yield. The available nitrogen is made up of organic nitrogen, ammonia-nitrogen, and nitrate-nitrite nitrogen. Organic nitrogen is a long-term, slow-release fertilizer. As organic matter decomposes in the soil, microorganisms convert the organic nitrogen to inorganic ammonium nitrogen, a process called mineralization. The assumptions that will be used to determine the available nitrogen are:

- Ninety percent of mineralized organic nitrogen (total Kjeldahl nitrogen-ammonia nitrogen) will be available to the crop in the first year. The balance will be lost to volatilization.
- There will be a 50 percent loss of ammonia from volatilization as a result of application with sprinklers; therefore, 50 percent will remain available.
- One hundred percent of nitrate-nitrogen will be available for crop use.

Table 1-9 summarizes the fertilizer requirements typically used in the Willamette Valley for various crops.

Table 1-9. Example Fertilizer Requirements of Various Crops for Willamette Valley Region, pounds per acre

Nutrient	Pasture grass/turf grass	Field corn	Alfalfa hay	Spring grains	Winter grains	Fall grass seed
Nitrogen	180 to 250	150 to 180	200 to 480	40 to 50	100 to 140	100 to 140
Phosphorus	50 to 75	20 to 30	20 to 30	40 to 60	30 to 60	30 to 60
Potassium	240 to 290	100	160 to 200	40	30 to 100	60

Notes: 1. Nutrient uptake rates for pasture/turf grass, alfalfa, and field corn were taken from EPA's *Process Design Manual for Land Treatment of Municipal Wastewater.*
2. Nutrient uptake rates for spring grains, winter grains, and grass seed were taken from Oregon State University fertilizer guides.
3. Alfalfa hay does not require nitrogen fertilization, but is capable of utilizing the rates indicated.

Seasonal Limitations/Storage Requirements. There are seasonal limitations to an effluent reuse system because during parts of the year (November through February) crops do not require water, and in October they require very little water. Most of the crops grown in the Newberg area will typically have a growing season from April through September.

General Requirements. A number of general requirements have been outlined in OAR 340-55. These requirements address agricultural and nonagricultural uses that are acceptable based on the effluent water quality level, irrigation system, public access requirements, and buffer zones for irrigation. Tables 1-10 and 1-11 summarize the general requirements according to different levels of reclaimed water quality. For the purposes of the tables, the following terms are defined:

Surface: Surface irrigation where application of reclaimed water is by means other than spraying so that contact between the edible portion of any food crop and reclaimed water is prevented.

Spray: Spray irrigation where application of reclaimed water to crops is by spraying it from orifices in piping.

Processed food crops:	Those that undergo thermoprocessing sufficient to kill spores of *Clostridium botulinum*. Washing, pickling, fermenting, milling, or chemical treatments are not sufficient.

Table 1-10. Agricultural Use Allowed with Different Levels of Reclaimed Water Quality

Category level	I	II	III	IV
Required treatment	Biological treatment	Biological treatment plus disinfection	Biological treatment plus disinfection with more stringent effluent requirements than Level II	Biological, clarification, coagulation, and filtration treatment plus disinfection
Public access	Prevented (fences, gates, locks)	Controlled (signs, rural or nonpublic lands)	Controlled (signs, rural or nonpublic lands)	No direct public contact during irrigation cycle
Buffers for irrigation	Surface: 10 feet Spray: site-specific	Surface: 10 feet Spray: 70 feet	10 feet	None required
Irrigation method allowed				
Food crops	Not allowed	Not allowed	Not allowed	Unrestricted
Processed food crops	Not allowed	[1]	[1]	Unrestricted
Orchards and vineyards	Not allowed	[2]	[2]	Unrestricted
Fodder, fiber, and seed crops not for humans	[3]	[1]	[1]	Unrestricted
Pasture for animals	Not allowed	[4]	[4]	Unrestricted
Sod	Not allowed	[1]	[1]	Unrestricted
Ornamental nursery stock	Not allowed	[1]	[1]	Unrestricted
Christmas trees	Not allowed	[1]	[1]	Unrestricted
Firewood	Not allowed	[1]	[1]	Unrestricted
Irrigation method allowed				
Commercial timber	[2]	[1]	[1]	Unrestricted
Firewood: not customer cut	Surface or spray	Surface or spray	Surface or spray	Surface or spray

[1]Advisory Notice Only: the Oregon Health Division (OHD) recommends that there should be no irrigation of this level of effluent for 3 days prior to harvesting.

[2]Surface irrigation where edible portion of crop does not contact the ground, and fruit or nuts shall not be harvested off the ground.

[3]DEQ may permit spraying if it can be demonstrated that public health and the environment will be adequately protected from aerosols. Advisory Notice Only: OHD recommends that there should be no irrigation of this level of effluent for 30 days prior to harvesting.

[4]Surface or spray irrigation: no animals shall be on the pasture during irrigation. Source: OAR 340-55.

Table 1-11. Nonagricultural Use Allowed with Different Levels of Reclaimed Water Quality

Category level	I	II	III	IV
Required treatment	Biological treatment	Biological treatment plus disinfection	Biological treatment plus disinfection with more stringent effluent requirements than Level II	Biological, clarification, coagulation, and filtration treatment plus disinfection
Public access	Prevented (fences, gates, locks)	Controlled (signs, rural or nonpublic lands)	Controlled (signs, rural or nonpublic lands)	No direct public contact during irrigation cycle
Buffers for irrigation	Surface: 10 feet Spray: site-specific	Surface: 10 feet Spray: 70 feet	10 feet	None required
Irrigation method allowed				
Parks, playgrounds, schoolyards, golf courses with contiguous residences	Not allowed	Not allowed	Not allowed	1, 2
Golf courses without contiguous residences	Not allowed	1, 3	1, 3	1, 2
Cemeteries, highway medians, landscapes without frequent public access	Not allowed	1, 3	1, 3	1, 2
Industrial or commercial use	Not allowed	5, 6, 7, 8	5, 6, 7, 8	5, 6, 8
Construction use	Not allowed	5, 6, 7, 8, 9	5, 6, 7, 8, 10	5, 6, 8, 9
Unrestricted impoundments	Not allowed	Not allowed	Not allowed	4, 6
Restricted impoundments	Not allowed	Not allowed	4, 6, 10	4, 6
Landscape impoundments	Not allowed	4, 6, 10	4, 6, 10	4, 6

[1]Signs shall be posted around the perimeter of the facility and other locations indicating that reclaimed water is used for irrigation and is not safe for drinking, and in the case of effluent quality Levels II and III for body contact (e.g., for Level IV, ATTENTION: RECLAIMED WATER USED FOR IRRIGATION. DO NOT DRINK. ATENCION: RECLAMADO DESPERDICIO DE AGUA USADO PARA LA IRRIGACION. NO BEBA EL AGUA; for Levels II and III, ATTENTION: RECLAIMED WATER USED FOR IRRIGATION. AVOID CONTACT. DO NOT DRINK. ATENCION: RECLAMADO DESPERDICIO DE AGUA USADO PARA LA IRRIGACION. EVITE EL CONTACTO. NO BEBA EL AGUA.).

[2]Reclaimed water shall be applied in a manner so that it is not sprayed onto areas where food is prepared or served, or onto drinking fountains.

[3]Reclaimed water shall be applied in a manner so that it is not sprayed within 100 feet from areas where food is prepared or served, or where drinking fountains are located.

[4]Signs shall be posted around the perimeter and other locations indicating that reclaimed water is used and is not safe for drinking, and in the case of effluent quality Levels II and III for body contact (e.g., for Level IV, ATTENTION: RECLAIMED WATER. DO NOT DRINK. ATENCION: RECLAMADO DESPERDICIO DE AGUA. NO BEBA EL AGUA; for Levels II and III, ATTENTION: RECLAIMED WATER USED. AVOID CONTACT. DO NOT DRINK. ATENCION: RECLAMADO DESPERDICIO DE AGUA. EVITE EL CONTACTO. NO BEBA EL AGUA.).

[5]DEQ may impose more stringent limits on the use of reclaimed water if it believes it is necessary to protect public health and the environment.

[6]There shall be no disposal of reclaimed waters into surface or groundwater without authorization by an NPDES or water pollution control facility permit.

[7]Use of reclaimed water in evaporative cooling systems shall be approved only if the user can demonstrate that aerosols will not present a hazard to public health.

[8]Members of the public and employed personnel at the site of the use of reclaimed water shall be notified that the water is reclaimed water. Provisions for how this notification will be provided shall be specified in the reclaimed water use plan.

[9]Unless decontaminated in a manner approved in writing by OHD, tanker trucks or trailers that transport and/or use reclaimed water shall not be used to transport potable water intended for use as domestic water. A tanker truck or trailer used to transport and/or use reclaimed water shall have the words NONPOTABLE WATER written in 6-inch high letters on each side and the rear of the truck. The words NONPOTABLE WATER shall not be removed until decontamination as approved by the OHD has occurred.

[10]Aerators or decorative fixtures which may generate aerosols shall not be used unless approved in writing by the DEQ. Approval will be considered if it can be demonstrated that aerosols will be confined to the area of the impoundment or a restricted area around the impoundment.

Agricultural and Nonagricultural Uses. Agricultural uses include irrigation for food crops, processed food crops, orchards and vineyards, fodder, fiber, seed crops, and pasture for animals. Nonagricultural uses include irrigation at parks, playgrounds, golf courses, cemeteries, highway medians, and other landscape irrigation.

The potential uses for Level II quality effluent range from the irrigation of agricultural crops processed before human consumption or crops not for human consumption, to irrigation at golf courses without contiguous residences. Level IV effluent is the least restrictive with respect to the types of uses for which the treated effluent can be beneficially reused, and it is the most costly to produce.

DEQ provides guidelines on public access and buffer zones for irrigation systems depending on the effluent water quality level beneficially reused. As listed in Tables 1-8 and 1-9, public access requirements for the different effluent levels range from prevented (fences, gates, locks) to no direct public contact during the irrigation cycle. The public access under a Level II effluent quality reuse program must be controlled. This means that irrigation using this effluent can occur only on rural or nonpublic lands that limit the potential for direct public contact. The site used would also require signs indicating the use of reclaimed water in the irrigation system. This level of public access control would be similar for Level III effluent quality. It would be reduced to no restrictions except prevention of direct public contact during the irrigation cycle under a reuse program using Level IV effluent quality.

Buffer zones for surface and spray irrigation systems are intended to protect public health and the environment. As with the public access requirements, the buffer zones are least restrictive for Level IV effluent quality. When irrigating with Level II effluent, the buffer zones for surface (flooding and overland flow) and spray irrigation systems are 10 feet and 70 feet, respectively. DEQ may reduce the buffer distances, as identified in Tables 1-10 and 1-11, if it determines that alternative controls would adequately protect public health and the environment.

To achieve Level IV, additional treatment such as clarification, coagulation, and filtration would be required. More stringent disinfection and turbidity effluent levels would also need to be met.

Other Reuse Design Requirements. Other requirements to consider in designing a wastewater reuse system are alarm devices, standby power, redundancy, cross-connection, and construction and marking of piping, valves, and other portions of the reclaimed water system. As outlined in OAR 340-55, alarm devices are used to provide the necessary warning of loss of power and/or failure of process equipment essential to the proper operation of the WWTP. This requirement is consistent with the design guidelines of any WWTP, whether or not a wastewater effluent reuse system is implemented. In addition to the alarms, appropriate redundancy is required so that a sufficient level of treatment facilities and monitoring equipment is available to effectively prevent use or discharge of inadequately treated water.

There is no cross-connection allowed between a potable water system and the distribution system that carries the reclaimed water unless the connection is through either an unrestricted air gap or a reduced-pressure-principle backflow preventer. This backflow preventer must be tested and serviced professionally at least once per year.

Unless otherwise approved by DEQ, the construction and marking of piping, valves, and other portions of the reclaimed water system must conform with requirements outlined in the *Guidelines for Distribution of Nonpotable Water* of the California-Nevada Section of the American Water Works Association. In general, the requirements that have not already been discussed are:

- *Pipe Separation:* Potable pipelines must maintain a separation of 10 feet horizontally and 1 foot vertically with parallel reclaimed water (nonpotable) pipelines. When potable pipelines cross reclaimed water pipelines, the potable water pipeline must maintain a separation of 1 foot above the reclaimed water pipeline.

- *Pipe and Valve Identification:* Reclaimed water pipeline must be adequately marked with a warning tape. The warning tape should be prepared with specified purple color and printing with the words CAUTION: RECLAIMED WATERLINE. Above-ground or exposed facilities should be marked to differentiate reclaimed water pipelines from potable water systems or wastewater facilities.

Reuse Siting Requirements. Reuse siting requirements are regulated by Senate Bill 212 (2001). Specified requirements for siting reuse facilities outside the UGB include:

- DEQ determination that facility will apply reclaimed water at agronomic rates
- Continued agricultural, horticultural, or silvicultural production
- No reduction in productivity
- Compliance with the Land Use Compatibility (LUC) process
- Limit to future uses of the land
- Restoration of the land to its original condition
- Not applicable to biosolids application

After applying for LUC approval, the local government issues the public notice of its intent to make a decision. The applicant must respond to each alternative identified in public comments in sufficient detail to allow consideration of the alternative. The response must explain how the alternatives were considered and, if not used, why they were not used. As long as explanations are provided, approval of the application cannot be reversed or remanded.

If the application is approved, the uses allowed outside the UGB include:

- Treatment resulting from land application
- Onsite facilities
  - Accessory to and reasonably necessary for land application
  - Buildings (storage, sheds, etc.)
  - Equipment (pumps, hoses, tractors, etc.)
  - Water impoundments (aerated and nonaerated)
- Offsite facilities
  - Building and equipment if located in the public right-of-way or other land with owner's written permission
  - Return of land to its original condition

If the application is approved, the following uses are not allowed by Senate Bill 212:

- Establishment and use of treatment facilities
- Establishment and use of utility facility service lines except in specific circumstances where they are allowed (Oregon Regulatory Statutes 215.213(1)(b) and 215.283 (1)(y))

1.4.3 Biosolids Management Criteria

Biosolids are a primarily organic solid product produced by wastewater treatment processes that can also be beneficially recycled. Various state and federal rules govern use of biosolids.

Regulations. Title 40 of the Code of Federal Regulations, Part 503 (40 CFR §503) discusses standards for the use or disposal of biosolids, also known as sewage sludge. The Part 503 rule establishes requirements for the final use or disposal of biosolids when they are:

- Applied to land to condition the soil or fertilize crops or other vegetation grown in the soil
- Placed on a surface disposal site for final disposal
- Fired in a biosolids incinerator

The Part 503 rule is designed to protect public health and the environment from any reasonably anticipated adverse effects of certain pollutants and contaminants that may be present. As such, the rule includes five subparts: general provisions, requirements for land application, surface disposal, pathogen and vector attraction reduction, and incineration. Part 503 does not replace any existing state regulations; rather, it sets minimum national standards for the use or disposal of biosolids. In some cases, the state requirements may be more restrictive or administered in a manner different from the federal regulation.

Although biosolids can be beneficially used, they can also be placed in a landfill under the provisions of 40 CFR §258. DEQ promotes the beneficial use of biosolids for agricultural production in Oregon. Almost all the biosolids generated in Oregon are used to grow crops on DEQ-approved sites by agronomic rate applications. Biosolids applications are controlled by detailed site authorization letters, which, together with biosolids management plans, are linked directly to the permits for wastewater treatment facilities.

The state biosolids regulations were most recently revised in July 1995. They incorporate by reference many of the federal technical biosolids regulations (40 CFR §503), including limits on trace pollutants and pathogens. Currently, both the state and federal regulations are proposed for amendment, but the state rule-making process has been halted pending issuance of the revised federal regulations. When the state rule-making process restarts, public notice will be made of additional opportunities to comment on the proposed state regulations.

The regulations described in OAR 340-50 and a biosolids program home page are available at www.deq.state.or.us/wq/Biosolids/BiosolidsHome.htm. This website includes links to state and federal regulations, information about biosolids management plans, site authorization letters, information to help with annual reports, and links to other biosolids websites. Biosolids management plans serve as a central administrative tool to guide the production, storage, transportation, and land application of beneficial use of biosolids in Oregon. The City has completed a Biosolids

Management Plan that will be discussed in Chapter 2 of the Facilities Plan. Site authorization letters specify the conditions for beneficial use of biosolids at particular sites. The rest of this report addresses product quality, site identification and approval, and special management considerations.

Biosolids Quality. According to current state and federal regulations (40 CFR §503), biosolids samples should be analyzed for the parameters listed in Table 1-12.

Table 1-12. Sampling Requirements for the EPA 503 Biosolids Regulations

Parameter	Units
Arsenic	Milligrams per kilogram (mg/kg) dry weight
Cadmium	mg/kg dry weight
Chromium	mg/kg dry weight
Copper	mg/kg dry weight
Lead	mg/kg dry weight
Mercury	mg/kg dry weight
Molybdenum	mg/kg dry weight
Nickel	mg/kg dry weight
Selenium	mg/kg dry weight
Zinc	mg/kg dry weight
Total nitrogen	percent dry weight
Nitrate nitrogen	percent dry weight
Ammonia nitrogen	percent dry weight
Phosphorus	percent dry weight
Potassium	percent dry weight
pH	standard units
Total solids	percent
Volatile solids	percent

The nitrogen, phosphorus, and potassium content of the biosolids is important for determining agronomic rates. Nitrogen content can vary significantly depending on source, age, and history. The concentration levels of these nutrients should be determined near the time biosolids are applied because stored biosolids can lose nitrogen rapidly. The assumptions used to determine the available nitrogen in the biosolids are:

- 30 percent of the organic nitrogen will mineralize and be available.
- 50 percent of the ammonia nitrogen will be lost to volatilization with surface application.
- 100 percent of the nitrate-nitrite nitrogen will be available.

Under the Part 503 rule, ceiling concentrations, cumulative pollutant loading rates, exceptional quality or clean biosolids, and annual pollutant loading rates have been established for heavy metals. Table 1-13 lists the acceptable levels for land application based on federal regulations. These limits are the same as those specified in OAR 340-050. The limits are used to determine site life, which is the number of years that biosolids with a uniform metal content can be applied to a specific site.

However, almost without exception, biosolids in Oregon fall below the pollutant concentration limit. In this case, tracking cumulative loadings at a site is not required.

Table 1-13. Federal Regulations (Part 503) for Heavy Metals

Parameter	Ceiling limit, mg/kg	Cumulative loading, kg/hectare (ha)	Pollutant concentration limit, mg/kg	Annual pollutant loading rate, kg/ha/year
Arsenic	75	41	41	2.0
Cadmium	85	39	39	1.9
Copper	4,300	1,500	1,500	75
Lead	840	300	300	15
Mercury	57	17	17	0.85
Molybdenum	75	--[1]	--[1]	--[1]
Nickel	420	420	420	21
Selenium	100	100	100	5.0
Zinc	7,500	2,800	2,800	140

[1]When 40 CFR §503 was amended in 1994, it deleted pollutant limits for molybdenum in biosolids applied to land but retained the molybdenum ceiling limits.

Site Identification and Approval. Before approving any potentially sensitive application site (with respect to residential housing, runoff potential, or groundwater threat), DEQ may require an opportunity for public comment and public hearing. A statement of land use compatibility from the responsible planning jurisdiction should accompany requests for approval of biosolids land application sites. New sites or expansion of existing sites must be proposed to DEQ before use. Newly approved sites become part of the Biosolids Management Plan.

Site criteria for land-applying biosolids include physical features (geological formation, floodplain proximity, and groundwater and surface water proximity, topography, and soils), and method of application. DEQ's specific criteria are outlined in Table 1-14.

Table 1-14. Oregon DEQ Site Criteria for Biosolids Application

Parameter	Criteria
Geology	Must have a stable formation.
Floodplain	Restricted period of application and incorporate biosolids if in a floodplain.
Groundwater	At time of application, the minimum depth to permanent groundwater is 4 feet; the minimum depth to temporary groundwater is 1 foot.
Topography ▪ Slope less than or equal to 12 percent ▪ Well-vegetated slopes up to 30 percent	Liquid biosolids application with appropriate management to eliminate surface runoff ▪ May be used for liquid, dewatered, or dried biosolids ▪ May be used for dewatered or dried biosolids or liquid biosolids with appropriate management
Soils	▪ Minimum rooting depth of 24 inches ▪ No rapid leaching (coarse material) ▪ Avoid saline or alkaline soil
Method of application and proximity to water bodies	Buffer strips may be required to prevent nuisance odors and to protect water bodies. Size depends on method of application and proximity to sensitive area (variable with local conditions and left to discretion of DEQ), as described below: ▪ Direct injection: no limit required ▪ Truck spreading: less than 200-foot buffer strip ▪ Spray irrigation: 50- to 500-foot buffer strip ▪ Cake or dried solids: 0- to 50-foot buffer strip ▪ Near highways: discretion of DEQ ▪ Near ditch, pond, channel, or waterway: greater than 50-foot buffer strip ▪ Near domestic water source or well: greater than 200-foot buffer strip

Source: OAR 340-50, as amended.

Special Management Considerations. Land receiving biosolids for agricultural use requires special management considerations. These relate to site access, types of crops grown, plant nutrient rates, timing and duration of biosolids land application (seasonal constraints), and grazing restrictions. Biosolids are defined as Class A or Class B based on pathogen reduction and processing method. Exceptional quality biosolids are below federal requirements for trace pollutant concentrations that have been treated by a Class A pathogen reduction process and vector attraction reduction procedure. Class A biosolids are recognized as soil amendments which are acceptable for distribution and marketing to the public. Class B biosolids are most common, and they are appropriate for agricultural use as long as limits on cropping are followed.

Access. Controlled access to bulk Class B domestic biosolids and domestic septage land-application sites is required for a minimum of 12 months following surface application of solids. Controlled access means that public entry or traffic is unlikely. Rural private land is assumed to have controlled access, while parks or other public lands may require fencing to ensure control.

Crops. As a general rule, crops grown for human consumption should not be planted for at least 14 months after application of bulk Class B biosolids or domestic septage. This requirement may be waived if the edible parts will not be in contact with the biosolids-amended soil, or if the crop is to be treated or processed before marketing in such a way that pathogen contamination is not a concern. Most often, Class B biosolids are applied to forage or hay crops, so this is not an issue. The only requirement is that biosolids should not be applied within 30 days of harvest or grazing.

No restrictions on planting times are required where Class A biosolids derived products are land-applied to sites used for the cultivation of fresh market vegetables.

Nutrient Loading. Biosolids application to agricultural land should not exceed the annual nitrogen loading required for maximum crop yield and is, therefore, managed according to its fertilizer value. Biosolids may be applied to approved sites above agronomic rates on a one-time basis or less than once per year as long as runoff, nuisance conditions, or groundwater contamination does not occur. Nitrogen accumulation from higher than agronomic rates and annual nitrogen use will determine the acceptable loading rate and frequency.

Seasonal Constraints. In areas where soil damage may occur from application equipment traffic in the wet season, biosolids application should be restricted to the dry season. The main consideration in land application on sloping ground is to avoid surface runoff and soil erosion.

Grazing Restrictions. Grazing animals should not be allowed on pasture or forage and livestock feed should not be harvested for 30 days after application of bulk Class B biosolids or domestic septage.

1.4.4 Groundwater Protection Requirements

Groundwater is a critical natural resource providing water supply for domestic, industrial, agricultural, and other beneficial uses. Groundwater also provides base flow for rivers, lakes, streams, and wetlands. Because groundwater is difficult to clean up once it is polluted, DEQ has implemented an antidegradation policy so that the highest quality groundwater is maintained.

1.4.5 Pretreatment

Updated pretreatment regulations were issued by EPA in October 2005 that will streamline the pretreatment program requirements. EPA's Pretreatment Streamlining Rule revises certain program requirements to be consistent with NPDES requirements for direct dischargers to surface waters. The action will reduce the regulatory burden on both industrial users, state and control authorities without adversely affecting environmental protection; and it will allow control authorities to better focus oversight resources on industrial users with the greatest potential for affecting the POTW or the environment. The streamlining rule addresses 12 specific issues and a few miscellaneous changes pertaining to the general pretreatment regulations.

1.4.6 Reliability and Redundancy

EPA policy requires new or expanding treatment works that discharge to a receiving stream to meet minimum standards for mechanical, electrical, fluid systems, and component reliability. Redundancy and reliability refer to the level of protection required for the receiving stream classification.

Receiving water criteria are set to ensure minimum reasonable protection of the receiving waters as described in the NPDES permit. Reliability is therefore defined primarily as a function of the receiving water requirements, while the required redundancy is a function of the reliability required. The EPA, as part of its July 1984 Technical Bulletin, identifies three classes of reliability. The required level and grant-fundable elements of redundancy are defined in terms of these three classifications.

- Class I: Works that discharge into navigable waters that could be permanently or unacceptably damaged by effluent that was degraded in quality for only a few hours. Examples of Class I works are those discharging near drinking water reservoirs or into shellfish waters.
- Class II: Works that discharge into navigable waters that would not be permanently or unacceptably damaged by short-term effluent quality degradations, but could be damaged by continued effluent quality degradation (i.e., several days). Examples of Class II works are those that discharge into recreational waters.
- Class III: Works not otherwise classified as Class I or Class II.

DEQ has identified the Newberg WWTP as a Class I Treatment System, in their review comments for this Facilities Plan. The Class I redundancy requires that preliminary and primary treatment and disinfection have redundant components so that if one basin is out of service for several days, the treatment plant can still meet monthly treatment requirements.

Table 1-15 lists the minimum backup requirements for plant components that may be provided at the Newberg WWTP in accordance with the EPA's Works Design Criteria, Reliability Class I, for sewage treatment plants. In addition to these standards, unit operations must be designed to pass the peak hydraulic flow with one unit out of service. Also, mechanical components in the facility must be designed to enable repair or replacement without violating the effluent limitations or causing control diversion.

Table 1-15. Reliability Class I Requirements

Plant component	Requirement
Raw sewage pumps	Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period of the treatment works.
Mechanical bar screens	One backup with either manual or mechanical cleaning (manual cleaning if only two screens).
Grit removal	Minimum of two units.
Primary sedimentation	50 percent of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Activated sludge process	A minimum of two equal volume basins; no backup basin required.
Aeration blowers	Supply the design air capacity with the largest unit out of service; provide a minimum of two units.
Air diffusers	Isolation of largest section of diffusers (within a basin) without measurably impairing oxygen transfer.
Secondary sedimentation	75 percent of design flow capacity with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Disinfectant contact basin	50 percent of the design flow with largest unit out of service. Design flow is defined as the flow used as the design basis of the component.
Effluent pumps	Peak flow with largest unit out of service. Peak flow is defined as the maximum wastewater flow expected during the design period of the treatment works.
Electrical power	Provision of two separate and independent sources of electrical power, either from two separate utility substations or from a single substation and a works-based generator. Designated backup source shall have sufficient capacity to operate all vital components, critical lighting, and ventilation during peak flow conditions, except that components used to support the secondary processes need not be included as long as treatment equivalent to sedimentation and disinfection is provided.

The reliability criteria for sludge processes presented in Table 1-16 are also based on the guidance offered in EPA's Works Design Criteria.

Table 1-16. Sludge Handling System Reliability

System components	Required capacity/backup
Sludge holding tanks	The volume of the holding tank shall be based on the expected time necessary to perform maintenance and repair of the component in question.
Anaerobic sludge digestion	At least two digestion tanks shall be provided. At least two of the digestion tanks provided shall be designed to permit processing all types of sludges normally digested.
Aerobic sludge digestion	A backup basin is not required. At least two blowers or mechanical aerators shall be provided. Isolation of largest section of diffusers without measurably impairing oxygen transfer is allowed.
Sludge pumping	Pumps shall be sized to pump peak sludge quantity and maintain velocities above 2 feet per second. Provide a minimum of two pumps.

According to a recent study, *Efficient Redundancy Design Practices* (Water Environment Research Foundation, 2003), most treatment plants have redundant units for operational needs rather than to meet state or federal standards.

1.5 POTENTIAL FUTURE REGULATIONS

Potential future regulations that could influence the direction of the Newberg Facilities Plan are summarized below:

- Nutrients
- Mixing Zones
- Blending
- Capacity, Management, Operations and Maintenance Requirements (CMOM)

The Facilities Plan should preserve maximum flexibility by using an incremental approach for phased expansion to accommodate the need for more or less stringent requirements, triggered by water quality requirements.

1.5.1 Nutrients

This issue pertains to two nutrients nitrogen and phosphorus. The Willamette River is not, nor is it expected to be, water quality limited for nutrients. The need for nutrient removal should be driven by water quality. Nutrient removal will probably not be required initially, but the Facilities Plan describes the approach to be taken if nitrogen and/or phosphorus limits are imposed. The EPA is currently reviewing the need for nutrient removal requirements from WWTPs to protect the nation's waters. This is generally the first step in establishing standards for criteria in the future. Should the EPA promulgate nutrient removal requirements, DEQ would allow Oregon treatment facilities time to comply by incorporating compliance schedules into the next permit renewal following promulgation.

1.5.2 Mixing Zone

There is current public debate about removing the mixing zone and basing compliance on samples taken at the end of the pipe. This is not expected to occur.

The City is required to conduct a mixing zone study per the guidance document Regulatory Mixing Zone Internal Management Directive (July 2008). The mixing zone study was initiated June 2009.

1.5.3 Blending

There has been a national effort at EPA to develop a policy on blending. No funding is allocated for the blending policy development at this time. DEQ has stated that blending will be allowed as it has been in Oregon, into the foreseeable future.

1.5.4 CMOM

EPA is considering proposing NPDES permit regulations to improve the CMOM of municipal sanitary sewer collection systems and to improve prevention and public notification of sanitary sewer overflow events. The proposed rule would reduce health and environmental risks caused by exposure to raw sewage, improve the performance of treatment facilities, and protect the nation's collection system infrastructure by enhancing and maintaining system capacity, reducing equipment and operational failures, and extending the life of sewage treatment equipment. Under the new regulations, all avoidable SSOs will be prohibited. The regulations call for sound asset management practices by which agencies can safely operate, maintain, and manage their wastewater collection systems.

Many in the wastewater collection system business are predicting that CMOM regulations from the EPA will become a reality in the near future. The impact that these new regulations will have on wastewater utilities is broad—affecting all aspects of collection system management. Utilities will have to improve collection system performance by enhancing and maintaining system capacity to eliminate SSOs. Understanding the capacity of the collection system through hydraulic modeling will be a key requirement, together with developing and implementing an O&M program to prevent blockages and backups. Recordkeeping, reporting, and public notification procedures will also require enhancement. Implementation of these activities will improve system performance and extend the life of the collection system infrastructure through good asset management practices. Many utilities have started upgrading their collection system management programs in order to be better prepared when the regulations finally take effect.

1.6. WASTEWATER CHARACTERISTICS

Developing accurate estimates of current plant flows and loads is a critical step in the facilities planning process. Recent historical flows and loads serve as the basis for estimating future flows and loads. These flow and load projections are in turn used in estimating current capacity and the sizing of future wastewater treatment and conveyance facilities. For this evaluation, Newberg WWTP records for January 1998 through December 2004 were analyzed according to DEQ guidelines for developing wet weather and peak flow projections for sewage treatment in western Oregon.

1.6.1 Wastewater Flows

Average and peak flow rates are determined for different aspects of facility design. These flows are defined below.

Definitions of Flow Terms. Flow rates that are important in the design and operation of treatment plants include:

- *Average dry weather flow* (ADWF) is the average flow at the plant during the dry weather season, usually defined as May through October. DEQ uses ADWF to calculate mass discharge limits to receiving streams for BOD and TSS for the dry weather season.

- *Average wet weather flow* (AWWF) is the average flow at the plant during the wet weather season, typically November through April. AWWF is used to calculate mass discharge limits for $CBOD_5$ and TSS for the wet weather season.
- *Average annual flow* (AAF) is the average flow at the plant during the calendar year.
- *Maximum month dry weather flow* (MMDWF) is defined by DEQ as the flow at the WWTP when rainfall quantities are at the 1-in-10 year probability level for the month of May. MMDWF is important in the design of plant's secondary process.
- *Maximum month wet weather flow* (MMWWF) is defined by DEQ as the flow at the WWTP when rainfalls are at the 1-in-5 year probability level for the month of January. Typically, this period is October through April. MMWWF is also used in the design of a plant's secondary process.
- *Peak day flow* (PDF) is the flow rate at the plant that corresponds to a 1-in-5-year, 24-hour storm event that occurs during a period of high groundwater and saturated soils. For the hydrologic/hydraulic modeling analysis, this term is defined as the 24-hour flow that has an expected recurrence interval of once in 5 years. This measure is used for sizing disinfection systems.
- *Peak week flow* is the flow rate into the plant that is measured as the running average of the consecutive 7-day flows. This flow is used to calculate temperature compliance.
- *Peak hour flow* (PHF) is expected to occur during the peak day flow. PHF is the highest flow at the plant sustained for 1 hour and dictates the hydraulic capacity of the WWTP. This flow is also known as the peak instantaneous flow or peak wet weather flow.

Flow Records. In evaluating wastewater flow records, the first step is to identify any limitations in flow measurement, pumping capacity, or collection system capacity. In addition, any unique or unusual conditions that could affect historical flow records should be ascertained. Daily data used are from 2000 through 2004. Monthly data used are from 1998 through 2004. The plant flow meter is calibrated every 6 months. The available flow data and their limitations are discussed below.

Daily Flows. Daily flows for January 2000 through December 2004 are shown in Figure 1-7. Daily flows for the period of record range from 1.3 mgd to 14.9 mgd.

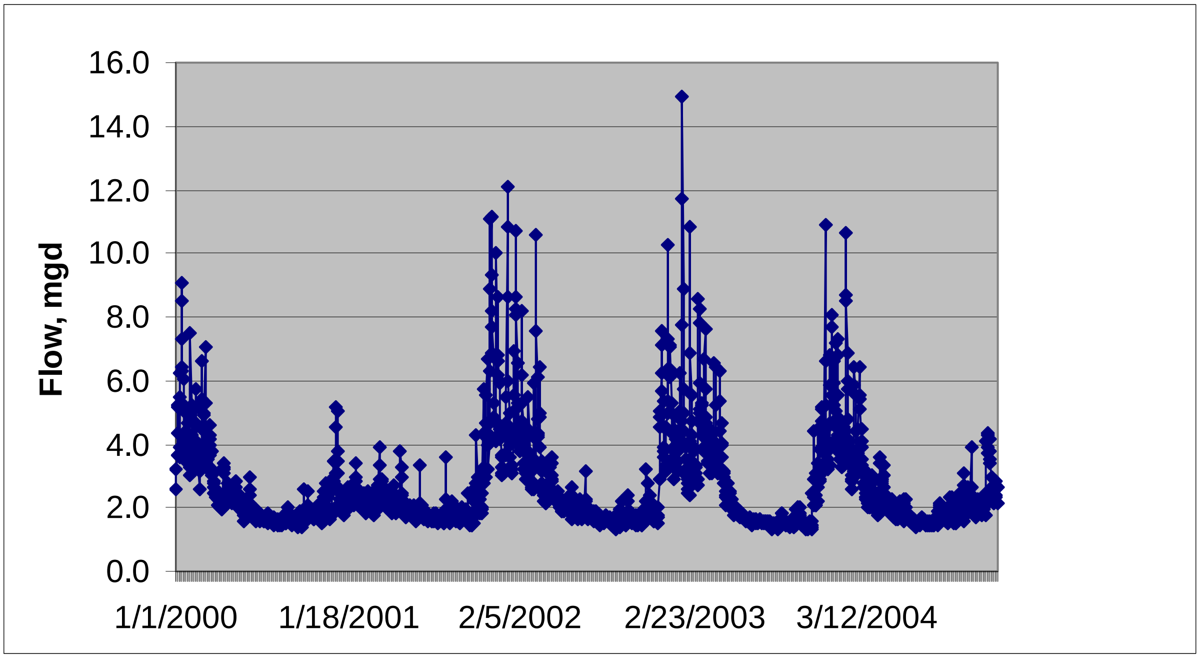


Figure 1-7. Newberg WWTP Historical Daily Influent Flows, 2000 to 2004

Monthly Flows. Monthly flows for the period of record are summarized in Table 1-17. Although outside the period of record, it should be noted that in January 2006, the monthly average was reported to be 8.1 mgd, related to an unusually wet month.

Table 1-17. Monthly Flows for Period of Record, mgd

Month	1996	1997	1998	1999	2000	2001	2002	2003	2004	Average	Maximum
January	5.32	4.86	5.24	5.52	4.77	2.18	5.69	5.14	5.21	4.88	5.69
February	6.15	3.48	4.31	6.18	4.34	2.33	4.10	4.18	4.20	4.36	6.18
March	2.95	4.93	4.07	4.23	3.68	2.33	4.01	4.77	2.76	3.75	4.93
April	3.72	2.78	2.35	2.79	2.38	2.26	2.51	4.06	2.35	2.80	4.06
May	2.96	2.03	2.95	2.17	2.23	2.12	2.03	2.30	1.87	2.29	2.96
June	1.86	2.12	2.09	1.79	1.93	1.92	1.89	1.73	1.73	1.89	2.12
July	1.71	1.58	1.71	1.62	1.66	1.72	1.77	1.58	1.54	1.65	1.77
August	1.53	1.57	1.62	1.63	1.55	1.76	1.57	1.49	1.64	1.60	1.76
September	1.82	1.78	1.66	1.61	1.62	1.78	1.68	1.53	1.76	1.69	1.82
October	2.31	2.44	1.86	1.69	1.80	1.95	1.69	1.62	2.04	1.93	2.44
November	4.11	3.70	4.32	3.49	1.89	3.85	1.93	2.13	2.06	3.05	4.32
December	6.58	3.27	5.61	4.61	2.72	5.58	4.16	4.89	2.82	4.47	6.58

PHF. The records for PHF are not accurate due to the limitations of the influent flow meter and configuration of the pump station and recycle flows. Plant staff estimate that the PHF is limited by the influent pumping capacity. It is important to note that the two influent pumps that have a rated capacity of 4.5 mgd actually discharged only 3.5 mgd when the pumps are worn. Likewise, the two pumps that have a rated capacity of 9 mgd actually discharged only 6.5 mgd when worn. Therefore, the combined peak pumping capacity of the available pumps is estimated to be 20 mgd when the pumps are worn. After pump repairs in September 2006, the pumps were tested and found to pump 25 mgd.

Rainfall Records. Peak wastewater flows can be heavily influenced by rainfall. Therefore, the techniques suggested by DEQ for calculating plant flows require the consideration of statistical recurrences of rainfall quantities. Monthly average rainfall values for the Newberg area are listed in Table 1-18.

Table 1-18. Monthly Average Rainfall Values, inches

Month	1998	1999	2000	2001	2002	2003	2004
January	8.18	8.74	6.87	1.50	8.96	7.60	5.09
February	6.00	10.56	4.70	1.41	3.84	3.24	4.19
March	4.67	4.89	3.02	2.87	4.90	6.19	1.61
April	1.12	1.28	2.27	1.89	3.02	5.58	2.19
May	4.87	2.44	2.44	1.38	1.57	1.15	1.12
June	0.86	0.81	1.47	1.70	1.61	0.19	1.25
July	0.06	0.05	0.07	0.48	0.09	0.00	0.02
August	0.00	0.57	0.08	0.84	0.34	0.21	2.21
September	0.63	0.03	0.34	0.70	1.55	0.99	1.96
October	3.22	2.03	2.42	3.35	0.53	2.03	3.11
November	11.66	8.31	2.49	7.76	3.32	4.90	1.92
December	9.19	5.57	3.58	7.59	9.39	8.13	3.82

Current flow characteristics for the Newberg WWTP are summarized in Table 1-19.

Table 1-19. Summary Current Wastewater Flow Characteristics

Flow, mgd	1998	1999	2000	2001	2002	2003	2004	Average	Maximum
Annual average	3.15	3.11	2.55	2.48	2.75	2.95	2.04	2.72	--
ADWF (May through October)	1.98	1.75	1.80	1.87	1.77	1.71	1.76	1.81	--
AWWF (November through April)	4.32	4.47	3.30	3.09	3.73	4.19	3.23	3.76	--
MMDWF	2.95	2.17	2.23	2.12	2.03	2.30	1.87	2.24	2.95
MMWWF	5.61	6.18	4.77	5.58	5.69	5.14	5.21	5.45	6.18
PWF	--	--	7.12	8.84	7.52	8.33	7.31	--	8.84
PDF	--	--	9.06	11.17	12.08	14.94	10.63	11.58	14.94
Minimum day flow	--	--	1.37	1.46	1.33	1.32	1.39	1.37	--
Peak 4-hour influent flow	--	--	20	20	20	20	20	20	20

Peak Flows. The peak flows of interest are the PDF, PWF, and PHF. The PDF of 16.99 mgd occurred on February 9, 1996. The plant undergoes significant but not extreme levels of infiltration/inflow (I/I). The peak day occurred on February 9, 1996 with a total I/I of 11.43 mgd. The peak week occurred in November to December 2001, with 8.84 mgd average of 7 days. As previously discussed, the PHF rate with all pumps running is 20 mgd (two pumps at 3.5 mgd each and two pumps at 6.5 mgd each).

According to plant staff, on September 30, 2005, influent flows measured 1.6 to 2.0 mgd until about 2 p.m. (14:00). At about 2:45 p.m. (14:45), the plant received a peak of 4.6 mgd and an average of about 4.0 mgd. Rainfall effects on the system were rapid considering the previous long spate of dry weather. This type of flow response is typically indicative of system inflow. Inflow could potentially be removed cost-effectively. The cost-effectiveness of I/I rehabilitation should be evaluated for the Newberg collection system as part of the planned sewer master plan work.

I/I Flows. The I/I flows are estimated by subtracting the winter water usage flows from the flows reaching the treatment plant. The I/I estimation has yet to be completed and will result from the ongoing Collection System Master Plan.

Per Capita Flows. The per capita flows are summarized in Table 1-20. These flows are calculated by dividing the yearly flows by the population. The ADWF value of 98 gallons per capita per day (gcd) is close to the 100 gcd typically used for municipal wastewater. The AWWF is 148 gcd, which is 50 percent higher than the dry weather flow.

Table 1-20. Per Capita Wastewater Flows

Flows, gcd	Value	Peaking factor
AAF	148	1.5
ADWF	98	1.0
AWWF	205	2.1
MMDWF	170	1.7
MMWWF	356	3.6
PWF	484	4.9
PDF[1]	765	7.8
PHF	1,005	10.2

[1] PDF may have been caused by housekeeping procedures.

Peaking Factors. The ADWF is the base flow used to determine peaking factors. The peaking factor for the other flow conditions are derived from dividing each flow condition by the ADWF. For example, the peaking factor for AAF is calculated by dividing 148 mgd by 98 for a peaking factor of 1.5. The peaking factor for PHF is high for typical municipal wastewater, indicating I/I. It is recommended that a collection system master plan be conducted to confirm future PHF.

Industrial Flows. Industrial flows are expected to grow at the same rate as domestic flows and remain at the same ratio of domestic to industrial flows as currently seen.

1.6.2 $CBOD_5$ and TSS Loads

The $CBOD_5$ and TSS loads at a treatment plant affect the following factors:

- *Secondary process sizing.* The design of a secondary process is based on the $CBOD_5$ load.
- *Aeration system design.* The capacity of the aeration system is determined by the peak $CBOD_5$ load.
- *Biosolids production.* $CBOD_5$ and TSS removed by the plant are converted into biosolids that must be stabilized and recycled.
- *Solids treatment and handling system design.* Solids handling facilities, such as digesters and thickeners, must be sized to accommodate expected biosolids quantities.

Current plant $CBOD_5$ and TSS loadings are evaluated below.

Monthly Plant Loading. Average monthly $CBOD_5$ and TSS influent loadings for January 1998 through December 2004 are shown in Figure 1-8 and listed in Table 1-21. Figure 1-8 shows that the $CBOD_5$ and TSS loads have remained fairly steady over the last 6 years. Both the $CBOD_5$ and the TSS loading data show a slow but steady increase over time, which is consistent with a slow, but steady, population growth. However, the TSS data also indicate that the TSS loading seems to have become more varied over the last 2 years. Since 2002, the range of TSS loadings has definitely increased, with many months showing lower TSS loadings than in the past while other months are having significantly increased loadings. Some of the loading variation may be from the discharge of the Newberg WTP solids that contain high iron.

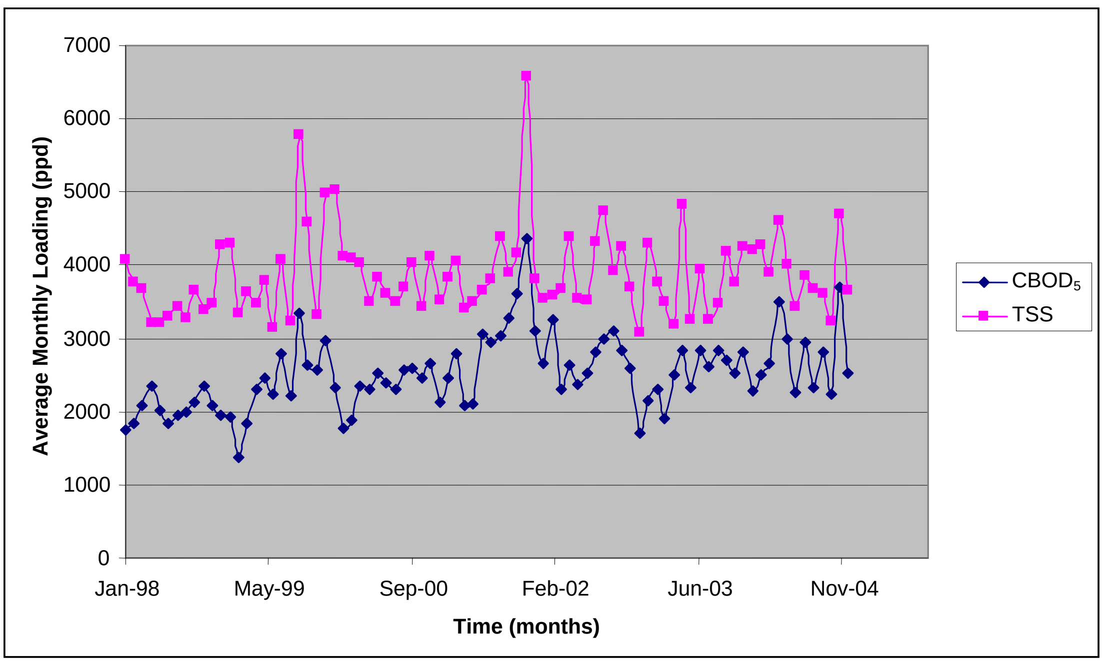


Figure 1-8. Average Monthly Plant Loading, $CBOD_5$ and TSS, 1998 to 2004

Table 1-21. Monthly Plant Loading, $CBOD_5$ and TSS, 1998 to 2004

Parameter	Average monthly influent loading, 1998 to 2004			
	$CBOD_5$		TSS	
	mg/L	ppd	mg/L	ppd
Minimum	28	1,372	68	3,089
Maximum	255	4,355	440	6,577
Average	138	2,511	203	3,872
Winter maximum	214	3,554	274	6,577
Summer maximum	255	3,610	440	4,833

The highest monthly $CBOD_5$ load of 4,355 ppd occurred in November 2001 and is an unusually high value. Similarly, the highest monthly TSS load was 6,577 ppd and also occurred in November 2001. The $CBOD_5$ loading for that month was actually 18 percent higher than any other month on record, while the TSS loading was more than 30 percent higher than any other month on record.

Examining the monthly loading can reveal whether seasonal variations in load occur. While there is definitely an increase in loading and flow during the winter months, the average monthly loading is not significantly higher than in summer.

Per Capita Loadings. For the period of record, 1998-2004, the average monthly $CBOD_5$ unit load was 0.14 pounds per capita per day (pcd), while the maximum monthly average $CBOD_5$ unit load in the same period was 0.24 pcd. These values fall in the range of typical design values, which range from 0.13 to 0.25 pcd. The average and maximum monthly TSS unit loads in the period of record were 0.21 pcd and 0.36 pcd, respectively. The average TSS unit load falls within the range of typical values; however, the maximum month unit value is significantly higher than the typical range of 0.13 to 0.25 pcd. Loads are summarized in Table 1-22.

Table 1-22. Summary of Per Capita Mass Loading Conditions for 1998 to 2004

Flow Related Description of Per capita loading, pcd	$CBOD_5$	TSS
Average month	0.14	0.21
Minimum month	0.10	0.18
Maximum month	0.24	0.36
Winter maximum month	024	0.36
Summer maximum month	0.20	0.33
Maximum peak week	0.64	0.57
Average peak week	0.36	0.44
Peak week–maximum with created data[1]	0.41	0.49
Peak week–average with created data[1]	0.31	0.42

[1] The 2001 peak week TSS 7-day running average resulted in an artificially high value of 10,472 pcd on November 28, 2001 because daily TSS loading data prior to that date were unavailable, and that date had a high TSS load. Data were created to compensate for the lack of data, resulting in what is likely a more realistic value of 9,000 pounds for the maximum 7-day average.

The 2004 peak week $CBOD_5$ 7-day running average resulted in an artificially high value of 12,697 pcd on November 30, 2004 because daily values $CBOD_5$ loading data prior to that date were unavailable, and that date had a high $CBOD_5$ load. Data were created to compensate for the lack of data, resulting in what is likely a more realistic value of 7,659 for the maximum 7-day average.

Industrial Loadings. Industrial flows are expected to grow at the same rate as domestic flows and remain at the same ratio of domestic to industrial flows as currently seen.

Other Wastewater Constituents. The Newberg WWTP has no nitrogen effluent limits in its current NPDES permit.

Ammonia. The average monthly influent ammonia concentration in the Newberg WWTP is only 15.9 mg/L. Figure 1-9 shows the monthly average ammonia influent at the Newberg WWTP for the period of record, 1998 to 2004. The maximum monthly influent concentration was 25.4 mg/L. Ammonia concentration in average domestic wastewater is typically about 25 mg/L.

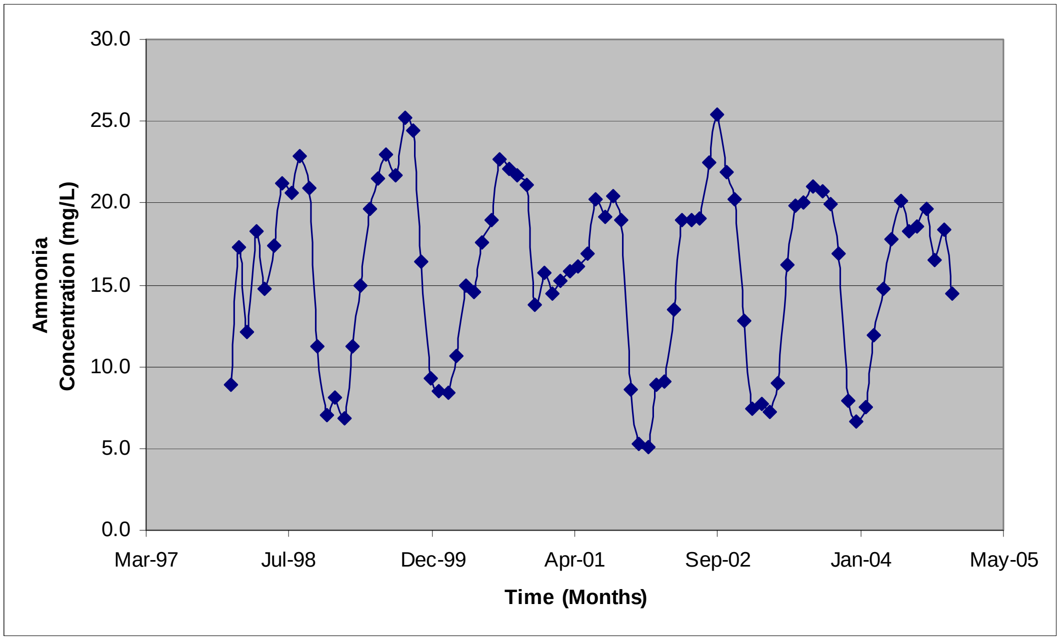


Figure 1-9. Influent Ammonia Concentrations, 1998 to 2004

1.6.3 Flow Condition Calculations using DEQ Method

To calculate maximum month flows, DEQ recommends plotting monthly plant flows and associated rainfall values for January through May of the most recent year. This plot was constructed but was later expanded, as shown in Figure 1-9, to include selected flows and rainfalls from all years of record (1995 to 2004), because little population growth occurred in this period and greater confidence in the results is gained with a longer record. MMWWF is estimated as the flow at the plant corresponding to the 1-in-5 year January rainfall. For the weather station at McMinnville, the 1-in-5 year January rainfall is 11.16 inches. This value was determined by analyzing 75 years of rainfall data for the 1-in-5 year recurrence of monthly January rainfall. Therefore, from Figure 1-10, MMWWF is estimated at 5.4 mgd. The highest monthly average flow reported for the period of record was 6.18 mgd in February 1999, a month with a rain total of 10.56 inches.

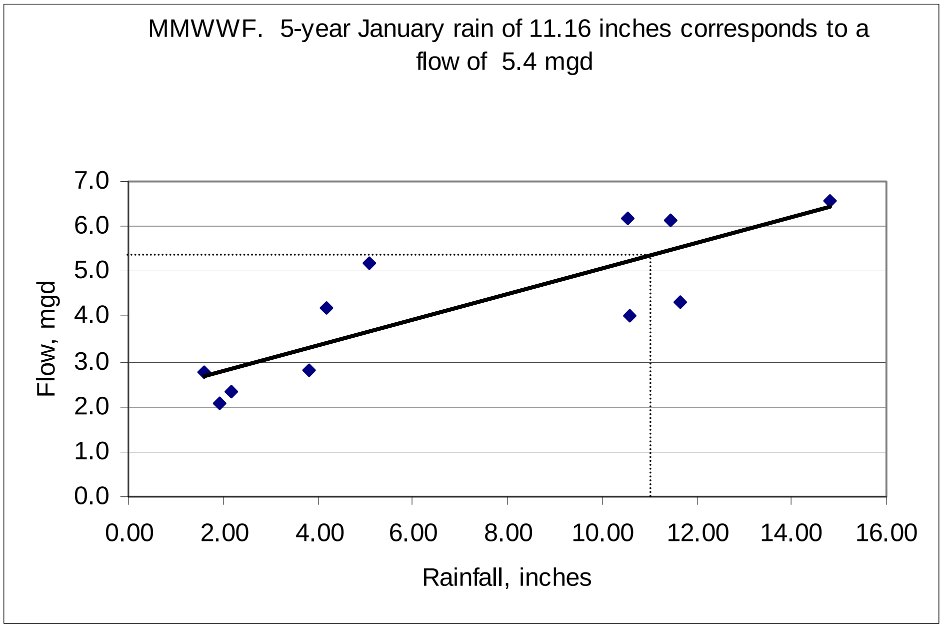


Figure 1-10. Maximum Month Wet Weather Flow, 1998 to 2004

MMDWF is approximated as the flow associated with a 1-in-10 year May rainfall (3.30 inches). This value was determined in a fashion similar to that for the 1-in-5 year MMWWF. As indicted in Figure 1-11, MMDWF is 2.1 mgd. The highest dry weather flow reported since May 1998 was 2.95 mgd in May 1998, a month with 4.87 inches of rain.

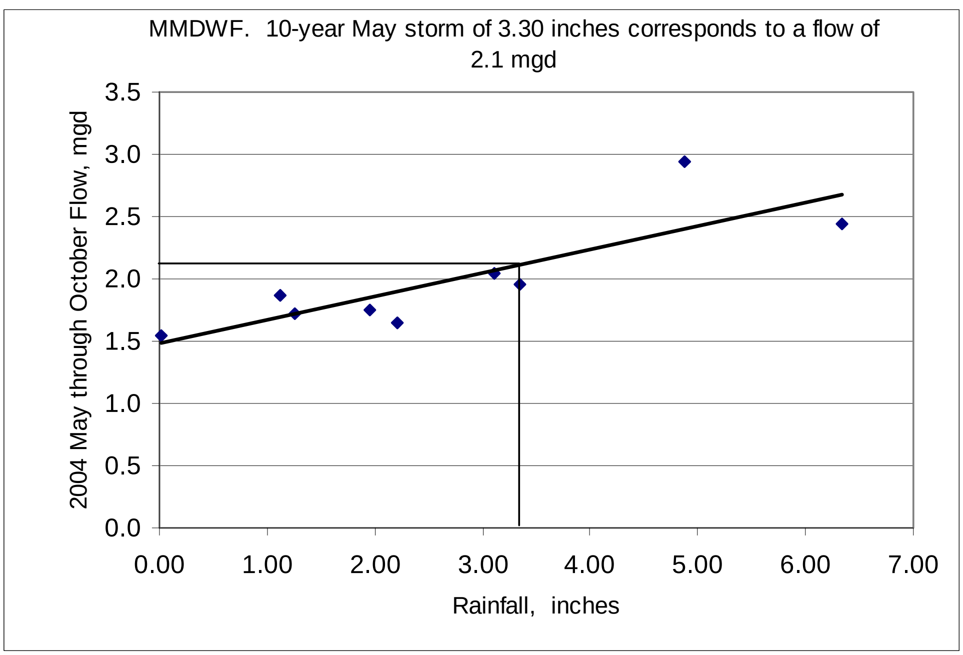


Figure 1-11. Maximum Month Dry Weather Flow, 1998 to 2004

Peak Flows. The peak flows of interest are the PDF, PWF, and PHF. PDF is estimated as the flow associated with the 1-in-5 year, 24-hour storm event. For Newberg, this storm event has been estimated as 3.0 inches of rainfall (Information provided by the National Weather Service Office of Hydrologic Development). However, it is also important to ensure that the appropriate period is selected such that representative antecedent rainfall conditions occurred. That is, the groundwater level should be high and there should be several days of significant rainfall prior to the 1-in-5 year, 24-hour storm event to ensure soil saturation. Only past storm events with probable high groundwater conditions and several days of rainfall preceding the storm were considered to ensure an accurate estimate of high flow events. Table 1-23 lists the storm events considered in estimating peak day flow. Figure 1-12 indicates that a first-order regression line through these points reflects a peak day flow of 11.1 mgd.

Table 1-23. Storm Events, 2003-2004

Event date	Rainfall, inches	Flow, mgd
1/4/2003	0.31	5.316
1/12/2003	0.46	4.919
1/13/2003	0.31	4.422
1/26/2003	0.47	4.726
1/31/2003	2.17	8.871
2/17/2003	1.52	6.850
3/7/2003	1.27	7.793
3/9/2003	0.74	5.907
3/21/2003	0.77	7.602
4/11/2003	0.27	6.531
4/12/2003	0.92	6.401
4/20/2003	0.25	3.670
4/23/2003	0.89	5.354
1/29/2004	0.67	8.476
1/30/2004	0.52	6.877
2/6/2004	0.20	3.738
2/17/2004	0.45	5.540
2/29/2004	0.42	5.081
3/3/2004	0.23	3.907

Note: Dates listed correspond to the date of the rainfall. Flow values listed correspond to the flow recorded on the following day at the WWTP, which more closely represents the peak flow generated by the storm event.

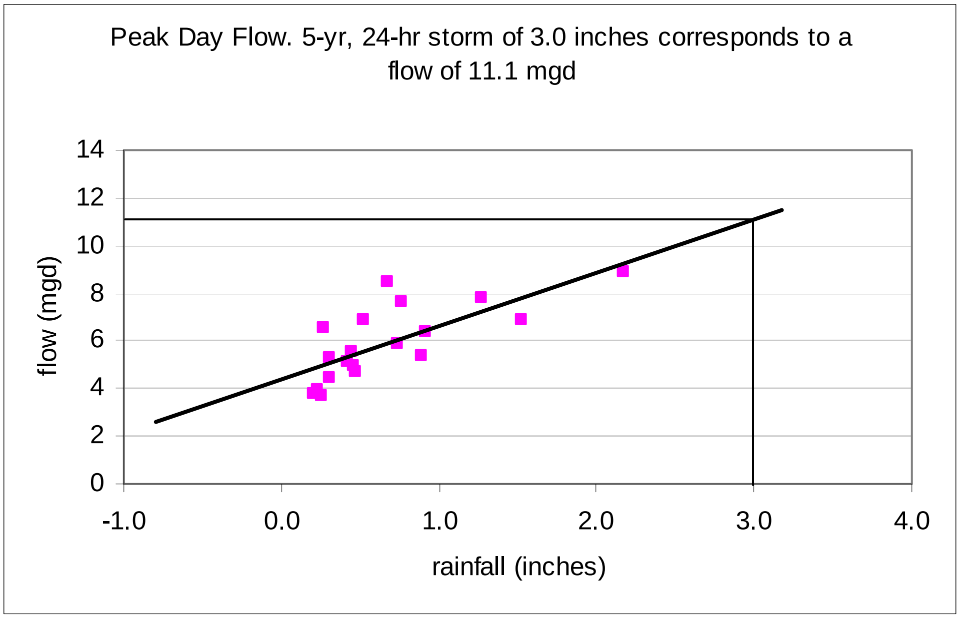


Figure 1-12. PDF, 2003 to 2004

DEQ suggests using probability methods to estimate other peak flows. From the above analysis of rainfall and historical flow data, three flow rates and their associated recurrence probability are known: AAF, MMWWF, and PDF. AAF has a recurrence probability of 50 percent. Assuming that the wet weather flows of interest all occur during a year with 1-in-5 year recurrence probability rainfall, the MMWWF has a recurrence probability of 1 month in 12 months, or 8.33 percent. Similarly, the PDF has a recurrence probability of 1 day in 365 days, or 0.27 percent. As predicted in the DEQ flow calculation guidelines, plotting these three points on log-probability scales approximates a straight line, as shown in Figure 1-13.

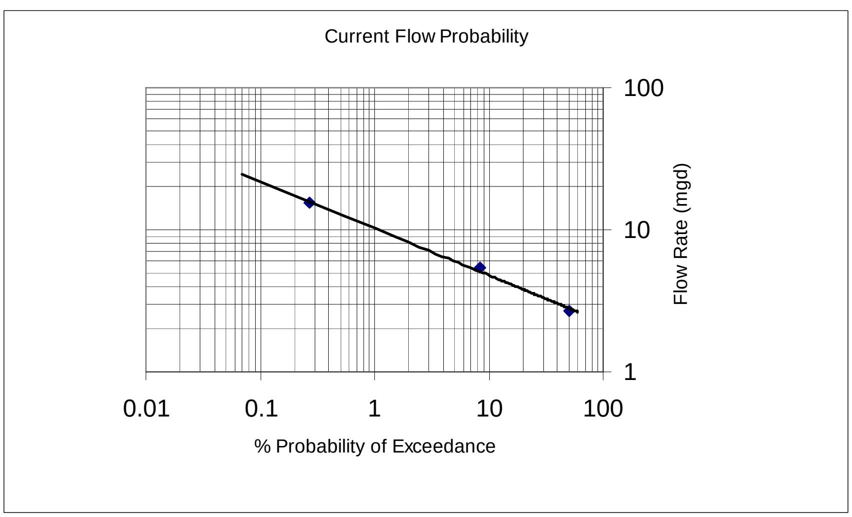


Figure 1-13. Current Flow Probability Analysis

Figure 1-13 can now be used to estimate PHF and PWF (indicated by three small diamonds on the figure). PHF is defined as the peak flow sustained for 1 hour. PHF has a recurrence probability of 1 hour in 8,760 hours (1 year), or 0.011 percent. Based on Figure 1-13, the current estimated PHF is about 26.9 mgd. The current actual capacity of the influent pumps, as indicated by plant staff, is 20 mgd.

PWF is estimated in the same manner as PHF. PWF has a recurrence probability of 1 week in 52 weeks (1.92 percent); this corresponds to a flow of about 6.9 mgd.

Current flows based on the period of record 1999 through 2004 for the Newberg WWTP are summarized in Table 1-24. The flows predicted by the DEQ methodology are less conservative than the flow calculations based on actual flows, except for PHF. The most conservative value for the flows will be used. These flow conditions should be updated after the Collection System Master Plan is completed that will provide a deeper understanding of the peak flows that can reach the plant.

Table 1-24. Current Wastewater Flow Comparison

Description	Estimated flow, mgd (DEQ method)	Flow, mgd[1]
ADWF	--	1.81
AAF	2.1	2.72
AWWF	--	3.76
MMDWF	--	2.95
MMWWF	5.4	6.18
MMWWF[2]	6	6.18
PWF	6.9	8.84
PDF	11.1	14.94
PHF	26.9	20

[1]Using most conservative value.

[2]2-year peak wet weather month.

The City's Sewerage Master Plan Update (SMPU) June 2007, projects current peak flows to be 17.6 mgd based on the 5-year storm event. The SMPU's projected peak flow is in alignment with the peak flow the plant has reported to date.

1.7 FLOW AND LOAD PROJECTIONS

To develop flow and load projections, unit design values are established based on current flows and loads and current population. These values are then used in conjunction with the future design population to develop flow and load projections. The median growth population estimates will be used for projecting flows from 2005 to 2025. The City desired that a range of low to high growth be used to project the 2040 flows to show the impact of the different population growth rates.

1.7.1 Unit Design Values

The current per capita flows and loads are summarized in Table 1-25. These unit design values will be used to projecting future flows and loads. The ADWF flow value of 98 gcd is near the typical range expected for wastewater flow rates (100 gcd) which includes base I/I that is not reasonably removable; therefore 98 gcd will be used in future projections. The AAF per capita value is 148 gcd, which is 50 percent higher than the dry weather flow. The average $CBOD_5$ unit value of 0.14 pcd and the average TSS rate of 0.21 pcd are within the expected design value range. Design values are typically in the range of 0.13 to 0.25 gcd. The actual per capita values based on the period of record will be used to calculate the future loading conditions.

Table 1-25. Per Capita Flows and Loads

Parameter	Per capita value	Peaking factor
Flows, gcd		
AAF	148	1.51
ADWF	98	1.0
AWWF	205	2.09
MMDWF	170	1.7
MMWWF	356	3.63
PWF	484	4.94
PDF	765	7.81
PHF	1,005	10.05
Loads, pcd		
Average $CBOD_5$ load	0.14	-
Maximum month $CBOD_5$ load	0.24	-
Peak week $CBOD_5$ load[1]	0.41	-
Average TSS load	0.21	-
Maximum month TSS load	0.36	-
Peak week, TSS load[1]	0.49	-

[1]Loading values for peak week are maximum values with created data. See footnote to Table 1-23.

1.7.2 Projected Wastewater Flows

Based on a breakdown of land use types, wastewater comes primarily from residential sources, commercial sources, and schools. Commercial sources and schools are expected to grow at approximately the same rate as the overall population. Therefore, the projections for the three sources can be combined into one projection based on population growth. The projected population is multiplied by the unit per capita design to get the ADWF. For example, applying the unit design value of 98 gcd to the design population of 38,352 in 2025 yields a projected ADWF of 3.76 mgd. The projected flows for the other conditions are determined by multiplying their associated peaking factors listed in Table 1-25 by the ADWF for that year. These projected flows are summarized in Table 1-26.

Table 1-26. Monthly Flow Projections from 2005 to 2025

Year	2005	2010	2015	2020	2025	2030[1]	
Population	21,132	24,497	28,712	33,683	38,352	43,600	Peaking factor
ADWF	2.07	2.40	2.81	3.30	3.76	4.27	1
AAF	3.11	3.61	4.23	4.96	5.65	6.42	1.5
AWWF	4.33	5.02	5.88	6.90	7.86	8.93	2.09
MMDWF	3.52	4.08	4.78	5.61	6.39	7.26	1.7
MMWWF	7.52	8.71	10.21	11.98	13.64	15.51	3.63
PWF	10.23	11.86	13.90	16.31	18.57	21.11	4.94
PDF	16.17	18.75	21.98	25.78	29.35	33.37	7.81
PHF[2]	20.81	23.65	27.15	31.19	34.77	38.47	10.05

[1] Flows for 2030 are based on straight line method calculations from projected flows.
[2] PHF peaking factor (varies) decreases with time (0.2 mgd subtracted for every 5 years of population growth).

Typically, as the population increases, the peak hour and day peaking factors decrease. What this implies is that although the ADWF rate will increase as the population increases, the flow rate from the peak I/I events (during large rain events) will stay more constant. The current amount of I/I at the peak day flow is approximately 13 mgd. This is considered high for a town the size of Newberg. As the city develops, typically the new infrastructure will allow proportionally less I/I as new technology and construction methods are able to decrease I/I in new construction. Also, as the City continues to upgrade and replace its old infrastructure, it is expected that these improvements would decrease the amount of I/I caused by the current infrastructure. Therefore the peaking factor of 10.05 (the smaller value resulting from the two methods of calculations in Table 1-26) will be used for PHF projections starting with 2005. For subsequent years, 2010, 2015, 2020, and 2025, 0.25 mgd will be subtracted from the PHF for every 5-year period to account for the decrease in peaking factors as the population increases.

The projected flows for 2040 were determined in a similar manner. The flow rates for 2040 are calculated for a range of expected population projections, as are summarized in Table 1-27. The projected flows are summarized in Figure 1-14.

Table 1-27. Flow Projections for 2040

	Low growth estimate	Median growth estimate	High growth estimate	
Population	44,505	54,097	79,701	Peaking factor
ADWF	4.36	5.30	7.81	1
AAF	6.55	7.97	11.74	1.5
AWWF	9.12	11.08	16.32	2.09
MMDWF	7.41	9.01	13.28	1.7
MMWWF	15.83	19.24	28.35	3.63
PWF		26.19		4.94
PDF		41.40		7.81
PHF[1]		45.86		10.05

[1] PHF peaking factor (varies) decreases with time (0.2 mgd subtracted for every 5 years of population growth).

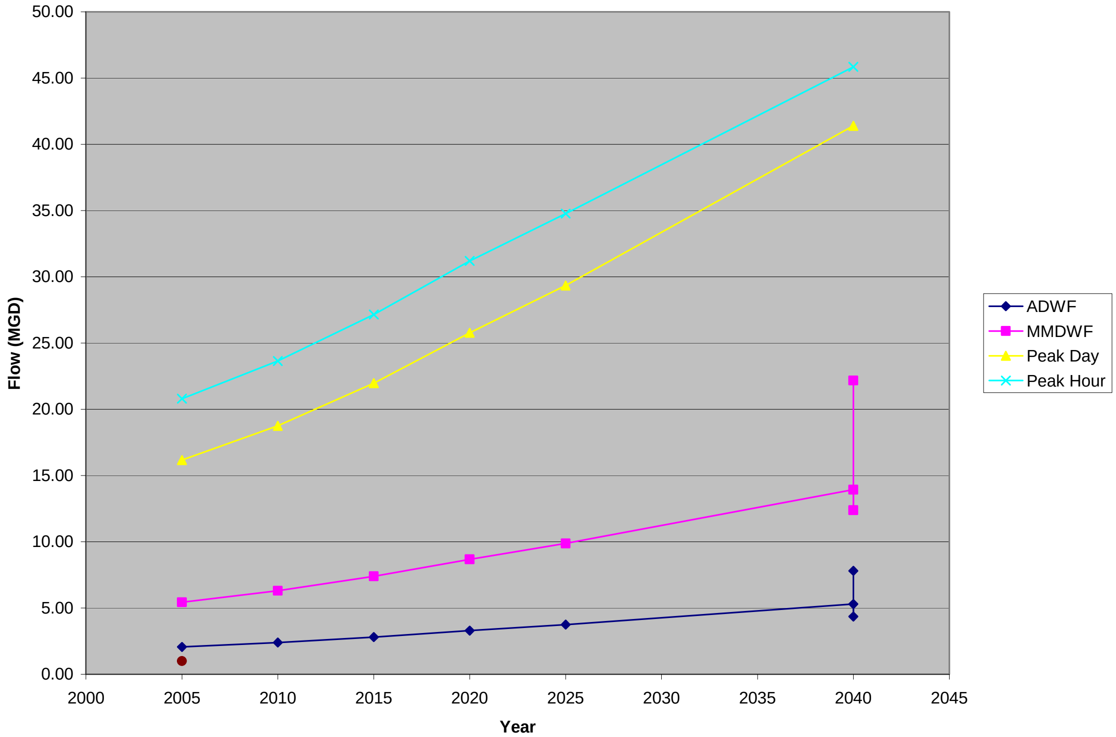


Figure 1-14. Projected Flows

A comparison of the projected flows based on historical data averaging to arrive at peaking factors versus the flow projections based on DEQ methodology is discussed below. To determine flows by DEQ methodology, a probability exceedance curve is developed from the average and peak day flows, as shown in Figure 1-15. This technique is similar to that used previously to develop the current peak flows. Peak month, week, and hour flows are determined based on the fraction of the year that these periods represent. Generally, these points fall in a straight line, assuming no limitations to the collection system. These points are indicated in Figure 1-15. The corresponding flows are listed in Table 1-28.

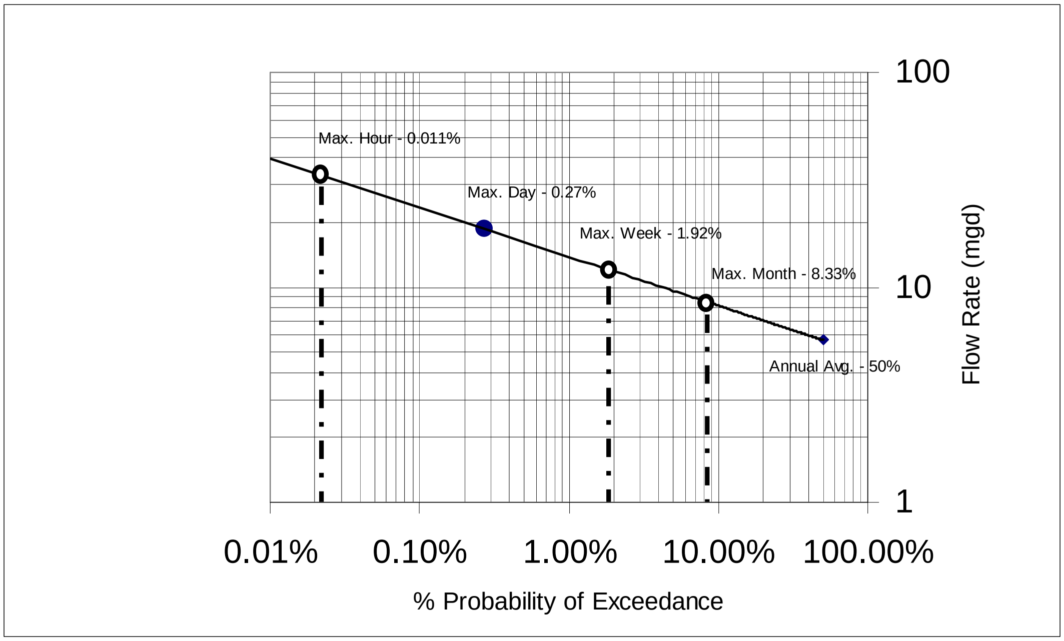


Figure 1-15. Future Peak Flow Projection

Table 1-28. Future Peak (2025) Flows

Flows	Percent probability of exceedance	Flow, mgd (DEQ method)	Flow, mgd (traditional method)
ADWF		--	3.76
AAF	50	5.68	5.65
MMWWF	8.33	8.56	13.64
2-year MMWWF	4	10.12	13.64
PWF	1.92	11.97	18.57
PDF	0.27	18.74	29.35
PHF	0.011	38.95	34.77

The recommended flows for design are derived by the traditional method, which results in comparable or more conservative values for the design flows (except for PHF). For example, Table 1-29 presents the traditional method result of 13.64 mgd for MMWWF, whereas the DEQ method results in 8.56 mgd. Table 1-29 presents flow projections for 2005 to 2025, and for 2040 using the traditional method. Table 1-30 presents the flow projections for 2040 with low, median, and high population estimates using the traditional method. Peak flows associated with the median growth rate will be used for all the growth assumptions in 2040.

Due to inherent limitations in the collection system, a smaller peaking factor is expected for future PHFs as population increases. Peak flow predictions should be refined with the upcoming collection system master planning information that will take into account the collection system capacity to deliver peak flows to the plant.

The recommended flow projections for 2005 through 2040 based on median projected growth rates are summarized in Table 1-29 and Figure 1-16.

Table 1-29. Incremental Flow Projections

Year	2005	2010	2015	2020	2025	2030[1]	2040
Population	21,132	24,497	28,712	33,683	38,352	43,600	54,097
ADWF	2.07	2.40	2.81	3.30	3.76	4.27	5.30
AAF	3.11	3.61	4.23	4.96	5.65	6.42	7.97
AWWF	4.33	5.02	5.88	6.90	7.86	8.93	11.08
MMDWF	3.52	4.08	4.78	5.61	6.39	7.26	9.01
MMWWF	7.52	8.71	10.21	11.98	13.64	15.51	19.24
PWF	10.23	11.86	13.90	16.31	18.57	21.11	26.19
PDF	16.17	18.75	21.98	25.78	29.35	33.37	41.40
PHF	20.81	23.65	27.15	31.19	34.77	38.47	45.86

[1] Flows for 2030 were calculated on a straight line basis from 2025 to 2040.

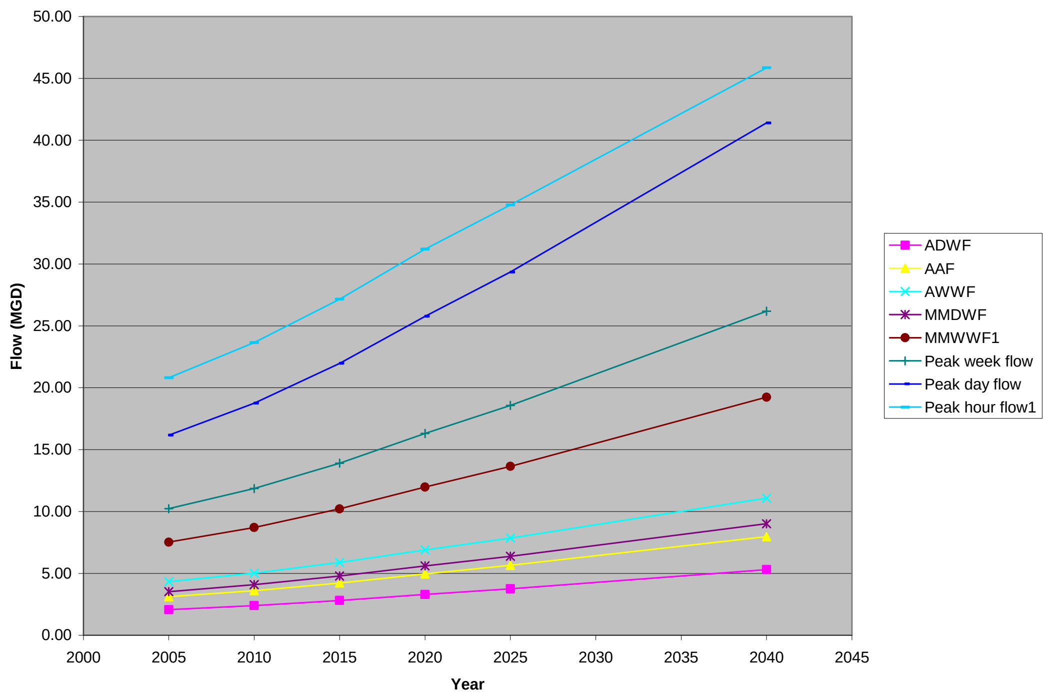


Figure 1-16. Recommended Flow Projections

The range of MMDWF for 2040 varies from 11.5 mgd for low growth to 20.5 mgd for high growth.

Table 1-30. Flow Projections for 2040

	Low growth estimate	Median growth estimate	High growth estimate	Peaking factor
Population	44,505	54,097	79,701	
ADWF	4.36	5.30	7.81	1
AAF	6.55	7.97	11.74	1.5
AWWF	9.12	11.08	16.32	2.09
MMDWF	7.41	9.01	13.28	1.7
MMWWF	15.83	19.24	28.35	3.63
PWF		26.19		4.94
PDF		41.40		7.81
PHF		45.86		10.05

1.7.3 Projected Wastewater Loads

Wastewater load projections are developed by applying the unit design values to the design population. The loads are assumed to increase in proportion to population. The design loads for 2005 to 2025 are presented in Table 1-31.

Table 1-31. Load Projections from 2005 to 2025

	2005	2010	2015	2020	2025	2030[1]
$CBOD_5$, ppd						
Minimum	2,080	2,411	2,826	3,315	3,774	3,927
Maximum	5,034	5,836	6,840	8,025	9,137	9,508
Average	2,862	3,318	3,888	4,562	5,194	5,404
Winter maximum	5,034	5,836	6,840	8,025	9,137	9,508
Summer maximum	4,173	4,838	5,670	6,652	7,574	7,881
TSS, ppd						
Minimum	3,705	4,295	5,034	5,906	6,725	6,998
Maximum	7,603	8,814	10,330	12,119	13,799	14,359
Average	4,423	5,128	6,010	7,050	8,028	8,354
Winter maximum	7,603	8,814	10,330	12,119	13,799	14,359
Summer maximum	6,929	8,032	9,414	11,044	12,575	13,085

[1] Loads for 2030 were calculated on a straight line basis from projected loads

Table 1-32 lists load projections for 2040.

Table 1-32. Load Projections for 2040

ppd	Low growth estimate	Median growth estimate	High growth estimate
Average $CBOD_5$ load	6,027	7,326	10,794
Maximum month $CBOD_5$ load	10,603	12,888	18,988
Peak week $CBOD_5$ load[1]	18,447	22,423	33,036
Average TSS load	9,316	11,323	16,683
Maximum month TSS load	16,013	19,464	28,676
Peak week TSS load[1]	21,912	26,635	39,241

[1] Per capita values for peak week are maximum values with created data (see footnotes in Table 1-23).

The projections for influent ammonia will be based on current influent concentrations. An average monthly influent ammonia concentration of 15.9 mg/L and a maximum monthly influent concentration of 25.4 mg/L will be used. Ammonia concentration in average domestic wastewater is typically about 25 mg/L.

1.8 BIOSOLIDS CHARACTERISTICS

The biosolids characteristics at the Newberg WWTP are discussed below.

1.8.1 Historical Biosolids Information

The City produced approximately 545 metric dry tons of composted biosolids in 2004 as listed in Table 1-33.

Table 1-33. Total Annual Production

Parameter	Million gallons	Dry tons
Amount generated	12.23	545.1
Amount composted	12.23	545.1

The City conducts quarterly chemical tests of the composted sludge, including analyses of nutrients and metals, and pathogens (Tables 1-34 and 1-35). Based on the analyses, there are no known potential impacts from the compost distribution program.

Table 1-34. Biosolids Data

Parameter	mg/dry kg	§503.13 Table 3 limit	Parameter	mg/dry kg	§503.13 Table 3 limit
Arsenic	5.7	41 mg/kg	Molybdenum	3.2	No limit
Cadmium	1.5	39 mg/kg	Nickel	13.7	420 mg/kg
Chromium	15.6	No limit	Selenium	4.4	100 mg/kg
Copper	150	1,500 mg/kg	Silver	8.92	No limit
Lead	21.2	300 mg/kg	Zinc	442	2,800 mg/kg
Mercury	0.77	17 mg/kg			

Table 1-35. Pathogen Analyses

Date	March 2004	June 2004	September 2004	December 2004	Average
Salmonella	<3/4 gram (g)	<3/4g	<3/4g	<3/4g	<3/4g
Fecal coliform	----	----	----	----	----

Note: Results reported as colonies per grams of sample. June and December analyses done on three separate samples (all reported as <3/4g).

Laboratory: Alexin Analytical Laboratories, Tigard, Oregon

1.9 BASIS FOR COST ESTIMATES

To make a valid comparison among alternatives, a present worth analysis is needed in order to incorporate both capital and annual costs in the evaluation. In developing costs for the present worth analysis, many assumptions must be made to compensate for the lack of detail available during the facilities planning process. The analysis techniques and assumptions made are described below.

1.9.1 Present Worth Analysis

In a present worth analysis, annual costs over the economic life of the alternative are brought from the future back to the present and are discounted by an annual percentage rate called the discount rate. Once the annual costs are brought to the present as a single sum, they can be added to the capital cost to derive the total present worth. For this analysis, a discount rate of 4 percent is assumed for the comparative analysis. The analysis period, or economic life, is assumed to be 20 years. Salvage values, or the value at the end of the 20-year study period, are not considered in this analysis.

1.9.2 Precision of Cost Estimates

The precision of a cost estimate is a function of the detail to which alternatives are developed and the techniques used in preparing the actual estimate. The American Association of Cost Engineers divides estimates into three basic categories:

1. *Order-of-Magnitude Estimate.* An order-of-magnitude estimate is made without detailed engineering data. Techniques such as cost-capacity curves, scale-up or scale-down factors, and ratios are used in developing this type of estimate. This type of estimate is normally accurate within +50 percent or -30 percent. That is, the final cost may be as much as 50 percent more or 30 percent less than the estimated amount. A relatively large contingency is normally included to reduce the probability of underestimating.
2. *Budget Estimate.* This estimate is prepared using process flow sheets, layouts, and equipment details. An estimate of this type is usually accurate within +30 percent and -15 percent.
3. *Definitive Estimate.* As the name implies, this estimate is prepared from well-defined engineering data, including construction plans and specifications. At a minimum, the data would include comprehensive plot plans and elevations, piping and instrument diagrams, electrical diagrams, equipment data sheets and quotations, structural drawings, soil data and drawings, and a complete set of specifications. The definitive estimate is expected to be accurate within +15 percent and -5 percent.

The estimates presented in this document are order-of-magnitude estimates because the design has not been developed in sufficient detail for a more precise estimate. Although the final project cost may vary significantly from these estimates, the estimates are useful in evaluating alternatives because they are fairly accurate relative to each other.

1.9.3 Basis for Costs over Time

Future changes in the costs of material, labor, and equipment will cause comparable changes in the costs presented in this analysis. However, because the relative economy of the alternatives are expected to change only slightly with overall economic changes, the decisions based on the economic evaluation should remain valid.

Costs can be expected to undergo long-term changes in keeping with corresponding changes in the national economy.

The cost of steel and concrete has increased significantly in the last year and this trend is continuing due to the natural disasters that occurred in 2005. Construction costs are expected to increase by as much as 10 percent per year in the next 2 years.

1.9.4 Construction Costs

Construction costs include a 35 percent contingency.

1.9.5 Engineering and Administrative Costs and Contingencies

The cost of engineering services for major projects typically covers special investigations, a predesign report, surveying, foundation exploration, preparation of contract drawings and specifications, construction management, start-up services, preparation of O&M manuals, and performance certifications. Depending on the size and type of project, engineering costs may range from 15 to 25 percent of the construction contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects and projects that involve extensive remodeling of existing plants.

The City has its own administrative costs associated with any major construction project. These include internal planning and budgeting, administration of engineering and construction contracts, legal services, and liaison with regulatory and funding agencies. For a typical project of this size, the City's administrative costs are assumed to be approximately 4 percent of the construction contract cost. The total cost for engineering and administration is assumed to be 25 percent. The costs associated with services during construction are assumed to be 12 percent.

1.10 SUPPORTING DOCUMENTATION

The following documents provided resources for this Facilities Plan update:

- *Sewer Master Plan*, KCM, June 1985
- *Wastewater Treatment Plant Construction Drawings*, KCM, 1987
- *Comprehensive Land Use Plan*, City of Newberg, January 2000
- *NPDES Permit and Permit Evaluation Sheet*, Oregon DEQ, June 2004
- *Reuse Water System Predesign Study*, CH2M HILL, October 2005
- *Dump Station and Headworks Study*, Brown and Caldwell, 2002
- *Headworks and Odor Control Design*, Brown and Caldwell, 2003
- WWTP process records and equipment records
- Others, as identified

1.11 SUMMARY OF PLANNING CRITERIA

The recommended planning criteria are summarized below.

1.11.1 Flows

Planning criteria related to flow rates are summarized in Table 1-36.

Table 1-36. Monthly Flow Projections from 2005 to 2040 Based on Median Growth Projections

Year	2005	2010	2015	2020	2025	2030	2040
Population	21,132	24,497	28,712	33,683	38,352	43,600	54,097
ADWF	2.07	2.40	2.81	3.30	3.76	4.27	5.30
AAF	3.11	3.61	4.23	4.96	5.65	6.42	7.97
AWWF	4.33	5.02	5.88	6.90	7.86	8.93	11.08
MMDWF	3.52	4.08	4.78	5.61	6.39	7.26	9.01
MMWWF	7.52	8.71	10.21	11.98	13.64	15.51	19.24
PWF	10.23	11.86	13.90	16.31	18.57	21.11	26.19
PDF	16.17	18.75	21.98	25.78	29.35	33.37	41.40
PHF[1]	20.81	23.65	27.15	31.19	34.77	38.47	45.86

[1]PHF peaking factor (varies) decreases with time (0.2 mgd subtracted for every 5 years of population growth).

The two flow condition scenarios, median and high growth for 2040 are listed in Table 1-37.

Table 1-37. Flow Condition Scenarios for 2040 Based on Median, and High Growth Projections

	Scenario 1: Median growth estimate	Scenario 2: High growth estimate
ADWF	5.30	7.81
AAF	7.97	11.74
AWWF	11.08	16.32
MMDWF	9.01	13.28
MMWWF	19.24	28.35
PWF	26.19	
PDF	41.40	
PHF	45.86	

The City has completed the SMPU and has verified the peak flows that the collection system will convey to the WWTP. The Facilities Plan Update calculation of current peak flows agrees with the SMPU current peak flows.

1.11.2 Loads

The design loads for 2005 to 2040 based on the median growth rate are summarized in Table 1-38.

Table 1-38. Load Projections from 2005 to 2040 Based on Median Population Growth

	2004	2005	2010	2015	2020	2025	2030	2040
$CBOD_5$, ppd								
Minimum	1	2,080	2,411	2,826	3,315	3,774	3,927	
Maximum	1	5,034	5,836	6,840	8,025	9,137	9,508	12,888
Average	2,759	2,862	3,318	3,888	4,562	5,194	5,404	7,326
Winter maximum	1	5,034	5,836	6,840	8,025	9,137	9,508	
Summer maximum	1	4,173	4,838	5,670	6,652	7,574	7,881	
TSS, ppd								
Minimum	1	3,705	4,295	5,034	5,906	6,725	6,998	
Maximum	1	7,603	8,814	10,330	12,119	13,799	14,359	19,464
Average	3,915	4,423	5,128	6,010	7,050	8,028	8,354	11,323
Winter maximum	1	7,603	8,814	10,330	12,119	13,799	14,359	
Summer maximum	1	6,929	8,032	9,414	11,044	12,575	13,085	

[1]Existing data based on a running 7-day average. Daily minimums and maximums do not apply.

Table 1-39 summarizes the load projections for 2040 for the two scenarios.

Table 1-39. Load Projections for 2040 Based on Median and High Growth Projections

ppd	Scenario 1: Median growth estimate	Scenario 2: High growth estimate
Average $CBOD_5$ load	7,326	10,794
Maximum month $CBOD_5$ load	12,888	18,988
Peak week $CBOD_5$ load	22,423	33,036
Average TSS load	11,323	16,683
Maximum month TSS load	19,464	28,676
Peak week TSS load	26,635	39,241

The projections for influent ammonia will be an average monthly ammonia concentration of 15.9 mg/L and a maximum monthly concentration of 25.4 mg/L.

1.11.3 Regulatory Criteria

The important regulatory criteria that could influence the direction of the Facilities Plan are summarized below:

- Mass limits
- I/I removal
- SSO elimination
- 85 percent removal
- Nutrients
- TMDLs
- Mixing zone
- Redundancy and reliability
- Class A biosolids

Mass Limits. Allowable mass limits should reflect design flows at effluent concentrations of 10 mg/L $CBOD_5$ and 10 mg/L TSS (10/10). The Oregon Environmental Quality Commission (EQC) approval is required in order to increase mass loads due to increased flows. The Facilities Plan should preserve maximum flexibility by using an incremental approach for phased expansion to accommodate the need for more or less stringent requirements, triggered by water quality requirements.

The City may request new loads, if necessary, of the EQC. Environmental, economic decision-guiding criteria, and existing water quality management policies need to be addressed prior to requesting EQC approval.

I/I. Cost-effective I/I removal should be performed along with continued flow equalization. The City must demonstrate that I/I will be reduced to the cost-effective point based on EPA requirements. No plant capacity will be provided for excess I/I.

SSO Elimination. The City is planning to remove SSOs by December 31, 2010.

85 Percent Removal. The plant should not be required to achieve 85 percent removal of $CBOD_5$ and TSS at all times, as long as permitted effluent concentration limits are being met and I/I is being removed to the extent that is cost-effective.

Nutrient Removal. The need for nutrient removal should be driven by water quality. Nutrient removal will probably not be required initially, but the Facilities Plan describes the approach to be taken if nitrogen and/or phosphorus limits are imposed. The EPA is currently reviewing the need for nutrient removal requirements from WWTPs to protect the nation's waters. This is generally the first step in establishing standards for criteria in the future. Should the EPA promulgate nutrient removal requirements, DEQ would allow Oregon treatment facilities time to comply by incorporating compliance schedules into the next permit renewal following promulgation.

TMDLs. The Temperature TMDL for the Willamette River was adopted September 21, 2006. The current minimum excess thermal load (ETL) allocation for Newberg calculated for the past two summers is 47 million kilocalories per day. The actual ETL was calculated to be 27 in 2004 and 25 in 2005. Therefore, the effluent flows could double before an impact might be seen for allowable effluent flow and temperature combinations at the 7Q10 river flow. The City's effluent reuse program initially using 1 mgd of effluent will extend the time before thermal loads will need to be addressed. The City plans to implement additional effluent reuse to address thermal loads.

Mixing Zone. The capacity and dilution of the Newberg outfall (Treated Effluent Outfall 001) is evaluated in Section 3. The evaluation considers the ability of the outfall and mixing zone to meet future water quality criteria for trace contaminants, such as mercury and other heavy metals, ammonia and dissolved oxygen. Dechlorination is currently provided so that total residual chlorine is not a water quality issue. Anew mixing zone study was initiated in June 2009 that follows the DEQ Regulatory Mixing Zone Internal Management Directive (July 2008).

Redundancy and Reliability. Class II reliability is appropriate for the Newberg WWTP.

Class A Biosolids. The City is planning to continue to produce Class A Biosolids since a market is developed for the Class A product.

1.11.4 Industrial Contribution

For this Facilities Plan effort, it is assumed that the proportion of industrial flows increases at the same proportion to domestic flows as currently seen.

CHAPTER 2
EVALUATION OF PLANT PERFORMANCE

2.1 PROJECT BACKGROUND

The Newberg Wastewater Treatment Plant (WWTP) is an activated sludge plant that uses the oxidation ditch process. Treated effluent and biosolids are the two products that the plant produces. Treatment plant effluent is discharged through an outfall to the Willamette River. Biosolids are composted to Class A and sold to the public. The plant was constructed in 1987, and the headworks were updated in 2003. The antiquated screen was replaced; screening compaction and redundancy were provided. Biosolids treatment and handling at the Newberg WWTP consists of belt filter press (BFP) dewatering prior to composting to produce a Class A biosolids product.

The WWTP is designed to treat municipal wastewater using the following sequence of unit processes, as shown in Figure 2-1.

- Influent pump station (IPS)
- Headworks screening
- Aerated grit tank
- Secondary treatment with two oxidation ditches and three secondary clarifiers
- Overflow basin
- Effluent disinfection with chlorine
- Effluent conveyance and discharge to the Willamette River
- Solids processing and handling systems: dewatering and composting
- WWTP support systems

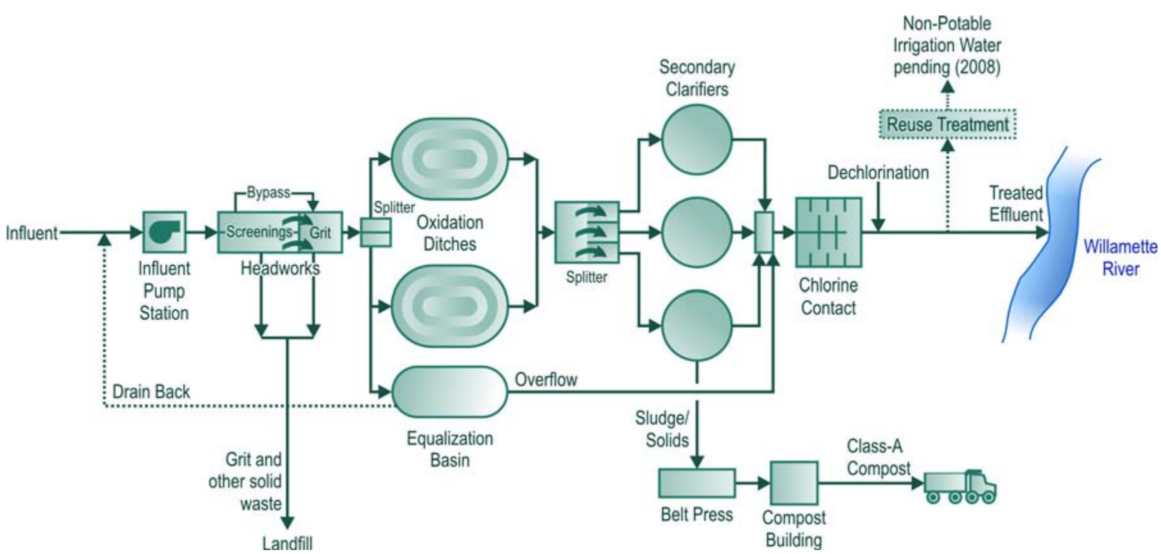


Figure 2-1. Newberg WWTP schematic

2.1.1 Liquid Treatment Performance

The design criteria for the existing Newberg WWTP are listed in Table 2-1. These are summarized from the Newberg Sewerage System Improvements, KCM, 1985. The design average dry weather flow (ADWF) is 4.0 million gallons per day (mgd) and the peak wet weather flow is 18 mgd.

Table 2-1. Existing Newberg WWTP Design Data Summary

Process or design criteria	Unit	Value
Influent flows and loads		
Flow rates		
ADWF	mgd	4.0
Maximum day dry weather flow (MDDWF)	mgd	6.0
Maximum day wet weather flow (MDWWF)	mgd	8.7
Peak (unequalized) wet weather flow (peak hour flow)	mgd	18.0
Biochemical oxygen demand (BOD) average day (AD)	pounds per day (ppd)	7,200
BOD maximum day (MD)	ppd	10,800
Total suspended solids (TSS) AD	ppd	5,000
TSS MD	ppd	7,500
Ammonia concentration	milligrams per liter (mg/L)	30
Effluent requirements		
Dry weather monthly average		
5-day carbonaceous BOD ($CBOD_5$)	mg/L	10
TSS	mg/L	10
Wet weather monthly average		
$CBOD_5$	mg/L	25
TSS	mg/L	30

Table 2-1. Existing Newberg WWTP Design Data Summary (continued)

Process or design criteria	Unit	Value
Influent Pump Station		
Peak Wet Weather Flow Design Value	mgd	18
Pumps		
Submersible- Constant Speed		
Units		2
Capacity	mgd each	4.5
Horsepower	HP	120
Head	feet	92
Submersible- Variable Speed		
Units		2
Capacity	mgd each	9
Horsepower	HP	200
Head	feet	92
Wet Well		
Width and Length	feet	22 BY 17.5
Depth	feet	36
Force Mains		
Number		2
Length	feet	500
Diameter	inches	20
Liquid unit process criteria		
Screen		
Mechanical		
Units	each	2
Type		Traveling plate screen
Capacity	mgd	27
Screen opening	millimeters (mm)	10
Motor	type	2-speed
Screenings washer/compactor		
Units	each	2
Capacity	cubic feet per hour	70
Type		Screw
Motor	horsepower (hp)	2
Estimated screenings production	cubic yards/mgd	.75
Screenings dumpster		
Capacity	cubic yards	10

Table 2-1. Existing Newberg WWTP Design Data Summary (continued)

Process or design criteria	Unit	Value
Grit tank		
Units	each	1
Size	feet	24 by 18
Depth	feet	12
Total volume	cubic feet	5,184
Detention time	minutes	14 at 4 mgd
Grit tank aeration equipment		
Rotary blower, unit	each	1
Capacity	cubic feet per minute	154
Motor, drive	hp	10, constant speed
Depth	feet	12
Grit pumps		
Units	each	2
Capacity	gallons per minute (gpm)	200
Motor, drive	hp	10, constant speed
Cyclone grit separator		
Units	each	2
Capacity	gpm	200
Inlet pressure	pounds per square inch (psi)	10
Underflow rate	gpm	15
Spiral classifier grit washer		
Capacity	tons per hour	0.5
Motor	hp	0.5
Oxidation ditches		
Total volume, each	million gallons (MG)	2
Total volume, each	cubic feet	267,000
Hydraulic retention time (HRT)	hours at mm flow	15 at 6.5 mgd
Solids retention time (SRT)	days (mm)	20 days summer, 25 days winter
Design, mixed liquor suspended solids (MLSS)	mg/L (mm)	2,000
Aeration equipment		
Surface aerators, each basin		
Units	each	4
Type		rotating brush
Capacity	pounds oxygen per hp per hour	2.0
Total connected hp per basin		200 (two-speed)
hp	each	50
Equalization basin		
Total volume	mgd	1.3
Maximum return rate	mgd	2
Equalization basin aeration and mixing pumps	type	jet aeration
Units	each	2
Motor	hp	7.5

Table 2-1. Existing Newberg WWTP Design Data Summary (continued)

Process or design criteria	Unit	Value
Secondary clarifiers		
Units	each	3
Size	diameter, feet	80
Sidewater depth	feet	15
Total surface area, each	square feet	15,020
2 mgd/clarifier	gallons per square foot per day (gsfd)	400
4 mgd/clarifier	gsfd	800
6 mgd/clarifier	gsfd	1,200
8 mgd/clarifier	gsfd	1,600
Return activated sludge (RAS) pumps		
Units	each	4
Capacity, each	gpm	2,800
Motor	hp	40
Maximum RAS rate	mgd	8
Waste activated sludge (WAS) pumps		
Units	each	3
Capacity, each	gpm	300
Drive	type	variable
Motor	hp	7.5
Chlorine contact basin volume		
Units	each	2
Chlorine contact basin volume, total	gallons	269,000
Detention time at 6.5 mgd	minutes	60
Detention time at 4 mgd	minutes	97
Detention time at 12 mgd	minutes	32
Chlorinators		
Units	each	2
Capacity, each	ppd	500
Container size	pounds	2,000
Dechlorination		
Storage tank	gallons	300
Dechlorination pumps		
Units	each	2
Capacity, each	gallons per hour	0.58
Maximum head	psi	250
Outfall		
Diameter	inches	24
Number of ports	each	1

2.1.2 Liquid Stream Performance

Permitted effluent limitations, as discussed in Chapter 1, are for $CBOD_5$ and TSS and vary based on the season. These effluent limitations are summarized in Table 2-2. The permit also limits on the total pounds of $CBOD_5$ and TSS that can be discharged to the Willamette River. See Tables 1-2 and 1-3 in Chapter 1.

Table 2-2. Summary of Effluent $CBOD_5$ and TSS Effluent Concentrations

Parameter	May 1 to October 31		November 1 to April 30	
	Monthly, mg/L	Weekly, mg/L	Monthly, mg/L	Weekly, mg/L
$CBOD_5$	25	40	10	15
TSS	30	45	10	15

There are currently no effluent limits for ammonia, nitrogen, or phosphorus; however the City of Newberg (City) is required by its permit to monitor these constituents as well as metals and priority pollutants on a regular schedule.

2.1.3 Historical Plant Performance

$CBOD_5$ and TSS. Figure 2-2 shows that the average monthly effluent concentration for both TSS and $CBOD_5$ is around 2 to 4 mg/L for the period of record (2000 to 2004), which is well below the allowable effluent concentrations of 10 mg/L in the dry weather summer months. As noted on the graph, the concentrations above 4 mg/L that occurred in January 2004 were still well below the allowable wet weather limit of 30 mg/L TSS.

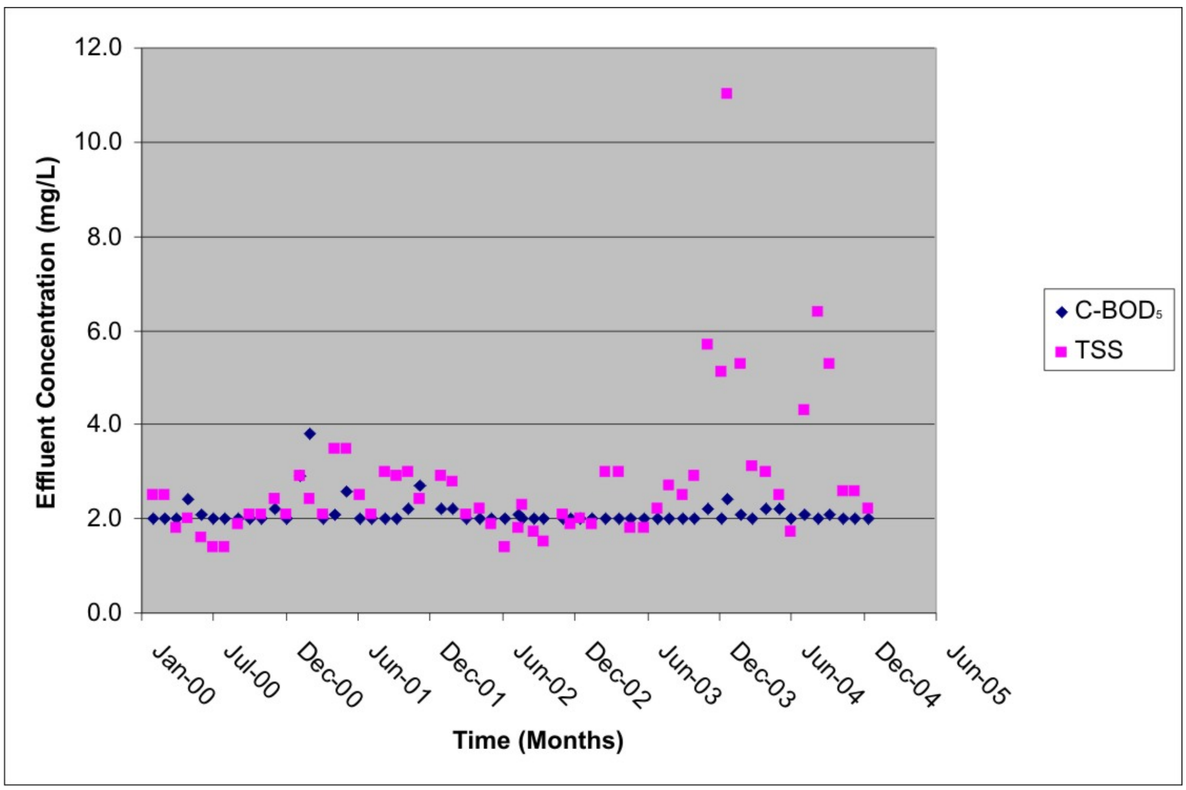


Figure 2-2. Effluent $CBOD_5$ and TSS Concentration

Figure 2-3 shows a similar result for the total ppd in the effluent. The maximum allowable ppd of 3,200 TSS and 2,700 $CBOD_5$ were not exceeded because of the low concentration in the effluent. What should be noted is that although the effluent averages meet the National Pollutant Discharge Elimination System (NPDES) permit requirements, the WWTP is allowed to exclude data from flows that exceed 8.0 mgd from the daily mass load limit calculation. The reason for this is that the Oregon Department of Environmental Quality (DEQ) understands that the Newberg WWTP is heavily influenced by rainwater infiltration.

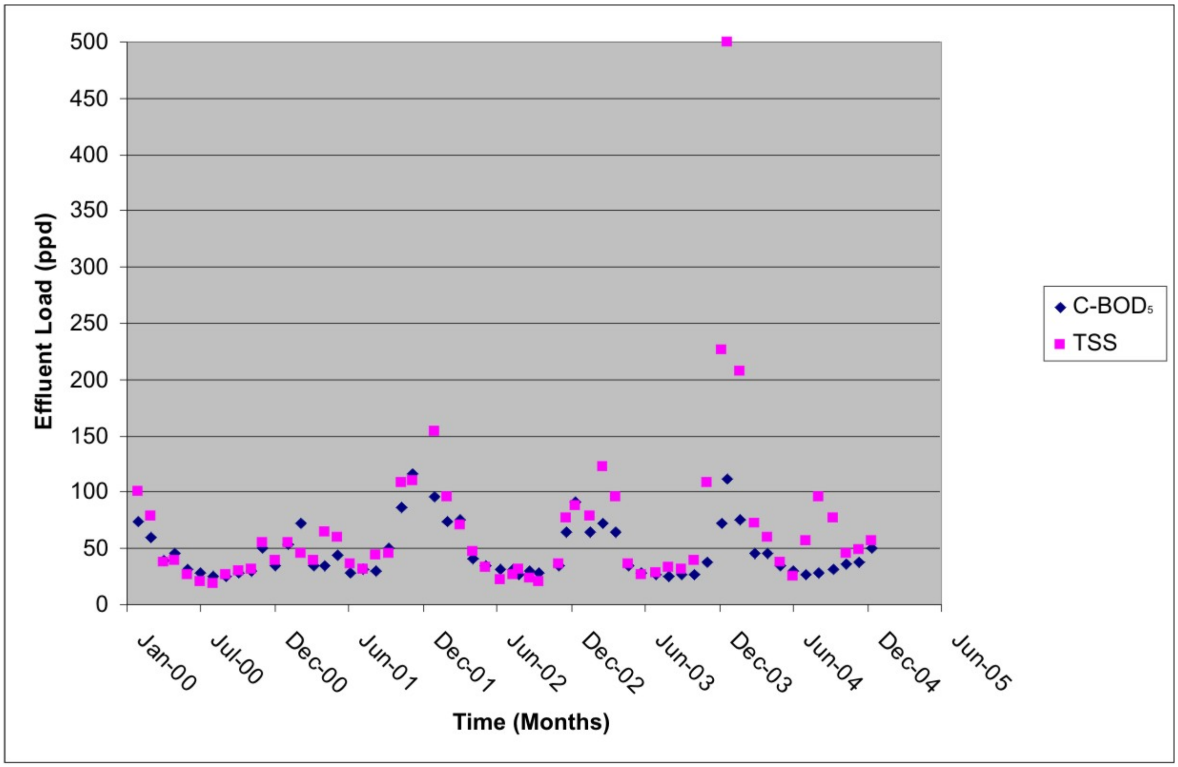


Figure 2-3. Average Daily Effluent $CBOD_5$ and TSS Loading

Ammonia. Ammonia concentration in average domestic wastewater is typically about 25 mg/L. Figure 2-4 shows the average monthly ammonia influent and effluent concentrations at the Newberg WWTP for the period of record 2000 through 2004. The average monthly influent ammonia concentration in the Newberg WWTP is 15.9 mg/L, while the average monthly ammonia effluent concentration is 0.3 mg/L; the last 4 years had only two average monthly values exceeding 0.32 mg/L. This illustrates that the Newberg WWTP is effectively nitrifying and currently reducing ammonia the majority of the time. There is no ammonia limit in the discharge permit, as the WWTP discharge does not have the potential for ammonia toxicity to fish.

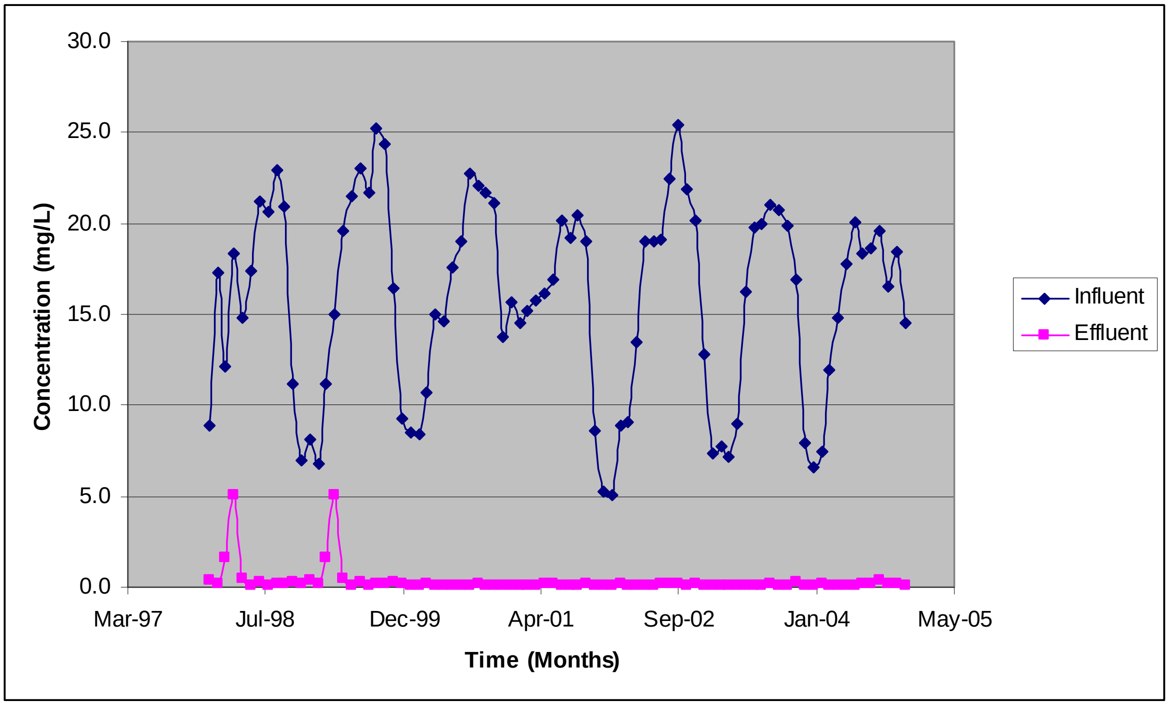


Figure 2-4. Average Monthly Ammonia Concentrations

Total Nitrogen, Nitrite, Nitrate, and Organic Nitrogen. The summer of 2004 was the first time that the effluent total Kjeldahl nitrogen (TKN), nitrite, and nitrate were measured at the Newberg WWTP. TKN is the sum of ammonia nitrogen and organic nitrogen. Organic nitrogen is not as readily oxidized as ammonia nitrogen, so therefore tends to travel through a WWTP unchanged. The average effluent TKN was 1.5 mg/L over the 4-month period.

These parameters were measured for 4 months. The data indicate that the influent ammonia is being oxidized (nitrification) to nitrite and nitrate in the effluent, since the effluent nitrite and nitrate values are approximately equal to the influent ammonia values. This indicates that denitrification is not occurring effectively.

Chlorine Residual. The NPDES permit regulates the total allowable chlorine residual to be discharged into the Willamette River since high chlorine residual is toxic to aquatic life. Figure 2-5 shows the monthly average chlorine residual over the period of record. The NPDES permit allows for a maximum monthly average chlorine residual of 0.02 mg/L. However, the permit adds a note that states that daily maximum concentrations below 0.10 mg/L is considered within compliance. The monthly average data indicate that the City has been operating within its permit limits.

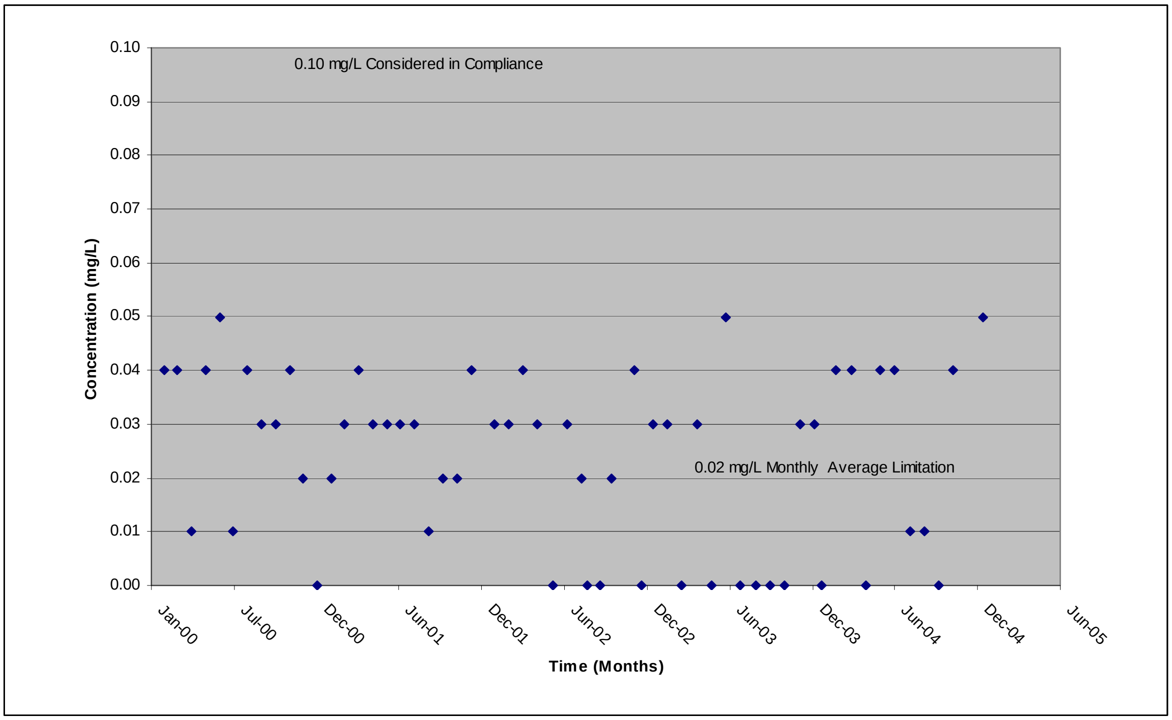


Figure 2-5. Average Monthly Chlorine Residual

***E. coli* Bacteria.** *E. coli* is used as an indicator organism to determine how well disinfection is being performed at the WWTP. The NPDES permit regulates the total allowable number of organisms that can be discharged per 100 milliliters (mL) of treated wastewater. The limit is 126 organisms per 100 mL. If the sample exceeds 126 organisms per 100 mL, then additional sampling is required to show compliance. Figure 2-6 shows the average monthly concentration of *E. coli*. The WWTP did have several individual grab samples that exceeded the maximum limit; however, following samples were within the acceptable ranges so no violation was triggered.

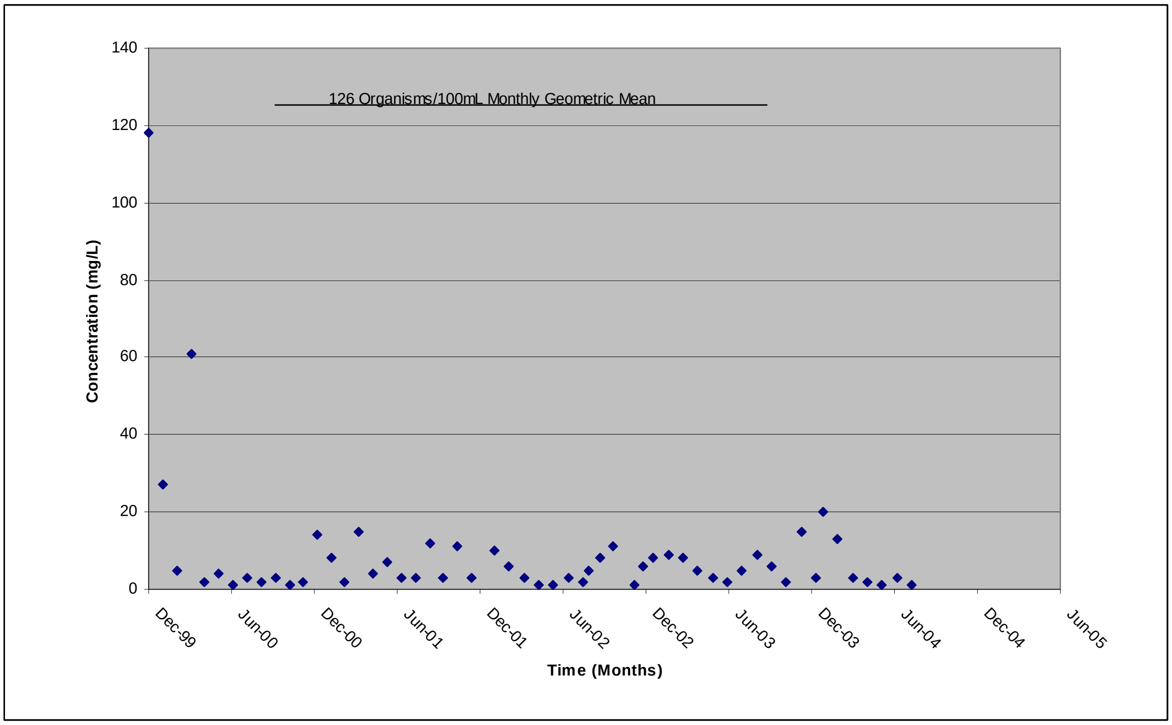


Figure 2-6. Average Monthly *E. coli* Concentration

2.2 LIQUID PROCESS PERFORMANCE EVALUATION

The following section evaluates the existing treatment processes in terms of their capacity limitations. The only concerns about performance at this time are capacity issues. The main capacity concern relates to the high peak flows experienced at the Newberg WWTP. When the WWTP receives peak flows over 18 mgd, the flow is pumped through the screens to the oxidation ditch distribution box and the flow in excess of 18 mgd overflows to the equalization basin. Flows over 18 mgd are stored in the equalization basin until the influent flows decrease. Flows greater than the storage capacity overflow to the disinfection process. The equalization basin is drained back to the IPS. It should be noted that observations in January 2006 indicate that the diversion to the equalization basin begins at approximately 15 mgd, not the design condition of 18 mgd. The peak wet weather flows overflow and blend with secondary effluent for disinfection. It is ineffective at containing all the wet weather storm flows.

2.2.1 IPS

The IPS is an essential component of the WWTP. It pumps the wastewater approximately 100 feet between the lowest point in the collection system up to the headworks that provides screenings and grit removal.

The pump station is currently under capacity. It cannot convey peak flows when one unit is out of service. Typical high influent flow events could cause permit violations, and there are safety concerns with the existing pump station wet well. The wet well is inefficient and causes frequent problems from rags and debris clogging the pump impellers, which decreases the pumping capacity and requires frequent cleaning. In addition, grit from the City's sewerage system and from the WWTP internal plant drains cause severe wear on the pumps, decreasing pumping capacity.

The IPS should be sized to accommodate the peak hour flow (PHF) requirements with the largest unit out of service. It is critical that the IPS be able to handle the capacity hydraulically to keep the influent flow from backing up in the collection system. There is no designed pump station overflow. The existing IPS is sized to handle a peak flow of 27 mgd with all units in service. However, based on City correspondence dated January 12, 2006, the actual maximum pumping rate seen at the IPS is 21 mgd with all units in service. The decreased capacity of the pump station is believed to be caused by the wear and age of the pumps and problems with the design of the pump station that accumulates rags and grit that clog the pumps. With the largest unit out of service the nominal capacity is 18 mgd.

After repair of the influent pumps, the peak flow capacity was seen to increase to 23 mgd with all units in service. This would theoretically equate to 18 mgd with the largest unit out of service. This does not meet the Oregon Standards for Design and Construction of Wastewater Pump Stations (May 2001) that requires the pumping capacity to deliver the rated flow with the largest unit out of service. For current flow conditions, this pump station needs to be able to handle 17.6 mgd (from the Sewerage Master Plan Update, June 2007) with the largest unit out of service. Figure 2-7 shows the current capacity versus future flows and population. The pump station expansion would have to meet the Oregon Standards for Design and Construction of Wastewater Pump Stations.

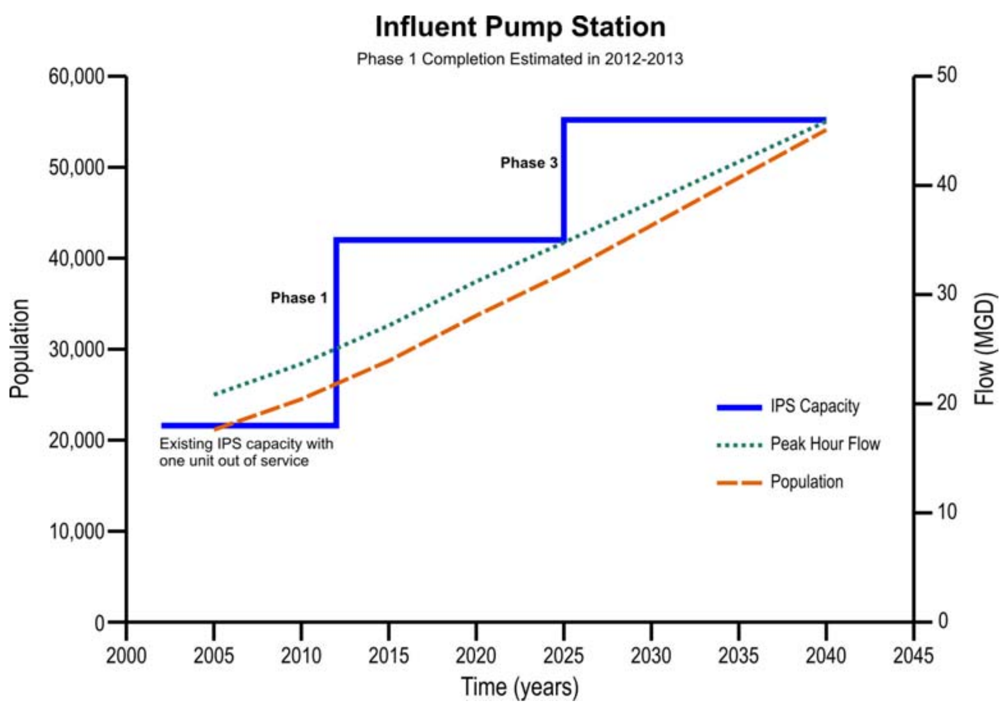


Figure 2-7. IPS Capacity Chart

2.2.2 Headworks

The headworks is sized to accommodate the PHF requirements. It is critical that the headworks be able to handle the hydraulics to keep from overflowing its structure. However, it is not critical that the headworks achieve the same level of treatment for these brief periods of time when the flows are at their peaks.

The headworks consists of two screens, a bypass pipe, and an aerated grit tank. The screens are FSM Traveling Band screens that have a nominal screen opening of 10 mm. The screens were installed in 2003 and each screen is rated for a peak capacity of 21 mgd. If one screen is out of service, a bypass pipe is designed to handle 6 mgd. If power goes out, the emergency generator is not wired nor currently sized to handle the screens. When the power goes out, at least one screen must be rotated out of the channel.

The City currently reports that the screens are performing very well with no maintenance issues. The headworks upgrade also included screenings washer/compactors and a redundant grit washer.

The aerated grit tank is sized to meet a peak influent flow of 18 mgd while maintaining a theoretical hydraulic detention time of 3 minutes. Flows greater than 18 mgd can be manually bypassed to the equalization basin. Figure 2-8 shows the projected plant flows with the existing capacity of the headworks and grit tank versus the projected future flows.

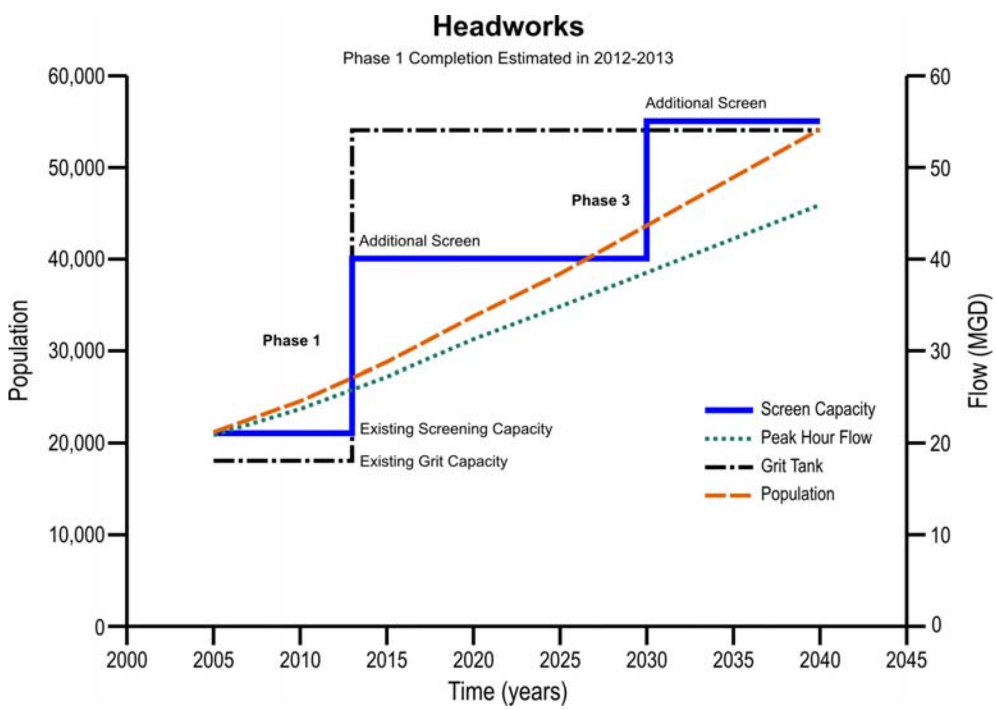


Figure 2-8. Future Projected Flows and Headworks Capacity

Figure 2-8 shows that the capacity of the headworks (screens/overflow pipe and the grit tank plus the overflow) of 27 mgd will be exceeded around 2015, based on PHF.

2.2.3 Secondary Treatment

The Newberg WWTP currently uses two oxidation ditches for secondary biological treatment. Each ditch has a volume of 267,000 cubic feet (2 MG), and is equipped with four 50-hp rotating brush aerators. The two tanks are capable of delivering approximately 800 pounds of oxygen per hour. The current operating SRT for the two oxidation ditches is 20 days in the summer and 25 days in the winter under maximum month wet weather flow (MMWWF) conditions. The HRT in the ditches is 15 hours during MMWWF, and the target MLSS concentration is 2,000 mg/L. Current NPDES permit requirements are 10 mg/L for both BOD and TSS during the summer months. No ammonia or nutrient limitations are specified in the permit.

The plant currently consists of three secondary clarifiers each with a diameter of 80 feet, a side water depth of 15 feet, and a volume 605,000 gallons.

Although future permit requirements for the City are not expected to change, nutrient removal may become a requirement. Significant population growth is expected to occur around Newberg over the next 35 years. Analyses were performed to calculate the secondary treatment expansion needs through 2040 based on median growth. The results of the analyses are listed in Table 2-3.

Table 2-3. Summary of Oxidation Ditch and Clarifier Expansion Needs Based on Median Growth Load Estimates from 2010 through 2040

Removal requirement	Design parameter	2010	2015	2020	2025	2040
Oxidation ditches						
BOD only	Number of ditches	3	3	4	4	6
	Aerators per ditch	4	4	4	4	4
	Percent of volume–aerobic	100	100	100	100	100
	Percent of volume–anoxic	0	0	0	0	0
	MLSS, mg/L	1,500	1,700	1,480	1,690	1,600
	WAS, dry ppd	3,775	4,420	5,190	5,910	8,330
Nitrogen and BOD	Number of ditches	3	4	4	5	6
	Aerators per ditch	3	3	3	3	3
	Percent of volume, aerobic	72	72	72	72	72
	Percent of volume, anoxic	28	28	28	28	28
	MLSS, mg/L	1,500	1,770	1,550	1,770	1,660
	WAS, dry ppd	3,775	4,420	5,190	5,910	8,330
Secondary clarifiers						
Same removal requirement for both scenarios	Peak day flow, mgd	18.8	22.0	25.8	29.4	41.4
	Number	3	4	4	5	6
	Surface overflow rate, gsfd	1,250	1,095	1,285	1,170	1.375

The analyses assume that clarifier performance can be increased with deeper more efficient clarifiers than exist. The recommended peak overflow rate is 1,200 to 1,300 gsfd. Analyses were also performed to examine the oxidation ditch capacity that will be required in 2040 to meet both the current NPDES permit limits and the number and configuration of oxidation ditches that would be needed to meet a supplementary requirement for an effluent total nitrogen limit of 10 mg/L. Flows and loadings for 2040 based on median and high growth estimates for Newberg are listed in Tables 2-4 and 2-5.

Table 2-4. 2040 Flow Scenarios Based on Median and High Growth Projections

Projected flow, mgd	Median growth estimate	High growth estimate
ADWF	5.30	7.81
Average annual flow	7.97	11.74
Average wet weather flow	11.08	16.32
MMDWF	9.01	13.28
MMWWF	19.24	28.35

Table 2-5. Load Projections for 2040 Based on Median and High Growth Forecasts

Projected load, ppd	Median growth estimate	High growth estimate
Average $CBOD_5$	7,326	10,794
Maximum month $CBOD_5$	12,888	18,988
Peak day $CBOD_5$	19,332	28,482
Average TSS	11,323	16,683
Maximum month TSS	19,464	28,676
Peak day TSS	29,196	43,014

For all calculations, it was assumed that the size, configuration, and aeration strategy of the oxidation ditches would remain the same. Results of the oxidation ditch analyses are listed in Table 2-6.

Table 2-6. Summary of Oxidation Ditch Requirements Based on 2040 Median and High Growth Load Estimates

Removal requirement	Design parameter	Median growth estimate	High growth estimate
BOD only	Number of ditches	6	8
	Aerators per ditch	4	4
	Percent of volume, aerobic	100	100
	Percent of volume, anoxic	0	0
	MLSS, mg/L	1,660	1,830
	WAS, dry ppd	8,340	12,260
Nitrogen and BOD	Number of ditches	6	8
	Aerators per ditch	3	3
	Percent of volume, aerobic	72	72
	Percent of volume, anoxic	28	28
	MLSS, mg/L	1,660	1,830
	WAS, dry ppd	8,340	12,260

2.2.3.1 BOD Removal Only

The analysis showed that eight total oxidation ditches of the same configuration would be required to meet the high growth load projections for 2040. This requirement is based solely on the oxygen transfer limitations of the current configuration of four brush aerators. The peak day oxygen required to meet the effluent BOD removal standards is 3,100 pounds per hour (pph). This oxygen requirement includes oxygen for both BOD and nitrification because nitrification will occur at a 20-day SRT. Because each basin can supply only 400 pph with the current brush aerator configuration, eight basins are required. The MLSS concentration in an eight-basin configuration (volume = 16 MG) would be approximately 1,830 mg/L. The corresponding WAS production rate would be approximately 12,260 ppd of dry solids.

For the median growth estimate, six basins would be required due to the lower BOD and nitrogen loads. The corresponding oxygen requirement is 2,110 pph. Both MLSS concentrations and WAS production quantities are also lower (1,660 mg/L and 8,340 dry ppd, respectively).

2.2.3.2 BOD and Nitrogen Removal

Nitrogen removal can be achieved using the same number of basins as for BOD removal alone. Incorporation of anoxic denitrification provides for oxygen and alkalinity recovery. For a target effluent concentration of 10 mg TN/L, an anoxic zone that is 28 percent of the total oxidation ditch volume would be required. The oxygen requirement for BOD oxidation and nitrogen removal is 82 percent of that required for aerobic BOD removal (discussed above). To accommodate the oxygen requirement and anoxic zone volume for denitrification, modifications would be needed to the aeration system, possibly by switching to fine bubble diffusion or using a mechanical aerator with higher oxygen transfer efficiency.

For denitrification to 10 mg/L total nitrogen, 28 percent of the total basin volume would need to be operated in an anoxic mode, and the air requirement would be 82 percent of that required for aerobic BOD removal.

2.2.3.3 Aeration Requirement

It should be noted that, because the required number of oxidation ditches is based on the aeration efficiency of the current brush aerators, eight basins is not a firm requirement. The total number of oxidation ditches could be reduced during expansion by retrofitting the current and newly constructed basins with fine bubble diffusers or with more efficient brush aerators that would increase oxygen transfer capacity. This would facilitate a decrease in the treatment volume, and the overall footprint of the treatment facility. It would, however, increase MLSS concentrations accordingly. There is significant capacity in the secondary clarifiers to accommodate the higher solids loading associated with the higher MLSS concentrations. The clarifier capacity will also need to be increased with increasing flows and loadings.

2.2.3.4 Secondary Clarifiers

The typical design values for secondary clarifier overflow rates are outlined in Tables 2-1 and 2-3. During peak hour flows the overflow rate is 1,600 gsfd at 8 mgd per clarifier, if the flows are not equalized. The design values for the existing secondary clarifier are outlined in Table 2-7. The values are for typical extended aeration plants with oxidation ditches and shallow clarifiers.

Table 2-7. Secondary Clarifier Design Criteria

Parameter	Units	Value
Average overflow rate	gsfd	400
Maximum month peak overflow rate	gsfd	600 to 800
Peak day overflow rate	gsfd	1,200 to 1,300
Average solids loading	pounds per square foot per day (psfd)	20
Peak solids loading	psfd	40

Using the design criteria from Table 2-7, Figure 2-9 was developed to project the number of clarifiers that would need to be in service to meet the median growth projected demands if the same type and depth clarifiers were constructed. The graph in Figure 2-9 indicates that seven clarifiers are needed to meet the maximum month conditions for 2040 and maintain 600 gsfd. The calculations summarized in Table 2-3 predict that only six secondary clarifiers are needed for median growth projections. The number can be adjusted in future planning activities as more operating experience is gained with deeper clarifiers and future regulations are better defined.

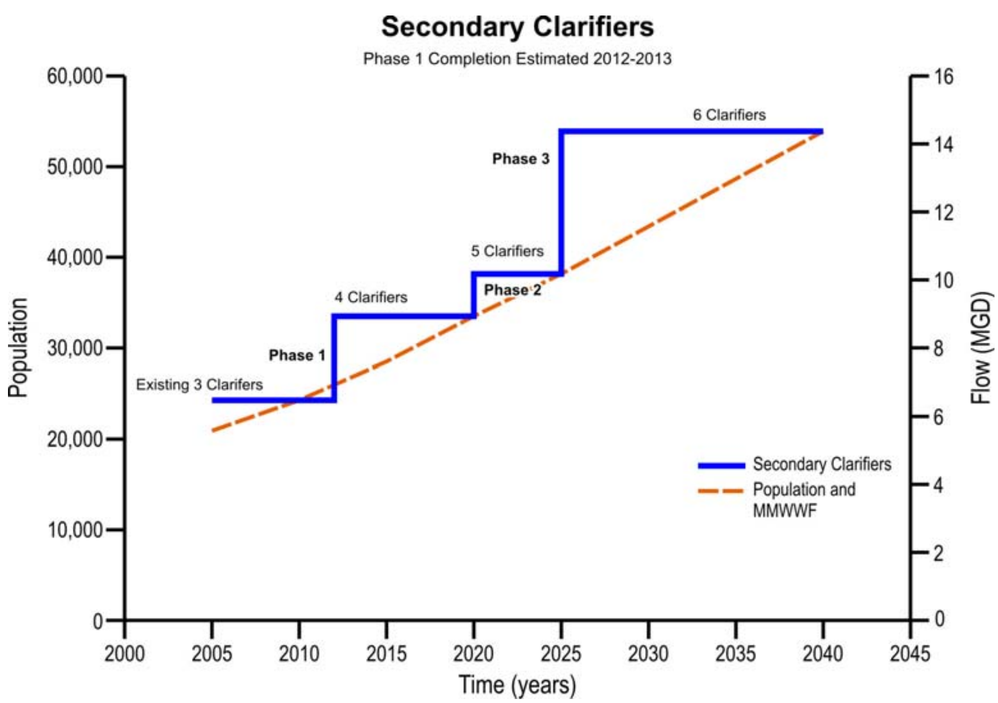


Figure 2-9. Secondary Clarifier Capacity

As shown in Figure 2-10, from 6 to 10 clarifiers may be needed depending on the growth rate actualized in 2040 and using the current design criteria for shallow clarifiers.

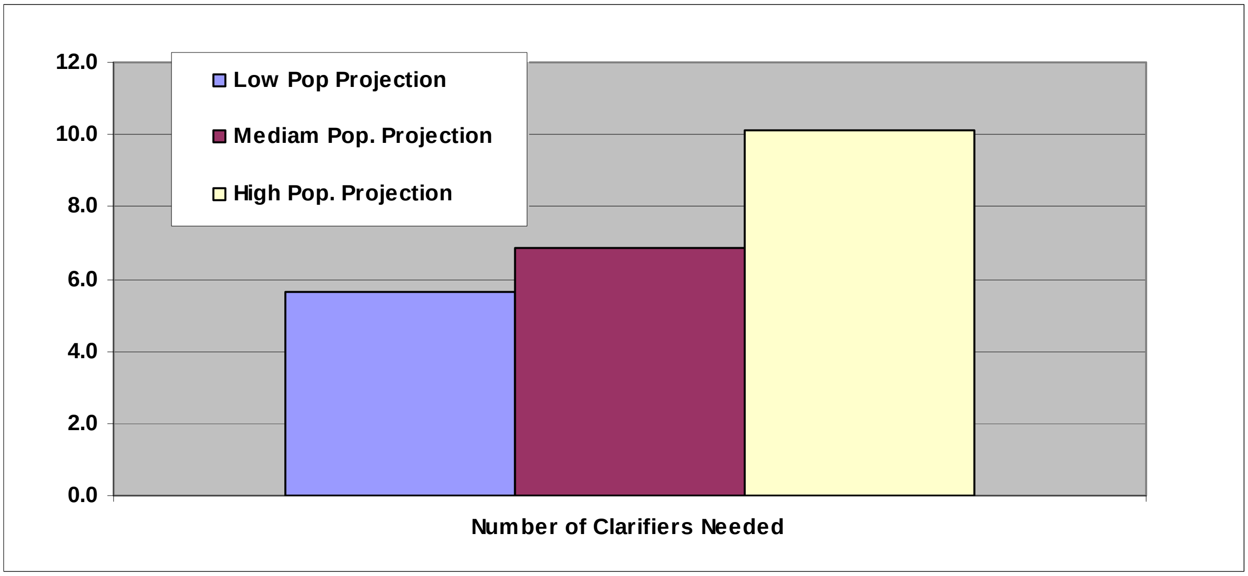


Figure 2-10. Number of Secondary Clarifiers Required for 2040 Peak Flow Projections using Existing Design Criteria

2.2.3.5 RAS/WAS Pumping

The RAS pumping system consists of four pumps. One problem is that rags frequently get caught in the pumps, requiring them to be disassembled. This takes excessive maintenance time. This problem should decrease, since the new screenings facility captures rags. Although the RAS pumps are oversized, they run at low speed and are functional. WAS pump No. 3 has a cracked volute and should be replaced.

The RAS building has no cooling upstairs where the motor control centers (MCCs) are located. To protect the MCCs, the building needs to be retrofitted with a new HVAC system including cooling, exhaust fan and inlet dampers.

2.2.4 Chlorine Contact Tank

The chlorine contact tanks are designed to provide disinfection to the wastewater prior to it being discharged to the Willamette River. Chlorine contact tanks are generally designed to provide 20 minutes of contact time at normal maximum flow, which is generally sufficient to disinfect wastewater that has already undergone secondary wastewater treatment. The current contact time for the peak hour flow is 14 minutes. Additional contact time is not provided in the outfall since the effluent is dechlorinated prior to the outfall.

The existing chlorine contact tank has a total volume of 269,000 gallons. Figure 2-11 shows the available contact time versus the average flow, peak day flow (PDF), and PHF.

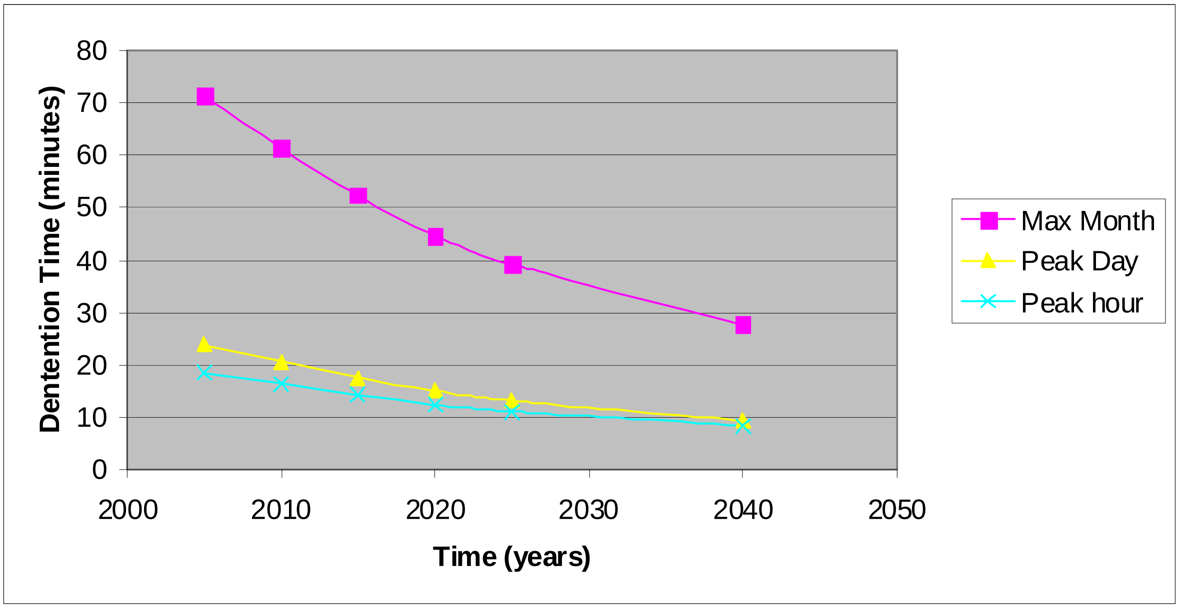


Figure 2-11. Existing Chlorine Contact Time for Future Flows

Figure 2-11 shows that the existing chlorine contact tank is able to meet the 20-minute recommended detention time easily for maximum month and average day flows until 2040. However, the existing facility is not able to provide much detention time for either PDF or PHF without using the equalization basins now. The outfall provides an additional 12 minutes of detention time at a peak flow of 18 mgd. Although Oregon does not have a standard for detention time, 20 minutes at normal PDF is commonly used. Improvements are needed to continue to ensure adequate disinfection.

2.2.5 Chlorine Disinfection System

The chlorination system uses ton cylinders of chlorine gas for disinfection. It has been upgraded in the last 4 years to include a new scale for the chlorine cylinders, new injector system, and new V-notch roto-meters. The upgrades replaced worn equipment and did not increase chlorination capacity. The chlorination system uses reclaimed water with a backup to allow the system to run off of potable water if the power fails or there is some other type of failure. The chlorination system is not automated; operators make manual adjustments based on plant flow and season. The operators anticipate rainfall-induced plant flow and adjust the chlorine accordingly. The chlorine is fed at the secondary clarifier effluent splitter box and is mixed as the chlorine is split to the two contact basins.

The plant has not had disinfection violations. During peak flow periods the plant has had individual samples that have been higher than 126 *E. coli organisms per mL* . The maximum allowable sample should not exceed 406 organisms per mL. The plant then adjusts the chlorine dosage and re-samples. This results in a decrease the geometric mean of several samples with extremely low *E. coli* counts.

There has been increased concern about the safety of using chlorine gas. Many municipalities in urban areas are converting their disinfection systems away from chlorine gas for safety reasons.

2.2.6 Dechlorination System

The dechlorination system was replaced in 1998 under a very stringent time schedule. The system was designed quickly to meet a deadline that was imposed by DEQ, and DEQ has mandated that the chlorine residual does not exceed 0.5 mg/L. The system operates by metering dilution water which is then fed to the effluent from the two chlorine contact tanks. It is delivered through poly-vinyl chloride Tees that have diffuser holes. One grab sample per day is taken to measure chlorine residual.

The bisulfite tank holds only around 300 gallons of bisulfite, and there are two pumps that feed it into a box at the end of the chlorine contact basin. Since the bisulfite tank is relatively small for the application, the tanks have to be refilled a minimum of once a week and frequently twice a week during periods of high flows. Each refill is about 150 gallons because when the tank is down to about 1/3 capacity the chemical feed pumps begin to lose pressure and are unable to feed the bisulfite to the system. There is also a lot of buildup of bisulfite crystals around the fittings, which are causing further operational problems. Improvements are needed immediately to alleviate these problems.

2.3 SOLIDS TREATMENT PERFORMANCE

Solids treatment consists of WAS dewatering followed by in-vessel composting.

2.3.1 Solids Treatment Process and Design Criteria Overview

WAS is pumped to the sludge storage tanks (SSTs) by the WAS pumps. Sludge from the SSTs is pumped to the BFPs for dewatering by centrifugal sludge transfer pumps. Polymer is added to the sludge and the sludge is dewatered by two 2-meter BFPs. Dewatered solids cake (16 percent solids concentration) are fed into a storage bin in the compost building, blended with sawdust and recycled compost, and sent to the tunnel reactors. Assuming temperature criteria are met in the reactors to document Class A pathogen reduction, compost product is then moved to an aerated cure building. If temperature criteria are not met, compost is recycled back into the system. In summary, the solids processing components include:

- Sludge storage—Two 80,000-gallon tanks with air injection for mixing
- Sludge dewatering—Two 2-meter BFPs and polymer feed system
- Composting—In-vessel type compost system with two tunnel reactors
- Cure bays—Three, aerated plus non-aerated amendment storage

2.3.2 WAS Pumps

Sludge is removed from the secondary clarifiers and pumped to sludge holding tanks using the WAS pumps prior to being dewatered by BFP. Each clarifier has its own dedicated WAS pumps with a nominal capacity of 300 gpm. Each of these pumps is equipped with a variable-speed drive. The pumps have enough capacity to keep up with solids loading from the clarifier.

2.3.3 Dissolved Air Flotation Thickener (DAFT)

The two DAFT units were taken out of service since they did not improve dewatering performance. The WWTP has been able to perform adequately without thickening, although the lack of thickening may require more capacity out of the sludge storage basins.

2.3.4 Sludge Storage Basins

The sludge holding tanks store the solids prior to being dewatered by the BFPs. The storage basins are aerated to provide mixing and prevent odor. There is a discrepancy between the total amount of WAS and the amount of sludge that is dewatered. The difference between the two numbers is about 15 percent which may be accounted for in the measurement device's inaccuracies.

2.3.5 Sludge Transfer Pumps

There are two 10-hp centrifugal transfer pumps feeding the BFPs. The pumps are Gorman Rupp trash pumps with variable-speed drives. They have been effective in meeting only BFP requirements in the 2 percent solids range. As a result, there is no thickening in the tanks and capacity for storage is compromised. Upgrading to positive displacement pumps in the future is recommended to increase storage capacity and provide a wider spot in the line in the event of required maintenance on other equipment.

2.3.6 BFPs

The plant has two BFPs onsite to dewater the sludge before it is sent to the composting facility to be processed to Class A biosolids. The two presses are capable of producing 30.6 wet tons per day in a single shift based on original design data which is all the capacity that is currently needed. If the presses are retained, booster pumps should be added to supplement recycle water pressure for washdown. The presses have been in operation for nearly 20 years. Newer technology has added benefits, and replacement of the BFPs with centrifuges should be considered. Centrifuge technology would increase solids concentration which is needed for compost feedstock to increase composter capacity.

2.3.7 Composter

Dewatered sludge, sawdust or wood shavings, and recycled compost are fed by variable-speed screw conveyors to a paddle-type mixer, where it is thoroughly mixed and then fed by drag chain conveyors to two tunnel reactors. There is no redundancy in the blending and conveyance system, and this is considered a weak link in the compost operation. The sawdust hoppers were recently upgraded by adding new augers to minimize bridging, and this has improved performance. Additional improvements or replacement of the feedstock blending hopper should also be considered.

Each tunnel is 18 feet wide by 12 feet high by 66 feet long, with an approximate detention time of 14 days at maximum loading rates. The infeed mixture is fed into the tunnels in a batch process. Each batch is compacted and the compost mass is moved through the tunnel by a hydraulically powered push door at the infeed end of the tunnel. Material is removed from the outfeed end of the tunnel by front-end loader, and moved to either the recycled compost bin or to an aerated cure pile.

Aeration is provided by two positive displacement pressure blowers located in an aeration gallery between the two tunnels. Each tunnel is divided into seven zones. Each zone has six air headers embedded in the floor of the tunnel. Air is blown up through the diffusers of each zone, using the pressure blowers. The temperature of each zone is monitored by a probe running through the compost mass the full length of each tunnel.

The composting process is monitored and controlled by a programmable logic controller (PLC). Mix ratios are determined by the PLC based on percent solids and bulk density of the sludge, sawdust, and recycled compost as entered into the PLC by the operator. Also, the feed rates can be set manually by the operator. Compost temperature is controlled by varying the output of the pressure blowers (done by the PLC), and by adjusting the header valves for each zone (done manually by the operator). Output of the pressure blowers is varied by the PLC according to the average temperature of any combination of the seven zones (selected by the operator) and a temperature set point entered by the operator. Pressure in the air headers feeding the tunnels is also monitored and the PLC controls the blowers to maintain a minimum pressure (selected by the operator). The minimum pressure setting overrides the temperature setting, i.e., the PLC will maintain the minimum header pressure regardless of the tunnel temperatures. This maintains air flow to the tunnels regardless of the tunnel temperature and thereby prevents the compost in the tunnels from becoming anaerobic.

Compost is cured in a covered structure that has three curing bays with air headers embedded in the floor. A fan-type variable-speed centrifugal blower pulls ambient air through the cure piles and exhausts it to the odor control system. Each cure bay has a modulating damper that controls air flow through the cure pile. Temperature of the cure piles is monitored by two temperature probes in each pile. Temperatures of the three cure piles are charted continuously. The system is controlled by a software program independent of the compost tunnel reactor control system. Blower speed is adjusted automatically to maintain an operator entered minimum vacuum, ensuring that ambient air is always being drawn through the cure piles. The temperature of each pile is maintained within operator entered set points by adjusting the dampers on each bay, also done automatically by the PLC.

Finished compost is sold in bulk at the composting facility on a first-come, first-serve basis. All off-site transportation of compost is done by the purchasers.

Construction of new odor control and curing systems, begun in 2003, was completed in 2004. The project consisted of construction of a covered cure pile structure and associated cure pile blower building, installation of new cure pile blowers, installation of a packed tower ammonia scrubber and modular biofilter, and installation of additional piping and a new blower to improve capture of odorous air from the sludge bin, mixer, conveyors, and work spaces. In addition, the existing vacuum blowers, previously used for tunnel aeration, were converted to capture odorous air from the top of the tunnel reactors. New doors at the discharge end of the tunnels were also installed.

Design problems and blower failures, however, prevented the curing and odor control systems from being fully functional. The cure system was not functional for all of 2005. The tunnel reactors were used for both pathogen reduction and vector attraction reduction, with no curing. Material discharged from the tunnels that had not met both the pathogen and vector attraction reduction requirements was either recycled back through the tunnels immediately or segregated in a reject pile on the opposite side of the facility from the finished compost pile, then recycled later.

In 2005, the odor control and curing system was modified from the original design. The two multi-stage centrifugal blowers in the cure building were replaced with a single fan-type centrifugal blower. The aeration trenches in each cure bay were also retrofitted with modified trench plates. The two original multi-stage centrifugal vacuum blowers and the new multi-stage centrifugal ventilation blower (installed in 2004) were also replaced with a single fan-type centrifugal blower. Testing of the new equipment was completed in December 2005. The cure system description reflects the new system operation. The tunnel doors failed and were removed.

No major equipment failures or process failures occurred in 2005, except for the cure system. Short-term shutdowns were required for replacement of portions of the mixing system, including the recycle bin live bottom screws, the sawdust feed screw, and the tunnel reactor infeed conveyor.

2.3.7.1 Evaluation of System in 2007

Detention time is approximately 22 days at average loading rates. Reject material that has not met the pathogen reduction requirements is recycled back through the tunnel reactors and material that does not meet the requirements is not released for sale. If reject material must be stockpiled, it is isolated from the finish pile and cure piles until it can be recycled back through the tunnel reactors. The measured concentration of all metals was below the Environmental Protection Agency's 40 CFR §503.13 Table 3 values. Compost produced at the Newberg WWTP meets the pollutant concentrations in 40 CFR §503.13(b)(3), the Class A pathogen requirements in §503.32(a), and the vector attraction reduction requirements in §503.33(b)(5), and is therefore exempt from the general requirements in §503.12 and the management practices in §503.14 (as stated in §503.10(c)(1)).

The annual amount of biosolids composted in 2005 is summarized in summarized in Table 2-8. The compost system is believed to be at or near capacity with this quantity of solids, although improvements may increase capacity in the future (see below). In 2040, solids production will increase to 8,340 ppd based on the median growth estimate (Brown and Caldwell, December 2005). This will more than double solids production to nearly 1,500 dry tons per year. Therefore, additional avenues for solids production need to be explored.

Table 2-8. Amount Composted in 2005

Value	Unit
11.89	mg
574.1	dry tons

Solids Capacity Analysis. The detailed composter tunnel capacity analysis is included in Appendix D. The objective of this analysis was to evaluate the capacity of the tunnel reactor for composting dewatered biosolids using sawdust and recycle from the composting process to provide the moisture content of the initial mix required for mixing and tunnel operation. A previous evaluation concluded that the system capacity ranged from 1.5 to 2 dry tons of solids per day in the dry season, and 1 dry ton of solids per day in the wet season (CH2M Hill, 1996). The original design specification was 3.5 dry tons per day (PWT, 1995). Operating experience has shown that the compost system has already been exceeded.

The capacity of the tunnel reactors will vary with the feedstocks used and the initial mix moisture content required for mixer and tunnel operation. The current capacity is estimated to vary seasonally between 1.1 and 1.4 dry tons per day.

Capacity Defining Variables. Since composting is a biological process, the primary objective of the facility design and operation is to provide conditions in the reactor that are suitable (and optimized if possible) for the activity of the organisms that do the desired work. For composting organisms, the environment is a warm, moist, and oxygenated condition. To facilitate the availability of nutrition for the organisms, it has been observed that mixing the material during composting also facilitates the composting process.

- *Compaction*—The degree of compaction associated with this style of tunnel composting is unusual. This process requires a unique aeration method designed to lift the pile to reduce friction when moving the pile as well as to help oxygen permeate throughout the composting mass.

- *Moisture content*—Biosolids moisture defines the quantity of sawdust and recycle needed to provide suitable conditions in the reactor for composting. The moisture content of the resulting mix is important for mixer function with the current mixer and for the composting function in the tunnel. The mix moisture content is affected by the moisture content of the biosolids, sawdust, and recycle.

Impact of Recycle Drying on Tunnel Capacity. Theoretically, the capacity of the tunnel can be increased by reducing the water content of any of the three components. Improved dewatering, use of dried sawdust or producing a drier recycled product will provide an increase in capacity. Figure 2-12 indicates the potential capacity for composting biosolids on a dry ton per day basis (assuming 16 percent total solids [TS]) for a range of possible recycled solids contents. As shown, an increase in recycled product moisture content from 50 to 55 percent TS increases the potential capacity by 30 percent from 1.35 to 1.75 dry tons per day. Since the analysis assumes that the sawdust to biosolids ratio is 1:1.5, the sawdust requirement would increase at the same rate as the biosolids.

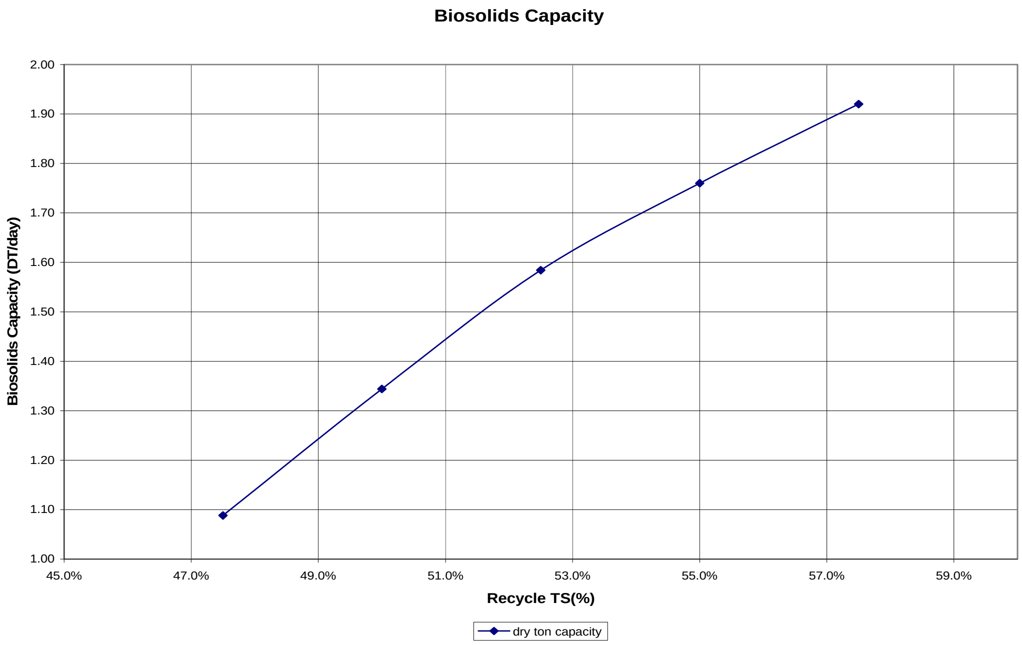


Figure 2-12. Potential Composter Capacity

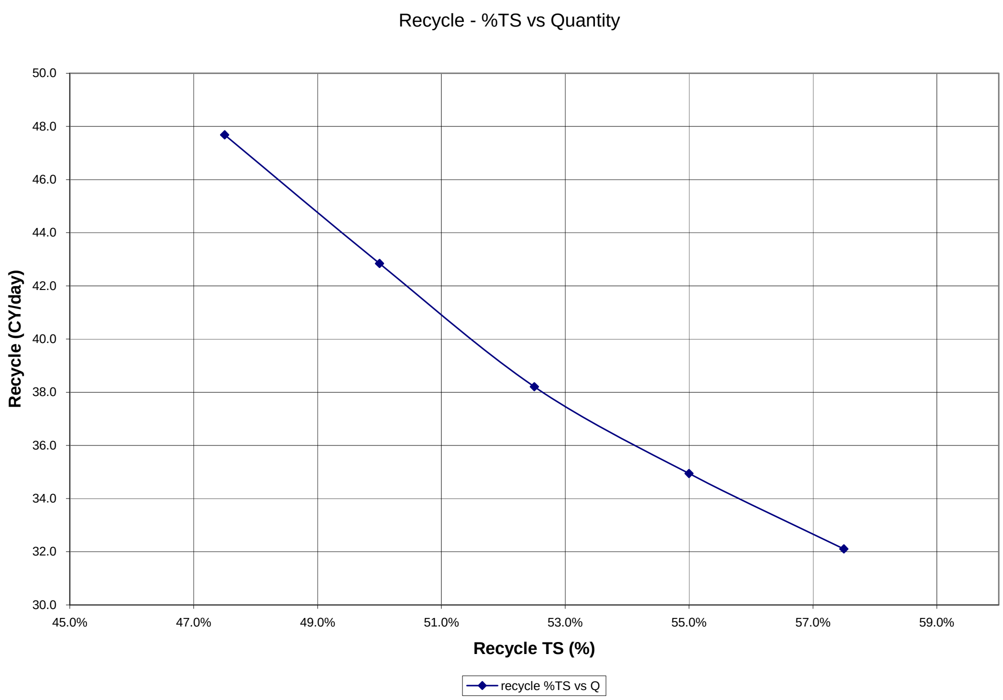


Figure 2-13. Impact of Recycled Solids Content on Amount of Recycled Product Required

Figure 2-13 shows the impact of recycled solids content on the amount of recycled product required per day. The same increase in recycled solids content (from 50 to 55 percent total solids) reduces the required recycled product from 43 to 35 cubic yards per day or a decrease of 19 percent.

The analysis assumes that the ratio of new sawdust to biosolids remains constant. However, it may be possible to replace a portion of the sawdust with the drier recycled product. This reduction in sawdust could produce a significant cost savings. On the other hand, if insufficient recycled product from the curing area is available to meet the demand, additional sawdust would have to be used to adjust the feed moisture content.

2.3.7.1 Evaluation of System in 2009

The City purchased and is currently installing a dehydrator for moisture control of the composter feed because the only available sawdust supply moisture content was limiting the capacity of the composter more than the original analysis in 2007. The dehydrator allows the City to select the moisture content of the sawdust, thus allowing additional capacity. It could potentially meet capacity through 2030. However, if the sawdust is dried too much it can become a safety consideration because of combustibility. The City needs operational experience with the dehydrator before a determination can be made on the realized capacity.

2.4 FACILITIES REVIEW

A Facilities Review was conducted as part of the Facilities Planning process. An equipment list and the equipment condition was discussed at the kickoff meeting, a walk-through of the plant was conducted, and plant personnel were interviewed. Brown and Caldwell then developed a paper assessment based on a walk-through of the plant and review of notes from interviews with plant personnel. A rating system using a 0 to 4 ranking was used for the assessment. The results were reviewed with the plant staff, and their input was incorporated into the initial spreadsheets. The results of the analysis are included in Appendix E.

Next, those items that were ranked in the 4 range of physical condition or functionality condition, or had other rankings that indicated no immediate problems, were eliminated. Numerous items fell into the equal to 3 category. Using the summary of items equal to 3 or lower, it was requested that staff define their understanding of a 3 ranking by looking at descriptors such as reliability, performance, and structural integrity, etc. An expanded listing resulted as the staff added more information to a spreadsheet to gather specific clarifying comments. Two documents entitled "Estimated Remaining Life" pages 1 and 2 of Appendix E, were then subjected to an expanded analysis using a procedure, with some abbreviations, developed by Brown and Caldwell.

Excerpts from the paper describing the process *Equipment Replacement Decision Support Tool* given at the WEFTEC 2003 conference are included in Appendix E. The paper uses a 1 to 5 ranking for condition with 1 as best, 5 as worst, while we used 0 to 4, with 4 as best and 0 as worst. These are the two documents entitled "Rating Graphs" pages 1 and 2.

Because a number of pieces of equipment which were initially ranked in the 4 category and were not put through the assessment included in the Appendix E, an argument can be made to determine their remaining life using the following generalized formula:

((Expected Life x % Base Life x % Utilization x Condition Grade) - Age) = Estimated Remaining Life, where:

- Expected Life = Useful Life from other Brown and Caldwell assessment data (plant staff have copies of these assessment spreadsheets)
- % Base Life = 1.2
- % Utilization = Estimates from Figure 3 "Utilization Factor"
- Condition Grade = 1
- Age = 20, for most equipment being considered

The results are included in the Appendix E and are used in future Facility Planning analysis and recommendations for Capital Improvements Program.

2.5 STAFFING

City of Newberg water and wastewater utility operation and maintenance (O&M) responsibilities are handled by the Public Works Department Operations Division. Several staff members have assigned duties for both the WTP and the WWTP; others are predominantly performing daily activities in water or wastewater-related facilities. An assessment was made of specific duties identified in time sheets for December 2006 and April 2007. The purpose was to compare workloads during typical wet months and dry months. This information is summarized in Appendix F.

2.5.1 Current Staffing

The City has 12 full-time employees plus one part-time administrative person (0.65 FTE) and a temporary labor man-hour pool of up to 2,000 hours annually an hourly day laborer. The positions are described in Table 2-9.

Table 2-9. Current Positions and Responsibilities

Newberg Public Works Department–Operations Division Staffing		
Positions	No.	Responsibilities
Operations Superintendent	1	Supervise and manage WTP, WWTP, pump stations, wells, and springs
Environmental Supervisor	1	Supervise environmental activities (laboratory, pretreatment program, etc.)
Water Supervisor	1	Supervise WTP, wells, reservoirs, springs, and distribution water sampling
Wastewater Supervisor	1	Supervise wastewater operation including the WWTP and major pump stations
Wastewater Maintenance Sr. Mechanic	1	Maintenance of wastewater system facilities
Water Operations Sr. Operator and Operator I	2	WTP operation and sampling
Water/Wastewater Maintenance Plant Mechanic	1	25 percent WTP maintenance, 75 percent wastewater system maintenance
Wastewater Operations Two Operator II and one Operator I	3	WWTP operator for liquids and solids systems
Laboratory Operations Lab Technician II	1	Sample collection and process lab testing
Utility Laborer	1	Available for a variety of jobs in both the water and wastewater systems, trainee for possible advancement
Administrative Position	0.65	Office Support
Temporary Laborer	varies	Available for a variety of jobs in both the water and wastewater systems. The amount of hours for this position varies with each budget cycle generally averaging 2,000 man-hours annually.

In reviewing the distribution of manpower between several areas, the following observations were made based on a review of data for a typical month, April 2007.

- The level of effort to operate and maintain the composter requires 16 percent of available labor.
- The water treatment and distribution system requires approximately 20 percent of the available labor.
- A commendable level of effort (6.7 percent) is devoted to safety and training this may decrease as the staff experience increases, however current senior staff will become eligible for retirement in the next 5 plus years.
- Earned time off (holidays, sick leave, vacation, etc.) averages 8.1 percent of the time, equivalent to one man month.
- Need to account for remaining 50 percent of the time.

A more comprehensive summary of work allocations for specific work areas is presented in Appendix F. This information was summarized from time sheets for two monthly time periods; November 20 through December 21, 2006 and March 21 through April 20, 2007, representing a wet period and a relatively dry period.

CHAPTER 3
EVALUATION OF ALTERNATIVES

3.1 INTRODUCTION

This chapter summarizes the wastewater treatment evaluations conducted to address the future needs of the Newberg Wastewater Treatment Plant (WWTP). Modifications need to be made in order to provide treatment capacity through 2030 and define the needs for ultimate buildout in 2040. The evaluation of alternatives was based on 2025 (20 years period from 2005 when the planning was started) and 2040 projections.

3.1.1 Evaluations

Options for expansion of the existing wastewater facility, use of new technologies, and inclusion of reuse facilities were analyzed, and the results are included in this chapter. Also included in this chapter are the planning level comparative costs.

3.1.2 Evaluation Process

The evaluation process included two Liquid Solids Workshops conducted by Brown and Caldwell. Liquids Solids Workshop No. 1 held on May 23, 2006, consisted of identification of the unit process deficiencies and brainstorming of technologies to be included in the analysis of wastewater treatment and biosolids alternatives analyses. An initial viability evaluation and screening was used to eliminate alternatives from further consideration. The screening tool is shown in Figure 3-1. The initial screening used the ratings +, 0, and – for relative scoring. An evaluation of the remaining viable alternatives was conducted by the Brown and Caldwell team. The evaluation was brought to Liquids Solids Workshop No. 2 held on December 14, 2006, to rank the alternatives in the group setting. If the alternative was not viable at the Newberg WWTP, it was so noted and no scoring was completed.

	Evaluation criteria								
Technology/description	Relative present worth cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	Operations and Maintenance (O&M)	Safety	Viability at Newberg WWTP	Total score
Alternative 1									
Alternative 2									
Alternative 3									
Alternative 4									

Figure 3-1. Example Screening Matrix

The preliminary list of secondary wastewater treatment and biosolids Class A technologies that were identified in the Liquids Solids Workshop No. 1 were reviewed, and the alternatives were ranked according to non-cost factors. Non-cost criteria included energy conservation, regulatory compliance, flexibility (for expansion), reliability, operability, safety, and viability at Newberg WWTP. A relative non-cost factor was also included.

After the initial screening and alternatives evaluation, the results were documented in the draft Chapter 3 and were discussed in Liquids Solids Workshop No. 2. The alternatives evaluation results were reviewed, and the rankings of the liquid and solids treatment alternatives were reviewed in the workshop. Comparative cost estimates specifically developed for Newberg applications to meet 2025 requirements were brought to the workshop. These comparative costs did not include any items common to all alternatives. The recommended alternative was identified for additional analysis and capital cost estimating for the City of Newberg's (City) capital improvements program (CIP) process.

The workshops provided a mechanism for screening the technologies to be used for the phased improvements to eliminate non-viable options from further consideration and to review the evaluation of the technologies in relation to the plant upgrades.

Comparative cost evaluations were conducted as part of the alternatives evaluation process. The comparative costs were order-of-magnitude net present value costs that included capital cost and operations and maintenance (O&M) costs for facilities needed through 2025. These costs did not include items that are the same for both alternatives. After the second workshop, the recommended alternatives were developed in more detail. The preferred alternatives for each unit process were developed for phased implementation in 2010, through 2025, and ultimate buildout in 2040. Order-of-magnitude costs were developed for the recommended phased implementation and these are included in Chapter 4.

The treatment expansion recommendations address each unit process. The unit processes addressed in this chapter are summarized below.

- Influent pumping
- Headworks
- Septage receiving
- Oxidation ditches
- Secondary clarification/solids separation
- Disinfection
- Outfall
- Reuse (by others)
- In-plant drains
- Solids handling
- Class A biosolids treatment

3.2 INFLUENT PUMP STATION (IPS)

The IPS is located at the base of a hill adjacent to Hess Creek. Influent must be pumped up over 90 feet in elevation to the headworks. The IPS was sized to handle a peak flow of 27 million gallons per day (mgd), although this requires the use of all pumps without redundancy. It lacks the capacity to handle the full range of anticipated influent flows effectively. The pump station should be refurbished in the near future to increase capacity and correct other deficiencies described below. Certain portions of the existing facilities may be incorporated into a refurbished pump station.

The design data for the existing submersible pumps is listed in Table 3-1 based on the design drawings and confirmed with the WWTP personnel.

Table 3-1. Existing Influent Pumps Design Data

Pump no.	Horsepower (hp)	Drive type	Capacity, mgd	TDH[1], feet
1	200	Variable speed	9.0	92
2	200	Variable speed	9.0	92
3	120	Variable speed	4.5	92
4	120	Variable speed	4.5	92

[1]TDH = total dynamic head

Influent flow from the collection system enters the pump station through a 42-inch-diameter pipe from a 72-inch-diameter manhole. The influent flow then is distributed to the four pumps via a lateral chamber with sluice gates on openings leading to the pumps located within confined inlet wells.

The pumps discharge to two parallel 20-inch-diameter force mains that run from the IPS to the headworks. These pipes have a capacity of approximately 24 mgd, but have insufficient velocities at minimum flow rates to keep solids in suspension.

3.2.1 Design Criteria

The IPS should be sized to accommodate the peak hour flow (PHF) requirements as well as the low flow requirements. It is critical that the influent pumping be able to handle the capacity hydraulically to keep the influent flow from backing up in the collection system at high flows and to keep solids in suspension at low flows. Per Oregon Department of Environmental Quality (DEQ) standards, the rated capacity of the IPS should be established with one of the large pumps out of service. The pump station is required to handle the range of flow conditions. The pump station design should be capable of handling low flows during the initial stages of operation and peak hour future flow rates at all stages of operation. The low flows currently are 0.5 mgd. The flow range extends from an instantaneous low of 0.5 mgd for current dry weather minimum to approximately 46 mgd for the peak hour future (2040) flow rate.

PHFs for the design conditions are summarized in Table 3-2. It should be noted that the Collection System Master Plan is ongoing, and it will identify inflow that may be able to be removed by 2020. Therefore, the design criteria should be updated during predesign of the IPS expansion and upgrade to identify the design capacity of the new pump station(s). The IPS design should include phasing that will accommodate flexibility to handle a range of expected peak flows.

Table 3-2. PHF Projections from 2005 to 2040

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
PHF[1] (mgd)	20.81	23.65	27.15	31.19	34.77	45.86

[1]Projected PHFs may decrease after inflow is removed from the collection system.

3.2.2 Summary of Existing Deficiencies

The IPS lacks the capacity to handle the full range of anticipated influent flows effectively, requires frequent maintenance, and is a safety concern. Due to wear and age of the pumps, the maximum pumping rate seen at the IPS is 21 mgd, based on City correspondence dated January 12, 2006. After a pump was rebuilt, the pump station operated at 23 mgd in November 2006. Flows to the pump station vary significantly, with minimum overnight flow rates of 1.2 to 1.5 mgd in wet weather and 0.5 to 0.7 mgd in dry weather to the peak instantaneous flow of 23 mgd experienced in November 2006. The rated capacity of the IPS should be established with one of the large pumps out of service per DEQ standards. As such, the rated capacity of the IPS is only 18.0 mgd with one large pump out of service. However, as noted above, the IPS has an observed maximum pumping capacity of 23 mgd with all pumps in service due to the condition of the existing pumps. The pumps undergo early wearing ring failure.

The pumps and wet well are hard to maintain, and access to the wet well is a safety concern. The stairs for accessing the submersible pumps and the wet well at the existing pump station are old and need to be replaced. No ventilation is currently available except by the use of portable fans. The remaining useful life of the pump station is estimated to be 5 years or less. Another issue reported is the accumulation of grit at the wet well and the associated wear on the pumps. This is related to low velocities in the pump inlets and discharge force mains during low flows.

Plant staff also report that pump clogging from rags and other debris creates frequent O&M difficulties, particularly after a wet weather flow event. The inlet and flow distribution system is not adequate to ensure that self-cleaning conditions are maintained in the pump approaches. Additionally, the large capacity pumps greatly exceed the overnight low flows. Combined, these conditions create an operating environment that induces pump clogging, particularly after periods of low flow. The low velocities experienced in the wet well for most of the operational time is not sufficient to keep the wet well clean. At the high peak flows, the velocity through the wet well changes dramatically. After long periods without rain, the first big rain that comes through washes everything from the wet well into the pumps, and then the pumps get clogged. Freeing the rags from the clogged pumps requires frequent shutdowns and high O&M costs.

The influent flow velocities at peak flows are too high according to Hydraulic Institute (HI) Standards. HI sets the standards that control the design of pump stations, one of which is that the influent flow to the wet well should not exceed 4.0 feet per second (fps). The velocity of water coming into the wet well at peak flows is 4.5 fps, which is created by the slope of the inlet pipe into the wet well. A flat section of pipe immediately upstream of the IPS is recommended to reduce the inlet velocities.

There are two existing IPS discharge pipelines to the headworks that are 20 inches in diameter. When flows exceed 24 mgd, the pipelines experience velocities exceeding 8 fps, the recommended peak velocity for pipeline design. When the pump station is expanded, the IPS discharge piping capacity must also be expanded. The addition of a parallel 24-inch-diameter pipeline will provide adequate capacity to accommodate peak flow rates through the year 2040. This assumes that the existing 20-inch ductile iron (DI) force mains are in good condition. These pipes should be inspected to determine their condition.

3.2.3 Identification of Expansion Alternatives

The City is considering either replacing the IPS or retrofitting it to allow it to meet either the 27 mgd peak flow demand or possibly to increase its capacity to meet future peak flows.

The IPS alternatives considered include:

Alternative 1: Building additional capacity at the north end of the plant

Alternative 2: Expanding the existing facilities

Alternative 3: Replacing the existing IPS with a new structure next to the existing structure

Alternative 4: Building additional capacity next to the existing and upgrading the existing IPS

Alternative 1. For hydraulic reasons, building additional influent pumping capacity at the north end of the plant was eliminated from further consideration because of the problem conditions at the existing pump station that are exacerbated by low flows. Low velocities in the influent pipe and IPS result in excessive pump clogging from rags and other debris. This condition would not be improved if additional influent pumping capacity was provided to the north. Ideally all flows should be directed to the existing pump station to maintain a high enough flow to improve the low flow conditions. In addition, it is very hard to find a pump that can reliably meet high static head and low flow conditions. If wastewater from the north flows to the existing pump station, the consistent low flow would be higher, thus making the pump selection easier and allowing for more efficient pump operation.

Alternative 2. Expanding the existing facilities is not recommended because the wet well is poorly designed and the peak influent velocity to the wet well is too high. Furthermore, the existing pump station is based on a vendor design concept that has proven to be less than reliable.

Alternative 3. Replacing the IPS is the ideal solution. This would allow the pipe entering the wet well to be re-laid, thereby reducing the velocity coming into the wet well. The new wet well could be designed to minimize the maintenance associated with clogged pumps. Due to site constraints, however, there is insufficient room to replace the existing IPS in its entirety, while maintaining current operations.

Alternative 4. Building additional capacity next to the existing IPS and upgrading the existing IPS includes the advantages of Alternative 3 while requiring a smaller footprint. The range of flows expected at the IPS is best accommodated by a dual pump station with low and moderate flows pumped by a station with a self-cleaning wet well, while higher wet weather flows are pumped by an overflow pump station with confined inlet pumps. The proposed pump station would be sited near the existing pump station and could use a portion of the existing structure. This alternative will also alleviate the conditions caused by low flows, which exacerbate the pump clogging problem. As flows to the pump station increase, this condition will diminish. A self-cleaning wet well will also minimize the clogging problem until the flows increase. Depending on the size of the expansion, this option does not preclude addition of another pump station to the north in the future to reduce the required influent pumping elevation headloss. For the purposes of this initial analysis, costs for adding new capacity and upgrading the existing pump station at the existing IPS location to 2025 flows are included. The initial screening is summarized in Figure 3-2.

	Evaluation criteria								
Technology/description	Relative present worth cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	O&M	Safety	Viability at Newberg WWTP	Total score
Alternative 1: Building additional capacity at the north end of the plant	-	+	0	-	+	-	+	No/unless combined with No. 4	0
Alternative 2: Expanding the existing facilities								No	
Alternative 3: Replacing the existing IPS with a new structure next to the existing structure								No	
Alternative 4: Build additional capacity next to the existing and upgrade the existing IPS	0	0	0	0	+	+	+	Yes	3

Note: A higher score is better.

Figure 3-2. Initial Screening for IPS Alternatives

3.2.4 Evaluation of Viable Alternatives

For the purpose of this analysis, Alternative 4 is considered for expansion to capacity in 2025 in order to increase the minimum flows to the IPS. The IPS requires immediate substantial reconstruction to provide the required flow capacity range and to improve maintainability and safety. A potential pump station layout using the existing wet well for a portion of the flow is included as Appendix G.

It is recommended that a section of the influent pipe be elevated sufficiently to remove the slope to the IPS. If the City decides to remove the influent pipe from the creek, the slope of the influent pipe into the wet well could be improved, and the new pump station could be located adjacent to the existing but at a higher elevation. For this analysis, it is assumed that the Hess Creek pipeline will be raised above the Hess Creek 100-year flood plain.

During the facility planning process, the Motor Control Center building location for the IPS was discussed as part of the reuse design process. It was determined that a location to the west of the Administration Building would be optimum. This location avoids the influent piping at the east of the Administration Building, is in adequate proximity to the IPS, and avoids the additional costs of construction adjacent to the IPS on a steep slope and where the site is already constrained.

3.2.5 Cost Estimate

Order-of-magnitude construction cost for Alternative 4 of the IPS improvements is $2,723,000. Since there were no alternatives to compare, the present worth analysis was not conducted.

3.3 HEADWORKS

The headworks provides a preliminary process that involves screening and grit removal. Screenings removal from wastewater is necessary to allow proper operation of downstream mechanical equipment. Grit erodes mechanical equipment and piping, and collects in downstream treatment process tankage, reducing usable volume.

The headworks was recently upgraded with two new screens, screenings compactors, and a redundant grit classifier. The upgraded headworks was sized for 27 mgd based on the theoretical peak pumping capacity of the existing IPS. The original design criteria for each screen was 27 mgd each. However the capacity was compromised when actual field conditions did not match the as-built drawings and the screens could not be submerged to get the full capacity. With the actual submergence, the screens have a capacity of 21 mgd each and the bypass pipe has a capacity of 6 mgd.

The grit removal process was originally sized for 18 mgd. Above 18 mgd, design provisions allow flows to be manually bypassed around the grit process to avoid grit washout. Plant staff operate the grit removal without using the bypass feature.

3.3.1 Design Criteria

The headworks is typically sized to accommodate the PHF requirements with one unit out of service. It is critical that the headworks be able to handle the hydraulics to keep the influent from overflowing the headworks structure. It is not critical that the headworks achieve the same level of treatment for these brief periods of time when the flows are at their peaks, since the performance does not impact discharge permit compliance. However, based on reliability Class 1 reliability guidance policy, a backup screen is required with at least manual cleaning. The headworks consists of two screens, a bypass pipe for flows (6.0 mgd) greater than one screen can handle, and an aerated grit tank. Projected peak flows at the facility are listed in Table 3-3.

Table 3-3. Projected Peak Headworks Flows

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
Peak day flow (PDF) (mgd)	16.17	18.75	21.98	25.78	29.35	41.40
PHF[1] (mgd)	20.81	23.65	27.15	31.19	34.77	45.86

[1]Projected PHFs may decrease after inflow is removed from the collection system.

The redundancy and reliability requirements for the headworks are:

- One backup unit for mechanical screens
- A minimum of two units for grit removal

As noted in Chapter 2, the capacity of the headworks (screens/overflow pipe and the grit tank plus the overflow) of 27 mgd will be exceeded around 2015, based on PHF. The grit tank is already undersized and needs to be expanded to treat all flow to protect downstream equipment and minimize tank cleaning intervals. The headworks capacity in relation to flow was summarized in Figure 2-7.

3.3.2 Identification of Siting Alternatives

The alternatives identified at the kickoff meeting held on May 12, 2005 include:

Alternative 1: Adding capacity at new headworks at the north end of the plant to serve a new pump station also to be located at the north end of the plant.

Alternative 2: Expanding the existing headworks.

3.3.3 Initial Siting Evaluation of Viable Alternative

Building another headworks (and IPS) to the north would intercept the influent flow prior to it traveling downgrade to the existing IPS, which is at a lower elevation than the plant site. Alternative 1 is not considered further because it is not advisable to have screenings and grit facilities in more than one location on the site. Alternative 2 is considered further.

3.4 INFLUENT SCREENS

The headworks includes two channels, two perforated-plate screens, and two screenings compactors.

3.4.1 Summary of Existing Deficiencies

The 10-millimeter opening screens were installed in 2003, and each screen is rated for a peak capacity of 21 mgd. If flows exceed 21 mgd, a bypass pipe is designed to handle 6 mgd for a total of 27 mgd when one screen is out of service. The headworks upgrade also included screenings washer/compactors and a redundant grit washer.

An emergency bypass pipe was included to provide peak capacity of 27 mgd through one screen should power to the other screen fail or be out of service. Peak flows in 2025 are expected to be 35 mgd unless infiltration/inflow (I/I) can be reduced. Two channels and one screen would need to be provided to ensure that the headworks has sufficient capacity with one unit out of service. One of the new channels would replace the overflow pipe and be sized to handle to peak flow of 21 mgd if the largest screen is out of service.

3.4.2 Evaluation of Viable Alternatives

The screenings alternatives were evaluated in 2002 during the Newberg Dump Station and Headworks Study conducted by Brown and Caldwell. The most cost-effective screen was chosen at that time. Plant staff have had positive experiences with the existing screens and screenings compactors. These screens can be raised and swiveled out of the channel should the power fail.

Each of the existing screens was designed for installation under the conditions in Table 3-4. Also noted is the capacity based in actual installation. The screens could not be recessed for full capacity.

Table 3-4. FSM Perforated Plate Screen Design Data

Design data	Value
Peak design flow, mgd (with installation per manufacturer's recommendation)	27
Peak actual capacity flow, mgd (could not be installed per manufacturer)	21
Average flow, mgd	4
Minimum flow, mgd	0.5
Channel width, inches	48
Channel depth, inches	68
Maximum upstream water depth, feet	5.67
Minimum downstream water depth, feet	0
Maximum allowable operating headloss through screen, inches	7
Maximum differential head the screen shall withstand, feet	5.75
Channel invert elevation	171
Deck floor elevation	
Upstream	176.67
Downstream	176
Screen angle from horizontal, degrees	60
Screen panel circular opening size, mm	10

For the purposes of this analysis, it is assumed that the expansion will include the same type screening equipment as existing for ease of O&M and because the screening equipment were determined to be cost-effective in the previous study.

At a minimum, a redundant channel and new screen and channel are needed for ultimate buildout at average population projections. An additional 4 to 5 mgd may be needed in 2040 for peak flows if the I/I is not controlled. There are two alternatives for expansion:

Alternative 1: Expand to include two new screens at the headworks. This would provide one more screen to meet peak flows and one screen to provide redundancy.

Alternative 2: Expand to include two more channels and one screen. The channel would have a large bar rack.

These channels will be installed on the east side of the existing headworks as shown in Figure 3-3. It is recommended that at the time of new screen installation, the existing bypass pipe be replaced with a bypass channel that can take all the flow if the largest screen is out of service. The flow will need to be equally distributed between the screens with a new influent flow distribution box. Emergency power should be added to ensure critical headworks functions can continue in the event of a power outage. Odor control should also be provided as a good neighbor policy.

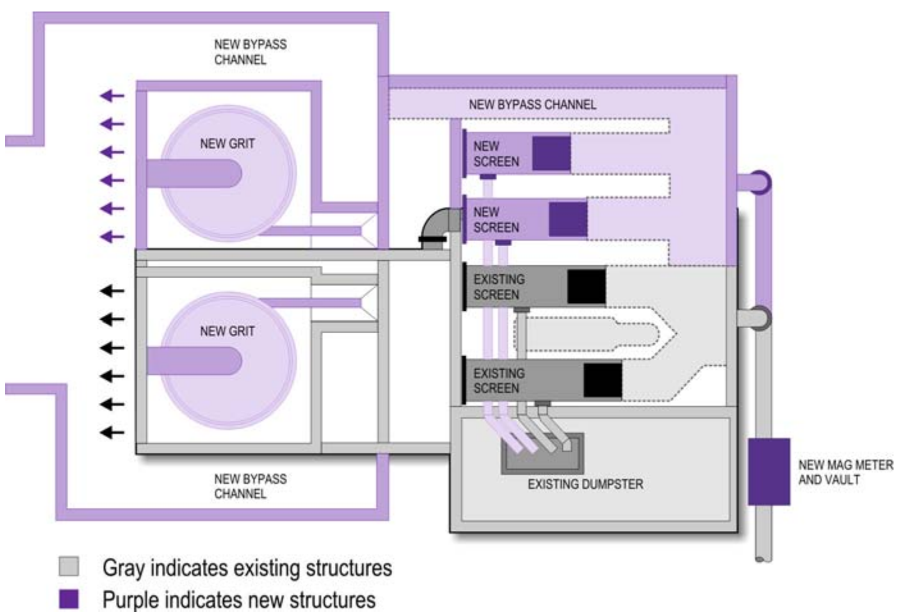


Figure 3-3. Headworks Channel Installation

3.4.3 Comparative Cost Estimates

Order-of-magnitude costs for the screening improvements are summarized in Table 3-5.

Table 3-5. Costs for the Screening Improvements

Technology/description	Construction cost, dollars
Alternative 1: Add two redundant screens (one is the redundant channel)	1,360,000
Alternative 2: Add one new screen and a bypass channel with bar rack	1,060,000

The O&M costs will be similar for both alternatives. The cost of operating the additional bar screen equipment will be offset by the labor cost of maintaining a bypass channel and raking the bar rack that accompanies it.

As a result of the analysis and Liquids Solids Workshop No. 2, Alternative 1 is the preferred alternative. Plant staff have had trouble with a manually-cleaned bar rack in the past and removed it. The rack plugged during high flow events when unattended. Plant staff did not want a manual bar screen, so Alternative No. 2 has been dropped from further consideration.

3.5 INFLUENT FLOW DISTRIBUTION AND METERING

The influent flow rate is measured at flow meters located on the influent piping at the headworks, as shown in Figure 3-4. According to the manufacturer's recommendations, the flow meters are not installed with sufficient straight runs of pipe before and after the meters. Magmeters require a straight run of pipe equivalent to five pipe diameters before the meter and a straight run equivalent to two pipe diameters after the meter to ensure accurate readings.

Figure 3-4. Influent Flowmeters at the WWTP Headworks

New flow distribution and flow monitoring will need to be provided with any alternative. The new flow monitoring will be provided a sufficient distance upstream of the headworks (approximately 20 feet) for the correct installation of magmeters to more accurately monitor the influent flow rates. The magmeters will be placed in a vault for ease of maintenance.

The capacity of the two existing 20-inch-diameter influent lines into the headworks will be expanded by adding a parallel pipe so that the velocity in the influent pipelines does not exceed 8 fps (about 11 mgd per pipeline). The flow monitoring for the new pipeline will be in the same flow meter vault to be provided for the existing pipes.

3.6 GRIT

The plant has one aerated grit tank and a bypass channel. Grit that settles to the bottom of this tank is pumped to two grit classifiers. Grit classifiers discharge classified grit into a dumpster. The tank is open to the environment, and no odor control mechanisms are in place.

3.6.1 Summary of Existing Deficiencies

Using the planning criteria of providing grit removal at all flows, the grit removal system is already out of capacity. The aerated grit tank is sized to meet a peak influent flow of 18 mgd while maintaining a theoretical hydraulic detention time of 3 minutes. Flows greater than 18 mgd are designed to be bypassed to the equalization basin. Current operation is to send all flows through the grit basin. Grit that is washed out accumulates in the oxidation ditches and can cause wear on downstream equipment. Plant staff expressed interest in planning for adequate grit removal for all flows. A grease blanket builds to 2 to 3 feet and is impossible to remove from the existing basins. An alternate configuration for effluent discharge from the basin is needed to facilitate grease and scum removal.

3.6.2 Alternatives Considered

The merits of several grit system alternatives were evaluated and both the advantages and disadvantages of each configuration were considered. The alternatives are representative of those options most applicable to the conditions to be encountered, but do not represent all possibilities.

The preliminary list of alternatives identified for grit removal include:

Alternative 1: Stacked tray separator
Alternative 2: Vortex grit settling with agitation
Alternative 3: Air vortex grit separator
Alternative 4: Free vortex separator
Alternative 5: Existing system expansion

Table 3-6 provides information about the grit removal alternatives.

Table 3-6. Grit Removal Alternatives

Information	Alternative 1 Stacked tray separator	Alternative 2 Vortex grit settling with agitation	Alternative 3 Air vortex separator	Alternative 4 Free vortex separator	Alternative 5 Existing system expansion
Maintenance requirements	No moving parts so low maintenance; only one unit for flows 1 to 5 mgd to maintain	Impeller/paddle with motor must be maintained	No in-basin moving parts; air system requires compressor	No moving parts so low maintenance requirements	No in-basin moving parts; air system requires compressor
Technology history	Only one permanent installation to date (Florence, OR)	Multiple manufacturers and installations	No known competitor; multiple installations	Multiple installations but only one manufacturer	Engineer designed
Low headloss through equipment	Headloss is less than 12 inches	Headloss is only 1/4-inch (Jeta)	Headloss is less than 1/4-inch	Headloss is 18 to 72 inches depending on number of teacups and size	Headloss would be same as existing
Removal efficiency	95 percent removal for 120 microns (eight 6-foot-diameter trays)	95 percent removal efficiency for 300 micron (Jeta); 65 percent removal efficiency for 150 micron (Jeta)	95 percent removal of grit 50 microns and larger	95 percent of design particle size (generally between 100 to 125 microns)	Has not been effective
Disadvantages	New technology; limited operating experience	Not effective with large variations in flow; manufacturer stated that capacity should be halved	Not effective with large variations in flow	Highest relative cost by order of magnitude; local municipality recently replaced the technology due to shortcomings	Large land area; has not been effective at removing grit

An initial screening analysis is summarized in Figure 3-5.

Technology/Description	Evaluation criteria								Total score
	Relative present worth cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	O&M	Safety	Viability at Newberg WWTP	
Alternative 1: Stacked tray separator	+	+	0	+	+	0	0	Yes	4
Alternative 2: Vortex Grit settling with agitation	0	0	0	0	0	0		Yes	0
Alternative 3: Air vortex grit separator	-	-	0	0	0	0		Yes	-2
Alternative 4: Free vortex separator	-							Yes	-1
Alternative 5: Expand existing system	-	-	0	-	0	0	0	Yes	-3

Note: A higher score is better.

Figure 3-5. Initial Screening Alternatives

The vortex-type and plate gravity settling grit removal systems score the highest and are the most feasible for use at the Newberg WWTP. The added cost of removing the contaminated soils adjacent to the headworks makes Alternatives 3, 4, and 5 are more expensive and less feasible and therefore removed from further consideration.

3.6.3 Evaluation of Economically Viable Alternatives

Alternative 1: Stacked Tray Separator. The stacked tray settling system removes grit using a series of stacked plates. Flow is distributed across the plates and grit settles onto the plates. The plates are slanted to a center well, where grit accumulates before removal. The large surface area created by the stacked plate system increases grit removal efficiency and decreases the footprint. Higher flows are accommodated by increasing the number of plates. Eutek is the only manufacturer with its Headcell Plate Gravity Settling Grit Removal System. Advantages and Disadvantages of this alternative are provided in Table 3-7.

Table 3-7. Alternative 1—Stacked Tray Separator (Eutek Headcell)

Advantages	Disadvantages
▪ Stainless steel construction ▪ Few mechanical parts ▪ Low maintenance requirements ▪ Low capital cost for manufacturer-provided materials ▪ Lower costs for simpler concrete form work ▪ Grit suspension equipment for pumped grit system included ▪ Performance not affected by influent flow rate ▪ Smaller footprint ▪ Small inlet channels needed ▪ Higher grit removal efficiency ▪ Can be retrofitted into existing grit basin	▪ Grit pump not included with package ▪ Deeper construction

Alternative 2: Vortex Grit Settling with Agitation. Vortex grit removal systems work by creating a vortex flow pattern by both forcing the flow with a rotating turbine and designing a tangential inlet. Settled grit accumulates in a bottom hopper that is periodically pumped down. The vortex pattern helps settle the grit through gravity and centrifugal forces. The vortex flow pattern also creates a longer fluid flow pathway to increase settling time. Together these help to create a smaller footprint than conventional aerated grit removal systems. Because of the strict requirements for creating a vortex pattern, large, straight inlet channels are typically required, which poses problems with settlement of grit in the inlet channel. Manufacturers under consideration include:

- Smith and Loveless Pista Grit Vortex Grit Removal System
- Waste Tech XGT Vortex Grit Removal System
- Hydro International Grit King Vortex Grit Removal System

Information for these products is provided in Tables 3-8 to 3-10.

Table 3-8. Alternative 2—Smith and Loveless Pista Grit Vortex Grit Removal System

Advantages	Disadvantages
▪ Stainless steel construction ▪ Grit pump included with the package ▪ Forced vortex means that performance is not as affected by influent flow rate	▪ Large inlet channels required ▪ Larger footprint ▪ Moderate capital costs associated with manufacturer provided materials ▪ High costs associated with complex formed concrete not included in capital cost ▪ More mechanical parts (mixer motor and grit pump) ▪ More maintenance requirements ▪ Requires additional method to suspend grit prior to pumping ▪ Lower grit removal efficiency

Table 3-9. Alternative 2—Waste-Tech XGT Grit Removal System

Advantages	Disadvantages
▪ Stainless steel construction ▪ Grit pump included with the package ▪ Forced vortex means that performance is not as affected by influent flow rate ▪ Lowest capital cost for manufacturer provided materials ▪ Shallow construction ▪ Grit suspension equipment for pumped grit system included	▪ Large inlet channels required ▪ Larger footprint ▪ High costs associated with complex formed concrete not included in capital cost ▪ More mechanical parts (Mixer motor and grit pump) ▪ More maintenance requirements ▪ Lower grit removal efficiency

Table 3-10. Alternative 2—Hydro International Grit King Grit Removal System

Advantages	Disadvantages
▪ Stainless steel construction ▪ Grit pump included with the package ▪ Fewer mechanical parts ▪ Less maintenance ▪ Grit suspension equipment for pumped grit system included ▪ Higher grit removal efficiency	▪ Large inlet channels required ▪ Larger footprint ▪ Highest capital cost for manufacturers provided materials ▪ High costs associated with complex formed concrete not included in capital cost ▪ Deep construction ▪ Lack of forced vortex means that performance is affected by influent flows

3.6.4 Comparative Cost Estimates

Order-of-magnitude costs for the grit removal improvements are summarized in Table 3-11. These costs were developed for another project and scaled for discussion at the Liquids Solids Workshop No. 2. No additional cost estimating was requested.

Table 3-11. Costs for the Grit Removal Improvements

Information	Stacked tray separator, dollars	Grit settling with agitation, dollars	Air vortex separator, dollars	Free vortex separator, dollars
Approximate capital cost	51,000	92,000 (two stainless steel tanks included)	90,000 (two units excluding a compressor	790,000 to 880,000 (ten units)
Cost per mgd	17,000	30,720	30,000	293,000
Cost per 21 mgd	357,000	645,000	630,000	6,200,000

Note: Comparative cost based on recent experience. Assumes two units in service and incremental cost increasing from 17.5 mgd each (2025) to 21 mgd each (2040) is negligible.

From the above options, we recommend using a Eutek Headcell grit removal system. This system is less expensive and easier to maintain, because it uses a purely physical separation and includes fewer mechanical parts. In addition, the Eutek Headcell does not require long influent channel transition lengths that are required by vortex grit removal systems, and it can be retrofitted into the existing aerated grit chamber. A minimum of two grit removal tanks to meet redundancy requirements will be assumed. A bypass channel will be included in the design if one or both grit removal units are out of service. This will reduce cost and footprint for the grit removal system.

3.7 ODOR CONTROL

There are odors associated with the screenings and dumpster at the headworks. These odors are prominent in the warmer months of the year. Odor control should be considered for the headworks improvements.

3.8 SEPTAGE RECEIVING

Trucks currently discharge septage into a catch basin. The septage then flows down to the IPS and is pumped back up to the headworks. Septage receiving was studied in a previous report entitled *Final Report for the Recommended Plan City of Newberg Dump Station/Headwork Studies* (*Final Dump Station/Headworks Studies* report) (Brown and Caldwell, June 2002). The conclusions are considered to be valid for this analysis.

3.8.1 Design Criteria

Design criteria was based on septage hauler truck size and is noted in the *Final Dump Station/Headworks Studies* report.

3.8.2 Summary of Existing Deficiencies

The catch basin was not designed to receive septage. The septage flows down to the IPS and takes needed influent pumping capacity. The additional pumping is also not cost-effective.

3.8.3 Identification of Expansion Alternatives

The *Final Dump Station/Headworks Studies* report considered several septage receiving stations including Lakeside, JWC, and FSM. Based on the 2002 analysis, the recommended improvements include modifications to the road southeast of the headworks (including a trench drain and catch basin), a Lakeside 31SAP-type septage receiving station, a buried septage receiving tank, duplex pumps in the septage receiving tank, piping to transfer the septage to the screening channel of the headworks, and a new access road around the north side of the headworks. The screenings from the station will be bagged, so no roof over the septage screenings dumpster will be required. Vactor trucks can discharge on the ramp leading to the septage receiving station. Rocks and debris will be manually removed.

3.8.4 Cost Estimate

The updated cost estimate for septage receiving improvements is $695,000.

3.9 SECONDARY BIOLOGICAL TREATMENT

This section evaluates existing secondary process capacity with respect to population growth and effluent permit compliance. The Newberg WWTP currently uses two oxidation ditches for secondary biological treatment. The City has expressed interest in continuing to use the oxidation ditch process because of its low energy and maintenance costs and its ability to treat a wide variation in flows and loads. The City has an interest in acquiring the adjacent Baker Rock property for expansion of the secondary system. However, should land not be available to expand the WWTP with the current technology, additional processes were also considered.

3.9.1 Design Criteria

The design of the secondary treatment system are based on maximum month flows. Design flows to the secondary treatment process for the 2010 and 2025 expansions are listed in Table 3-12.

Table 3-12. Design Flows for Secondary Treatment

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
MMDWF[1] (mgd)	3.52	4.08	4.78	5.61	6.39	9.01
MMWWF[2] (mgd)	7.52	8.71	10.21	11.98	13.64	19.24

[1]MMDWF = maximum month dry weather flow
[2]MMWWF = maximum month wet weather flow

The capacity requirements are more stringent if nitrogen removal is considered than if nitrogen removal is not required. The timing for improvements is given for both conditions.

3.9.2 Summary of Existing Deficiencies

The existing secondary treatment units do not have capacity for future flows and loads. The oxidation ditches have structural deficiencies that need to be repaired. For the purposes of this alternatives analysis, it is assumed that the existing basins will be rehabilitated and additional capacity will be provided by new basins and associated secondary clarification.

3.9.3 Identification of Expansion Alternatives

The screening process included evaluation and ranking of potential treatment technology alternatives to upgrade the WWTP to address the anticipated growth, regulatory requirements, and to confirm the cost-effectiveness of continuing the use of the existing technology. Potential treatment technologies were identified in the Liquids Solids Workshop No. 1. These technologies include:

Alternative 1: Conventional oxidation ditch
Alternative 2: Vertical loop reactors (VLR) oxidation ditch
Alternative 3: Cannibal
Alternative 4: Membrane bioreactors (MBRs)

Alternatives 1 though 3 use extended activated sludge which is land-intensive. Alternative 4 was included to compare to a treatment technology with a smaller footprint. The initial screening of the technologies included ranking them against three non-cost factors and relative cost comparison. Table 3-13 lists potentially viable liquid stream technologies.

Table 3-13. Liquid Stream Technology Descriptions Identified as Potentially Viable

Alternative	Technology/description	Comments
Conventional oxidation ditch	Wastewater is treated with a suspended sludge in a biological reactor the shape of a racetrack.	Current technology used at the Newberg WWTP. Low O&M.
VLR oxidation ditch	Wastewater is treated with a suspended sludge in a biological reactor the shape of a racetrack turned on its side. This system differs from a conventional oxidation ditch because the looped flow is made over and under the horizontal baffle. The process uses up-front aerated-anoxic looped reactors with mechanical aeration technology followed by second-stage reactors using fine bubble diffusers.	New technology with few recent municipal installations of this technology in the U.S. A new installation is being planned in the City of Albany, OR. Typical vertical loop reactors tanks are arranged in series, usually with three or more rectangular tanks—all of equal size. Storm flow problems are addressed by diverting raw wastewater (diluted by the rain water) into the second, third or fourth tank. The VertiCel™ process is ideal for medium to large size plants above 10 mgd.
Cannibal	Screens are used in a biological process to reduce solids in basins, which purports to eliminate solids handling.	Theoretically reduces sludge quantities for disposal. Additional solids removed from the system are sent to the landfill. Does not show benefits to plants that accept drinking water treatment plant solids because of iron content.
MBRs	Modification of conventional activated sludge that uses membranes as a barrier to solids such that secondary clarifiers are not required, and therefore takes up less footprint than the existing oxidation ditches.	MBRs produce high-quality effluent and they provide a high level of process control. Requires smaller footprint but is much higher in present worth costs because of high energy and chemical costs. Membranes clog unless very high mixed liquor is maintained. Sensitive to flow fluctuations greater than 2.5 times average flow.

Based on a previous analysis that compared the oxidation ditch process to other biological processes, the oxidation ditch is still the lower life-cycle cost over MBRs because of the low hp aeration and O&M costs.

The MBR alone does not treat large influent flow variations and therefore is no longer considered a viable option for the Newberg WWTP. However, in the future if land requirements become limiting at the plant, the MBR process should be reconsidered as an expansion alternative to treat a base flow and the existing oxidation ditches can treat the varying peak flows.

The Cannibal manufacturer does not recommend that the process be used when drinking water treatment plant solids are accepted. Drinking water treatment solids from the Newberg Water Treatment Plant (WTP) are accepted at the Newberg WWTP. Therefore the Cannibal process is not viable for the Newberg WWTP.

The alternatives ranking is shown in Figure 3-6.

	Evaluation criteria								
Technology/description	Relative present worth cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	O&M	Safety	Viability at Newberg WWTP	Total score
Alternative 1: Conventional Oxidation Ditch	+	+	0	0	+	+	0	Yes	4
Alternative 2: VLR Oxidation Ditch	+	+	0	0	+	+	0	Yes	4
Alternative 3: Cannibal								No	
Alternative 4: MBR	-	-	0	0	0	-	0	Maybe/future	-3

Note: A higher score is better.

Figure 3-6. Screening of Liquid Stream Technologies—Rating Table

3.9.4 Evaluation of Viable Alternatives

The VLR would provide the added benefit of lowering the surface area exposed to the warming effect of the sun, to minimize the thermal load to the river; and it has additional energy efficiencies from submerged aeration. For the purposes of this comparison, the alternatives for both expansion using the oxidation ditch and vertical loop reactor are considered further. For the analysis, it was assumed that the existing basins would remain conventional activated sludge and the new capacity would be provided by the alternative technologies.

3.9.5 Comparative Cost Estimates

Order-of-magnitude costs for the secondary treatment improvements are summarized in Table 3-14.

Table 3-14. Costs for the Secondary Treatment Improvements

Description	Cost, dollars	O&M	Total present worth
Alternative 1: Conventional oxidation ditch	8,489,000	3,981,000	12,470,000
Alternative 2: VLR	17,417,000	3,785,000	21,202,000

Note: Net present value includes O&M for 20 years to 2025.

Based on the results of this analysis and consensus at the Liquids Solids Workshop No. 2, expansion with the current oxidation ditch process is the preferred alternative.

3.10 DISINFECTION PROCESS

The disinfection process is a chlorination system that uses ton cylinders of chlorine gas. The chlorination system has been upgraded in the last 4 years. The upgrades replaced worn equipment, but did not increase chlorination capacity. Operators make manual adjustments based on plant flow and season. The chlorine is fed at the secondary clarifier effluent splitter box and is mixed as the chlorine is split to the two contact basins.

3.10.1 Design Criteria

Disinfection design is typically based on PHF. The chlorine contact tank is designed to provide disinfection to the wastewater prior to being discharged to the Willamette River. Chlorine contact tanks are generally designed to provide 20 minutes of contact time at normal maximum flow, which is generally sufficient to disinfect wastewater that has already undergone secondary treatment. The 2005 contact time for the PHF is 14 minutes. Since the effluent is dechlorinated at the outlet from the chlorine contact chamber, contact time in the outfall pipe is not effective and is not considered as part of the required capacity.

The PHFs for the design conditions are summarized in Table 3-15. It should be noted that the Collection System Master Plan is ongoing. The Collection System Master Plan will identify inflow that may be able to be removed by 2020. Improvements may be postponed if inflow can be removed.

Table 3-15. Peak Hour Flow Projections from 2005 to 2040

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
PHF[1] (mgd)	20.81	23.65	27.15	31.19	34.77	45.86

[1]Projected PHFs may decrease after inflow is removed from the collection system.

3.10.2 Summary of Existing Deficiencies of Disinfection

Immediate improvements are needed that include chemical induction mixer(s) at the chlorine injection point, scum removal, improved effluent flow monitoring, and automatic disinfection control strategy. Roof drainage needs to be re-routed out of the contact basin.

Figure 3-7 shows that the existing chlorine contact tank is able to meet the 20-minute recommended detention time for maximum month and average day flows until 2040. However, the existing facility is not able to provide sufficient detention time for either PDF or PHF, unless high-rate disinfection is adopted. DEQ requires that the contact time requirement be met at PHF.

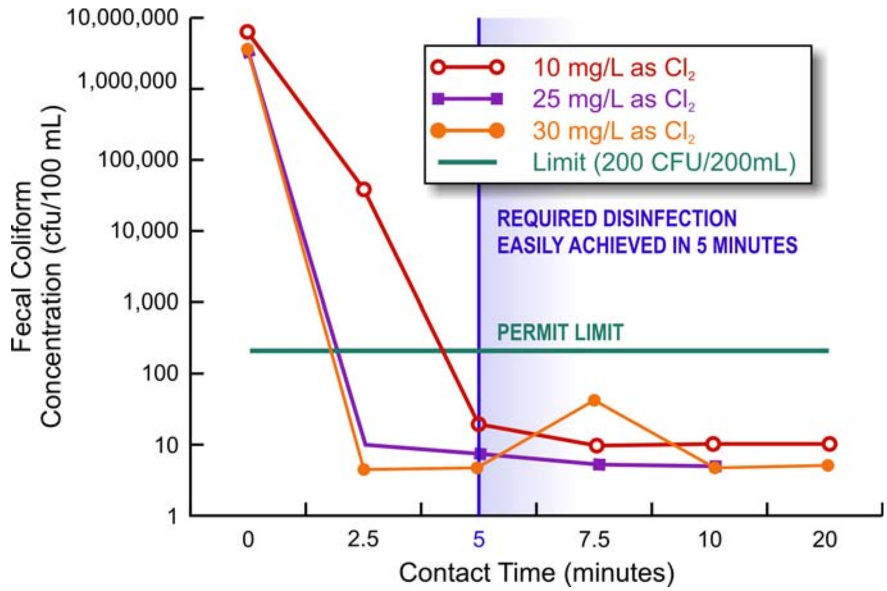


Figure 3-7. Effects of High Rate Mixing on Disinfection

Although the National Pollutant Discharge Elimination system (NPDES) permit is for disinfection to meet an *E. coli* bacteria, the disinfection should be adequate based on the analysis summarized in Figure 3-7. Fecal coliform concentration of 200 is roughly equivalent to an *E. coli* concentration of 126. The existing chlorine contact time provides 11 minutes of detention time at PHF of 35 mgd in 2025. Theoretically, this contact time is sufficient for adequate disinfection using high rate disinfection for this flow. Actual operating experience will verify the adequacy in the near term.

There has been increased concern about the safety of using chlorine gas. The City has expressed interest in the feasibility of other disinfection alternatives including ultraviolet (UV) light disinfection or onsite hypochlorite (HOCl) generation for expansion needs. However, since the reuse facility will require an effluent chlorine residual, a small HOCl system would also be needed if UV were implemented.

3.10.3 Identification of Alternatives

Alternatives for expansion include:

Alternative 1: High-rate disinfection: addition of chemical induction mixer and high chlorine dosing

Alternative 2: Additional contact basin

Alternative 3: Additive to Alternatives 1 and 2 for bulk delivered or onsite generation of HOCl

Alternative 4: UV Disinfection

Alternative 1. High-rate disinfection is defined as disinfection obtained by higher chlorine or hypochlorite doses at shorter detention times. This also requires that increased chemical doses be used for dechlorination. The theory is based on the use of the detention time-dose product. As detention time is shortened, effective disinfection can be maintained by use of higher chlorine application rates. There are limitations to this process that include the need for extremely good initial mixing of chlorine into the flow stream and a minimum detention time.

High-energy mixing using chemical induction mixers and sufficient chlorine dosing should preclude the need for construction of additional contact time through the 2025 planning period. The minimum detention time with high rate disinfection is about 10 minutes since chemical disinfection is both a physical and biological process that requires time to complete. The initial screening is summarized in Figure 3-8.

	Evaluation criteria								
Technology/Description	Relative present worth cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	O&M	Safety	Viability at Newberg WWTP	Total score
Alternative 1: High-rate disinfection	+	0	0	+	+	0	0	Yes	3
Alternative 2: Additional contact basin	0	0	0	0	+	+	0	Yes	2
Alternative 3: Additive of onsite generation of HOCl	-	0	0	0	+	0	0	Yes	0
Alternative 4: UV disinfection	-	-	0	0	+	0	+	Yes	0

Note: A higher score is better.

Figure 3-8. Initial Screening for Disinfection Alternatives

Effective disinfection beyond the 2025 flow rates will require that additional contact basin capacity be constructed.

Both Alternatives 1 and 2 are assumed to include the use of chemical induction mixers, as they reduce the amount of overall chemical needed. The City of Bend, Oregon, reported a 20 percent reduction in HOCl use when induction mixers were installed. One induction mixer is required in each of the two contact basin sides.

Alternative 2. If the high-rate disinfection concept is approved and implemented, construction of additional contact basin capacity can be deferred until after 2025 depending on the growth of flow rates. If chlorine or HOCl disinfection is used after the year 2025, a new contact basin would be required equal in size to one of the two existing contact basins. This decision can be deferred until that time, unless DEQ does not accept the high-rate disinfection concept. It is noted that DEQ approved a very similar high-rate disinfection process at Gresham, Oregon, that has been successful.

Alternative 3. This alternative is applicable to Alternatives 1 and 2 and specific details are discussed below. Many water and wastewater treatment plants have converted from chlorine gas disinfection to HOCl disinfection. The reasons relate to community safety and security, rather than cost or ease of operation. Chlorine gas leak detection and response plans are required to be in place at all larger facilities. The potential for sabotage or vandalism resulting in a chlorine gas leak is also a primary security concern wherever chlorine gas is used. For this reason, the conversion of the disinfection process to either HOCl or UV disinfection is recommended.

HOCl can be purchased for bulk delivery by chemical suppliers, or it can be generated onsite. The existing Newberg WTP uses HOCl generated onsite. There is significant benefit in coordination of equipment in terms of spare parts, training, and redundancy when the WTP moves in closer proximity to the WWTP.

Alternative 4. This alternative involves replacement of chemical disinfection and dechlorination with the use of UV light. UV disinfection is successfully used at a great number of WWTPs in Oregon and throughout the country. Specific UV lamp technologies and design configurations are discussed below. The emphasis is on fitting a UV disinfection system within the existing chlorine contact basins to reduce the plant site area required and to reuse existing tankage.

The benefits of high-rate mixing are shown in Figure 3-8. Initial screening is summarized in Figure 3-9.

3.10.4 Evaluation of Viable Alternatives

All of the disinfection alternatives identified remain viable. The first two alternatives could use either chlorine gas or HOCl. These are described in more detail below.

- Alternative 1: Bulk delivered chlorine gas
- Alternative 2: Bulk-delivered HOCl
- Alternative 3: Onsite HOCl generation
- Alternative 4: UV inactivation with hypochlorite

Table 3-16 contains a comparison of disinfection alternatives.

Table 3-16. Comparison of Disinfection Alternatives

Alternative	Safety	Site requirements	Water quality	Ease of operation
1. Bulk delivered chlorine gas	Most risk	Medium	More THM[1]	Status quo
2. Bulk delivered hypochlorite	Most chemical	Highest	More THM	Simplest one system
3. Onsite generation of hypochlorite	Lower chemical, higher fire	High	More THM	More mechanical
4. UV inactivation with hypochlorite for residual	Lowest chemical	Least	Least THM	Two systems mechanical. May not be feasible with iron discharge from Water treatment plant).

[1]THM = trihalomethanes

3.10.5 Bulk Delivered HOCl

The potential hazards and costs associated with handling and use of gas chlorine has led many utilities to convert to the use of liquid HOCl. HOCl is available in 12 percent solution in most locations. This solution is about 2-1/2 times as concentrated as household bleach. It is chemically aggressive, and should be regarded as an exposure hazard for operators and others in the vicinity of storage, transport, or application points. HOCl solution has a pH of over 11 and should not be acidified since it will release a concentrated cloud of chlorine gas.

HOCl solution should be stored in double-contained tanks or a secondary containment basin. Piping should be double-contained. Spill containment in unloading areas is also required.

HOCl decomposes during storage with the rate of decomposition impacted by concentration, temperature, pH, light, and the presence of metallic contaminants in the solution. In general, 12 percent HOCl should be stored for no more than 30 to 60 days. The solution strength will decrease by 20 percent over 30 days at 80 degrees Fahrenheit (F).

The cost of HOCl solution is substantially higher than chlorine gas per pound of equivalent chlorine. HOCl solution (12 percent) may range from $0.40 to $1.25 per pound of equivalent chlorine depending upon the size and location of the treatment plant.

The advantages of HOCl for disinfection include:

- Operators are familiar with storage, pumping, and regulating liquid solutions.
- There is no risk to the general public.
- It provides chlorine residual for reclaimed water.
- No capital improvements are required after current reuse project is complete.

The disadvantages of delivered HOCl for disinfection include:

- Solution strength deteriorates with time.
- Dechlorination is required prior to discharge.

HOCl/Onsite Generated. Onsite generation of HOCl solution has become more widely used in recent years as an alternative to bulk delivery. HOCl is generated onsite electrolytically from brine (sodium chloride in water solution). Hydrogen gas is the only byproduct of electrolytic decomposition of sodium chloride and must be dispersed to the atmosphere. Equipment is available to generate HOCl in various concentrations from 0.8 percent up to 12 percent. Low strength HOCl solution is less hazardous to use and can be stored for longer periods of time. The number and size of the required storage tanks increases as the solution strength decreases.

The advantages of onsite generation of HOCl include:

- Safety concerns related to transport and off-loading of bulk HOCl solution are eliminated.
- Lower strength HOCl solutions are less hazardous to operators.
- Lower strength HOCl solutions degrade at a much slower rate than 12 percent solution.

The disadvantages of onsite HOCl generation include:

- It is a newer technology that may require more supervision.
- It requires dechlorination prior to discharge to the receiving stream.
- Production costs depend upon the cost of power.
- Water used in process must be softened to prevent scaling.
- Electrodes are consumed and require periodic replacement.

UV Disinfection. UV light is generated by germicidal lamps that contain mercury vapor in an inert gas, such as argon, with tungsten electrodes at each end of the lamp. UV light is emitted when the lamp is powered and heated to vaporize the mercury. Power to each lamp is regulated by ballast, which controls electrical frequency and voltage.

The UV light penetrates the bacterial cells which consist of a cell wall and cell nucleus containing genetic material composed of deoxyribonucleic acid or DNA. Microbial inactivation occurs due to the impact of UV light on the cell's DNA.

UV light at 254 nanometers (nm) (or within the range from 220 to 260 nm) has the ability to penetrate the cell wall and disrupt the chemical double bonds of the DNA molecule. The DNA molecule is a double strand of nucleotides. The inactivation mechanism for UV light is dimerization of the thymine double bonds. If a sufficiently high number of thymine double bonds are disrupted, the microbe is unable to replicate and is no longer measurable in standard test methods, or infective in the environment.

There are a number of UV lamp technologies applicable for wastewater disinfection. The available lamp technologies are discussed below. These include conventional low-pressure lamps, low-pressure/high-output lamps, medium-pressure lamps, and pulsed xenon lamps.

The amount of UV light energy applied to water flowing through a UV reactor is called the dose rate, and is expressed in units of μW·s/cm2, mW·s/cm2, or mJ/cm2. Figure 3-9 shows the factors which influence the calculation of UV dose rate for a specific water sample.

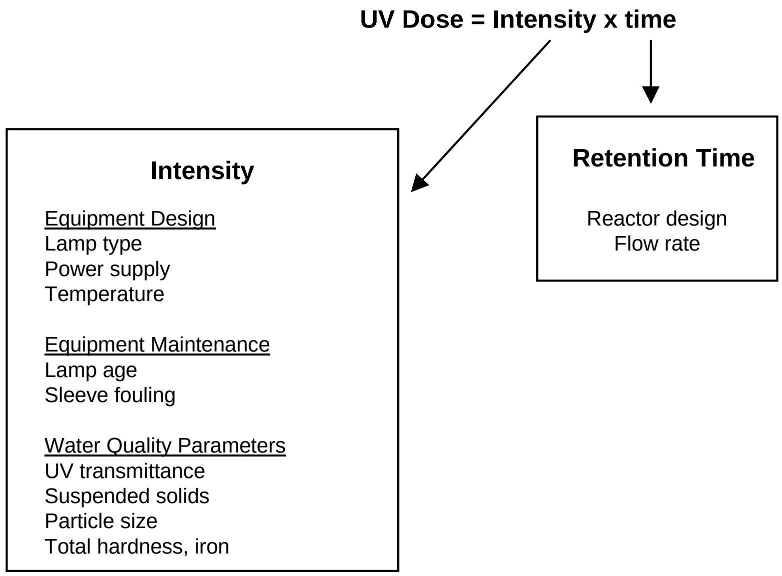


Figure 3-9. Factors Influencing UV Dose Rate

UV dose rate is defined as the product of UV intensity and time, although its actual calculation is far more complicated due to real-life factors. UV dose varies throughout a UV reactor as a function of equipment design, equipment maintenance, water quality and reactor hydraulics. The average UV dose is determined by a mathematical model based on the multiple point source summation approximation method. The actual applied UV dose will always be less than the calculated UV dose due to imperfect mixing within the UV reactor. Equipment manufacturers use hydraulic model testing and computational fluid dynamic analysis to determine the actual mixing characteristics of a specific reactor design. Confirmation of UV dose is also difficult to measure since the UV reactor is a dynamic system. The accepted calibration method is biodosimetry, in which a surrogate microbe (i.e., bacillus subtulis or MS2 coliphage) is used to prepare a dose-response curve for each UV system.

3.10.6 Comparative Cost Estimates

Order-of-magnitude costs for the disinfection improvements are summarized in Table 3-17.

Table 3-17. Costs for the Disinfection Improvements

Description	Capital cost, dollars	O&M cost, dollars	Total present worth, dollars
Alternative 1A+5B: high-rate disinfection/chlorine/dechlorination	400,000	221,000	3,397,000
Alternative 1B+5B: high-rate disinfection/hypochlorite/dechlorination	350,000	292,000	4,322,000
Alternative 3A+5A: high-rate disinfection/onsite generation/dechlorination	1,545,000	306,000	5,697,000
Alternative 2A+5B: conventional disinfection/chlorine/dechlorination	1,145,000	69,000	2,088,000
Alternative 2B+5B: conventional disinfection/hypochlorite/dechlorination	1,085,000	998,000	2,411,000
Alternative 3B+5B: conventional disinfection/onsite generation/dechlorination	1,545,000	445,000	7,586,000
Alternative 4: UV disinfection	3,065,000	64,000	3,940,000

Note: net present value includes O&M for 20 years to 2025.

Based on the results of this analysis and the Liquids Solids Workshop No. 2, high-rate disinfection is recommended for the immediate improvements for the first 5 years with either sodium hypochlorite or UV disinfection phased in for incremental expansion when needed in the future. However, UV feasibility would need to be verified with effluent testing by the UV suppliers to see if adequate disinfection could be obtained with iron from the water treatment plant, if it continues to be discharged to the plant.

3.11 DECHLORINATION

The dechlorination system was added in 1998. Prior to 1998 there was no requirement for dechlorination. DEQ has mandated that the chlorine residual not exceed 0.05 milligrams per liter (mg/L). The tank holds around 300 gallons of bisulfate and there are two pumps that feed the bisulfate into a box at the end of the chlorine contact basin. One grab sample per day is taken to measure chlorine residual.

3.11.1 Design Criteria

The PHFs for the design conditions are summarized in Table 3-18. The NPDES permit includes a requirement that total residual chlorine shall not exceed a monthly average concentration of 0.02 mg/L and a daily maximum of 0.05 mg/L. The NPDES permit Note No. 4 regards daily maximum levels lower than 0.10 mg/L chlorine residual in compliance.

Table 3-18. PHF Projections from 2005 to 2040

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
PHF[1] (mgd)	20.81	23.65	27.15	31.19	34.77	45.86

[1]Projected PHFs may decrease after inflow is removed from the collection system.

3.11.2 Summary of Existing Deficiencies

The bisulfite is not fully usable. The bisulfite tank is only 300 gallons and it has to be refilled a minimum of once a week and frequently twice a week during periods of high flows. Each refill is about 150 gallons, because when the tank is down to about one-third capacity, the chemical feed pumps begin to lose pressure and are unable to feed the bisulfate to the system. The tank needs to be raised. Another concern is that there is a lot of buildup of bisulfite crystals around the fittings. These crystals are causing further operational problems. Existing bisulfite feed water is from the potable system.

3.11.3 Identification of Expansion Alternatives

Design recommendations are provided for immediate improvements for sodium bisulfite feed system, including more advanced control system, updated and larger pumps and raising the tank for increased capacity. No feasible alternatives were identified.

3.12 OUTFALL

The Newberg outfall discharges to the Willamette River near River Mile 49.7. The outfall is a 24-inch-diameter open pipe extending approximately 30 feet offshore at a depth of 17 feet at ordinary low water. The outfall is parallel to a 24-inch outfall from the SP Newsprint Corporation. The two outfalls are at different depths and interact only at a marginal level.

Review of outfall analysis information indicates that the mixing provided within the current mixing zone is adequate to meet applicable water quality standards provided that effective dechlorination is provided and trace contaminants, particularly heavy metals such as mercury, are controlled by the enforcement of local limits regulations. Mercury needs to be controlled to meet Willamette River mass loading (TMDL) requirements. Since total residual chlorine is adequate by dechlorination and trace contaminants will be controlled by enforcement of local limits regulations, the existing outfall should be adequate until 2025. The mixing zone is an area that extends 75 feet out from the west bank of the river and from 15 feet upstream to 150 feet downstream to the point of discharge.

Any reduction in the size of the regulatory mixing zone will require reevaluation of the mixing provided by the currently permitted zone and would potentially result in the need for a more effective in-stream diffuser. The City initiated a new mixing zone evaluation in June 2009 in accordance with the DEQ Regulatory Mixing Zone Internal Management Directive guidance.

3.12.1 Design Criteria

The outfall is sized to accommodate the PHF requirements. The PHF for the design conditions are summarized in Table 3-19.

Table 3-19. PHF Projections from 2005 to 2040

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
PHF[1] (mgd)	20.81	23.65	27.15	31.19	34.77	45.86

[1]Projected PHFs may decrease after inflow is removed from the collection system. Peak effluent flows may be attenuated through the plant.

3.12.2 Summary of Existing Deficiencies

A simple hydraulic analysis of the Newberg WWTP shows that the outfall should be capable of handling the PHFs through the year 2025 when the PHF is expected to reach 35 mgd. Based on drawings prepared by KCM, the high water level in the Willamette River was assumed to be 83 feet. If the water level exceeds 83 feet, the plant would need to add pumping in order to discharge the PHF through the pipe.

The outfall is constructed for most of its length with 36-inch-diameter pipe at fairly gradual slopes and at some points uses surcharging by relying on the upstream head to push the water up the gradient. When the outfall is approximately 300 feet from the river, the size changes to 24-inch-diameter pipe and the slope becomes very steep. At one section the slope is 21 percent. Where the pipe transitions to 24 inches, there is potential for a hydraulic cannon.

The velocity of water in the pipe at PHFs may exceed 27 fps in the steep 24-inch section, and as the pipe will not be flowing full, there may be a strong likelihood for some hydraulic cannon effects to occur in the pipe. This happens when there is a hydraulic jump in the pipe, and the flow goes from super-critical to sub-critical, trapping air in the pipe. The air becomes pressurized and once it reaches a critical state, it will exit the pipe forcefully. This force can blow off manhole lids, separate manhole risers, and cause other problems with the outfall. The preliminary calculations show that the City is having problems with the outfall when flows reach 25 to 27 mgd. Plant staff verified that the effects of a hydraulic cannon are evident at the upstream manhole of the outfall pipe. Upon investigation of the manhole and outfall at the conclusion of Liquids Solids Workshop No.2, a hydraulic cannon event was actually witnessed.

The mixing zone study initiated in June 2009 will need to be completed to determine if other outfall modifications are required.

3.12.3 Identification of Expansion Alternatives

A viable alternative includes adding a pipe along that length to increase capacity. This can prevent the air entrapment and alleviate the possible hydraulic effects.

3.12.4 Evaluation of Viable Alternatives

No other viable alternatives were identified.

3.13 TERTIARY TREATMENT/REUSE

This section discusses tertiary treatment and reuse.

3.13.1 Irrigation

Tertiary treatment using membranes has been selected by the City after a predesign evaluation and recommendation documented in the *Reuse Water System Predesign Study* (CH2M HILL, October 2005). Membranes are assumed to be the preferred technology for future tertiary treatment.

3.13.2 In-plant Water

The current reclaimed water system filters are inadequate and the screening size is too large to be effective. A looped plant water system is recommended for inclusion in the CIP. Plant water is not currently available at the headworks and will be added to conserve potable water.

3.13.3 Temperature of Discharge and Reuse Requirements

The Willamette Total Maximum Daily Load Waste Load Allocation (WLA) for Newberg has been included in the City's NPDES permit by permit modification. The permit modification letter is included in Appendix A. The City has the opportunity to track river and effluent temperature and flow on a 7-day running average or comply based on the permitted WLA discussed in Chapter 1. The discharge limits are based on a 7-day average. Figure 3-10 summarizes the Excess Thermal Load (ETL) calculations for various river and effluent conditions. The calculations are based on a 6000 cfs for the 7Q10 river flow. Various effluent flows were used in the spreadsheet with effluent temperatures ranging from 21to 25 C, until the spreadsheet showed a violation, then the effluent flow was reduced by 0.1 mgd to show the maximum effluent flow that will not violate under each scenario. These calculated allowable 7-day average effluent flow values are shown in red. These results were checked against the DEQ spreadsheet used for reporting purposes and the results are consistent.

The bottom block is the worst case scenario when there is no river data. The block of calculations above that is a worst case scenario in terms of river temperature. The City has not recorded a two degree difference between the daily average and daily maximum river temps, historically but it may be possible, so it is included as worst case.

Even under the worst case scenario, the City will be able to discharge 3.5 mgd at 23°C, which was the highest 7-day average daily maximum temperature recorded in 2008. (In 2007, the highest single day maximum temperature was 23°C.) In most cases, the City will be able to discharge 4.0 mgd at 23°C or 3.0 mgd and 24°C. The City has implemented a reuse program to irrigate local golf courses that will decrease its effluent discharge by 1 mgd during the irrigation season. As the City grows, the City will meet the WLAs with increased irrigation reuse.

River 7-day Avg Q cfs	Daily Max Temp *C	Daily Avg Temp *C	Trm_n	Tra_n	a	Effluent 7-day Avg Max Temp *C	Effluent 7-day Avg Q mgd	ETL M-Kcals	ETL Limit Option A M-Kcals	ETL Limit Option B M-Kcals	ETL Limit Option C M-Kcals
6000	**20.0**	**19.5**	19.43	18.54	0	**21.0**	**12.0**	**45**	40	42	**45**
6000	**20.0**	**19.5**	19.43	18.54	0	**22.0**	**6.0**	**45**	40	42	**45**
6000	**20.0**	**19.5**	19.43	18.54	0	**23.0**	**4.0**	**45**	40	42	**45**
6000	**20.0**	**19.5**	19.43	18.54	0	**24.0**	**3.0**	**45**	40	42	**45**
6000	**20.0**	**19.5**	19.43	18.54	0	**25.0**	**2.4**	**45**	40	42	**45**
6000	**20.5**	**20.0**	19.93	19.01	0	**21.0**	**12.0**	**45**	40	42	**45**
6000	**20.5**	**20.0**	19.93	19.01	0	**22.0**	**6.0**	**45**	40	42	**45**
6000	**20.5**	**20.0**	19.93	19.01	0	**23.0**	**4.0**	**45**	40	42	**45**
6000	**20.5**	**20.0**	19.93	19.01	0	**24.0**	**3.0**	**45**	40	42	**45**
6000	**20.5**	**20.0**	19.93	19.01	0	**25.0**	**2.4**	**45**	40	42	**45**
6000	**21.0**	**20.5**	20.43	19.48	0.025795	**21.0**	**11.7**	**44**	40	42	**44**
6000	**21.0**	**20.5**	20.43	19.48	0.025795	**22.0**	**5.8**	**44**	40	42	**44**
6000	**21.0**	**20.5**	20.43	19.48	0.025795	**23.0**	**3.9**	**44**	40	42	**44**
6000	**21.0**	**20.5**	20.43	19.48	0.025795	**24.0**	**2.9**	**44**	40	42	**44**
6000	**21.0**	**20.5**	20.43	19.48	0.025795	**25.0**	**2.3**	**44**	40	42	**44**
6000	**21.5**	**21.0**	20.93	19.95	0.00229	**21.0**	**11.9**	**45**	40	42	**45**
6000	**21.5**	**21.0**	20.93	19.95	0.00229	**22.0**	**5.9**	**45**	40	42	**45**
6000	**21.5**	**21.0**	20.93	19.95	0.00229	**23.0**	**3.9**	**44**	40	42	**45**
6000	**21.5**	**21.0**	20.93	19.95	0.00229	**24.0**	**2.9**	**44**	40	42	**45**
6000	**21.5**	**21.0**	20.93	19.95	0.00229	**25.0**	**2.3**	**44**	40	42	**45**
6000	**22.0**	**21.5**	21.43	20.42	0	**21.0**	**12.0**	**45**	40	42	**45**
6000	**22.0**	**21.5**	21.43	20.42	0	**22.0**	**6.0**	**45**	40	42	**45**
6000	**22.0**	**21.5**	21.43	20.42	0	**23.0**	**4.0**	**45**	40	42	**45**
6000	**22.0**	**21.5**	21.43	20.42	0	**24.0**	**3.0**	**45**	40	42	**45**
6000	**22.0**	**21.5**	21.43	20.42	0	**25.0**	**2.4**	**45**	40	42	**45**
6000	**22.5**	**22.0**	21.93	20.89	0	**21.0**	**12.0**	**45**	40	42	**45**
6000	**22.5**	**22.0**	21.93	20.89	0	**22.0**	**6.0**	**45**	40	42	**45**
6000	**22.5**	**22.0**	21.93	20.89	0	**23.0**	**4.0**	**45**	40	42	**45**
6000	**22.5**	**22.0**	21.93	20.89	0	**24.0**	**3.0**	**45**	40	42	**45**
6000	**22.5**	**22.0**	21.93	20.89	0	**25.0**	**2.4**	**45**	40	42	**45**
6000	**20.6**	**18.6**	20.03	17.70	0.115114	**21.0**	**11.2**	**42**	40	**42**	41
6000	**20.6**	**18.6**	20.03	17.70	0.115114	**22.0**	**5.6**	**42**	40	**42**	41
6000	**20.6**	**18.6**	20.03	17.70	0.115114	**23.0**	**3.7**	**42**	40	**42**	41
6000	**20.6**	**18.6**	20.03	17.70	0.115114	**24.0**	**2.8**	**42**	40	**42**	41
6000	**20.6**	**18.6**	20.03	17.70	0.115114	**25.0**	**2.2**	**42**	40	**42**	41
NO RIVER DATA						**21.0**	**10.7**	**40**	**40**		
						22.0	**5.3**	**40**	**40**		
						23.0	**3.5**	**40**	**40**		
						24.0	**2.6**	**39**	**40**		
						25.0	**2.1**	**40**	**40**		

Figure 3-10. Newberg Temperature Excess Thermal Load Calculations

3.13.4 Design Criteria

Table 3-20 lists the projected average daily flows and peak week summer flows based on a peaking factor using 2005 average dry weather flow (ADWF) and peak summer week flow of 1.98.

Table 3-20. Projected Average Daily and Maximum Weekly Flows

Year	2005	2010	2015	2020	2025	2040
Population	21,132	24,497	28,712	33,683	38,352	54,097
ADWF (mgd)	2.07	2.40	2.81	3.30	3.76	5.30
Peak summer week flow (mgd)	4.1	4.75	5.56	6.53	7.45	10.49

Note: peak summer week based on ratio of ADWF to peak summer week in 2005.

3.13.5 Summary of Deficiencies

The temperature data provided showed a correlation between the higher temperatures and the lower flow rates, and every data point that showed a temperature exceeding 23 degrees C was equal or lower to the average daily flow. Based on the knowledge that 3.0 mgd of flow at 23.5 degrees C may cause the City to violate its temperature and that the temperature of the effluent will not likely reach a concern until the spring rains have run and the flow rates are down, the City can probably assume that the when the ADWF reaches 3.0 mgd there will be concerns about the temperature limits in the NPDES permit. Based on the information in Table 3-20, the ADWF will not reach 3.0 mgd until the 2015-2020 timeframe; however, the peak week currently could cause some concern. Storage for the peak week flows in excess of 3.0 mgd. Storage area requirements need to be coordinated with reuse project by others. The golf course that will use the treated water for irrigation has 3 million gallons of storage capacity.

3.13.6 Identification of Alternatives

A temperature management plan may be required in the future to show how the City will maintain compliance with the temperature TMDL. The City currently meets the TMDL requirements, so a plan is not included with the temperature allocations in the open permit. Options to maintain compliance include but are not limited to:

- Increasing reuse and storage for peak flows
- Storing effluent using a combination of night-time discharge when the ambient air and effluent is cooler
- Cooling the effluent prior to discharge through subsurface discharge to the hyporheic zone
- Implementing best management practices at the WWTP to minimize heating across the treatment processes
- Treating effluent using other methods such as wetlands mitigation
- Temperature trading

The City plans to add additional reuse to address the temperature WLA.

3.14 IN-PLANT DRAINS

Stormwater generated onsite is conveyed by gravity to the IPS along with recycle streams. In-plant stormwater handling alternatives were studied and documented in the *Final Dump Station/Headworks Studies* report.

3.14.1 Design Criteria

Plant staff provided a value of 60,000 gallons of runoff per 1 inch of rain as a design guideline. This value was based upon observation. Applying this amount to a 5-year rainfall amount of 3.10 inches resulted in a total volume of 186,000 gallons. This volume was applied over a 24-hour period, which resulted in a peak runoff rate of 510 gallons per minute (gpm). The 50,000 gallons per day of recycle water was converted to gpm, assuming 6 hours of flow per day. This rate totaled 140 gpm over a 6-hour period. The analysis also included an allowance for pumping 50,000 gpd of recycled water from plant operations. This rate was added to the peak stormwater runoff rate to arrive at a peak flow rate of 650 gpm.

3.14.2 Summary of Existing Deficiencies

Using the IPS to pump in-plant stormwater results in unnecessary pumping costs due to the 90 feet of static head that the IPS pumps against. Having an independent in-plant pump station was found to be more cost-effective.

3.14.3 Identification of Expansion Alternative

Alternatives were investigated to intercept the stormwater flow at a higher elevation than the IPS and pump it directly to the headworks to save on pump operating costs. The results were documented in the *Final Dump Station/Headworks Studies* report. The conceptual design for the stormwater/recycle pumping station assumes two submersible pumps, each pumping at a rate of 325 gpm. The pumps would be installed in a circular wet well consisting of precast manhole sections. The pump station would be located in the vicinity of Stormwater Manhole No. 1 and would be connected to the stormwater and recycle water systems through new gravity sewers. Flow would be conveyed from the new station to the headworks via approximately 320 feet of 6-inch DI pipe. An overflow pipe would allow flows exceeding the capacity of the pumping station to re-enter the existing stormwater system via a new manhole constructed over the stormwater pipe discharging to the IPS.

Based on the results of Liquids Solids Workshop No. 2, it was recommended that a larger pump station be included so that it can handle the future plant site expansion stormwater runoff.

3.15 EMERGENCY GENERATOR

The emergency generator needs were established as part of the Administration Building Predesign Report. The emergency generator is being implemented by others.

3.16 SOLIDS ALTERNATIVES EVALUATION

Sludge is removed from the secondary clarifiers and pumped to sludge holding tanks using the waste activated sludge (WAS) pumps prior to being dewatered by belt filter press. The WAS pumps have enough capacity to keep up with solids loading from the existing clarifiers. Additional pumping will be needed with additional clarifiers.

The plant has two belt filter presses (BFPs) on site that dewater the sludge before it is sent to the composting facility. Each BFP is capable of producing 30.6 tons per day of dewatered sludge at about 16 percent dry solids concentration. Approximately 600 dry tons of waste solids are generated on an annual basis. Annual solids quantity will more than double in 2040 to approximately 1,500 dry tons. Improved dewatering would increase the capacity of the compost system by reducing the amount of bulking agent required. Bulking agent requirements are relatively high at present to offset the low solids concentration in dewatered cake and provide blended feedstock at a minimum of 40 percent solids. Dewatering capacity will need to be expanded for growth as well.

Alternatively, procurement of a sawdust drying system is being considered to reduce compost feedstock moisture content and effectively increase compost system capacity. Partial funding may be available to offset capital cost, making the dryer an immediate priority. In this case, the upgrade for solids dewatering could be postponed until a later date.

3.16.1 Design Criteria

The capacity of the tunnel reactors varies with the feedstocks used and the initial mix moisture content required for mixer and tunnel operation. The current capacity is estimated to vary seasonally between 1.1 and 1.4 dry tons biosolids per calendar day. Additional capacity is provided through aerated curing after feedstock has passed through the tunnels to ensure that time and temperature requirements are met for Class A biosolids product. The system was essentially at capacity when the analysis was first completed. Since that time, the sawdust feed delivery is limited to a wetter product, thus reducing capacity even more. Plant staff have resorted to stockpiling feed solids and implementing a small version of static pile composting.

3.16.2 Summary of Existing Deficiencies

The compost process is at or near capacity. The existing system has been upgraded recently by adding aerated curing bays and additional improvements, and it can continue to meet baseline needs. Upgrading the dewatering would provide an immediate increase in capacity for solids, improve operational performance, and reduce required bulking agent cost, and material (recycle) handling. Drying bulking agent would also relieve existing compost capacity limitations.

Processes to meet expansion needs also need to meet the goal of producing a Class A biosolids product. Technologies that have been considered include high temperature digestion, lime pasteurization, composting (aerated static pile [ASP] process), and thermal drying. Based on operating experience, the existing tunnel reactor system should not be expanded to meet additional capacity needs. In addition, a private entity has proposed receiving dewatered solids at a regional energy recovery facility that will tentatively be located at the S.P. Newsprint mill.

3.16.2 Alternatives for Increasing Capacity of Existing System

Several alternatives have potential for increasing the capacity of the composting facility:

Alternative 1. Upgrade the dewatering system to produce a drier cake.

Alternative 2. Use the curing area aeration system to dry the product and recycle the drier material back to the tunnel.

Alternative 3. Use commercial drying equipment to dry a portion of the sludge, recycle stream or sawdust supply prior to composting.

Alternative 4. Dry sawdust bulking agent to reduce compost feedstock moisture content and increase system capacity.

Transferring the regulatory compliance step for vector attraction reduction from the tunnels to the curing area has already been implemented on a partial basis to address reduced detention in the tunnels. The capacity of the curing area to serve this purpose is not adequate to address growth but is meeting short-term needs.

3.16.3 Identification of Expansion Alternatives

The proven biosolids technologies identified as potentially viable for adding capacity for Class A biosolids production at the existing Newberg WWTP are:

- Thermal drying
- Lime pasteurization
- Temperature-phased anaerobic digestion (TPAD)
- Composting
- Energy recovery (offsite)

These technologies are summarized in Table 3-21.

Table 3-21. Class A Biosolids Technologies Identified as Potentially Viable

Technology	Description	Advantages	Disadvantages
Thermal drying	Heat is applied to the solids to evaporate the water and reduce the mass of solids.	■ Process is volume reducing ■ Product is acceptable to the public and suitable for a variety of uses	■ Requires relatively high energy ■ Requires stabilization prior to drying to minimize odorous product
Lime pasteurization	Undigested biosolids are dewatered and dosed with lime prior to being processed in a pasteurization reactor.	■ Product has lime value which may help marketability ■ Product is moderately acceptable to the public	■ Uses a proprietary system process ■ Requires product that is not highly marketable as opposed to other Class A biosolids ■ Requires additional electrical heat required to meet pathogen reduction requirements ■ Has product odor potential ■ Has quantity increase from lime addition
TPAD	A two-stage anaerobic stabilization process. The first stage is accomplished in the absence of oxygen and at temperatures of 130 to 140 degrees F. The second stage is accomplished in the absence of oxygen and at temperatures of 95 to 100 degrees F.	■ Reduces solids quantity and produces biogas	■ Expensive and perhaps impractical for small-to-medium facilities ■ Least marketable of Class A products and presents product storage problems
Composting	Self-heating, aerobic biochemical process used to stabilize biosolids.	■ Process can be implemented using ASPs, windrows, or in-vessel systems ■ Product is highly acceptable to the public ■ Product market already exists ■ Process has operational familiarity	■ Requires a bulking agent to increase porosity of the biosolids and reduce moisture content ■ Requires significant land area ■ Is labor-intensive ■ Has odor potential with incorrect operation ■ Has quantity increase from bulking
Energy recovery offsite	Private entity receives dewatered biosolids for thermal drying followed by gasification.	■ Operational requirements are minimal ■ Process is cost-effective	■ Feasibility not confirmed ■ Long-term reliability not confirmed

3.16.4 Evaluation of Viable Alternatives

For the Newberg WWTP, the most viable options for accommodating future growth are composting, thermal drying, and off-site energy recovery. TPAD would require a substantial investment, and the process would not result in a highly marketable Class A product. Experience on other Brown and Caldwell projects has shown that for small-to-medium facilities, initiating anaerobic digestion and TPAD is much more expensive than process technologies that do not require digestion. Lime pasteurization would be cost-competitive with other technologies but generates a product that is less compatible with existing markets. Lime pasteurization works best at plants with primary sludge in which a high percent solids cake can be produced by the dewatering equipment. Dewatered biological sludge has resulted in a mud-like product from lime pasteurization which is difficult to manage and market. Composting and thermal drying can be operated around this limitation. Figure 3-11 shows the rankings of the five options.

Technology	Cost	Regulatory compliance	Storage needs	Flexibility	Reliability	Operability	Safety	Odor potential	Viability at Newberg WWTP	Total score
Thermal drying	-	+	+	+	+	+	-	+	Yes	5
Lime pasteurization	+	+	-	-	0	0	0	-	No	0
TPAD	-	+	-	-	0	0	+	0	No	--1
Composting	-	+	-	0	+	+	+	+	Yes	3
Energy recovery	+	+	+	+	-	+	+	+	Yes	6

Note: A higher score is better.

Figure 3-11. Biosolids Technology Rankings

Based on the scores in Figure 3-11, the three most feasible options—energy recovery, thermal drying, and composting—are evaluated in more detail. The cost evaluation is summarized in the following section.

Each of these technologies would be complemented by a dewatering upgrade. Even if solids are simply trucked offsite, a higher percent solids reduces material handling, hauling, and tipping fees. Applicable technologies in addition to the belt filter press include centrifuge, screw press (FKC), and rotary press (Fournier). From experience, we know that the screw press and rotary press work best on solids when primary sludge is in the mix. For biological or extended aeration sludge, the centrifuge will generate a consistently higher solids concentration than the BFP. Where the existing belt presses produce approximately 16 percent solids, we expect the centrifuge will produce 20 percent. Therefore, we recommend centrifuge dewatering as the most proven means of increasing solids concentration for the Newberg WWTP.

3.16.5 Comparative Cost Estimates

Order-of-magnitude costs for the solids improvements are summarized in Table 3-22. Costs for dewatering are based on providing capacity for 100 percent of solids generated while costs for Class A biosolids processes are only intended to accommodate future growth.

Table 3-22. Projected Costs for the Solids Processing Improvements

Description	Capital cost, dollars	O&M cost, dollars	Total present worth, dollars
Sawdust drying	500,000	TBD[1]	TBD[1]
Centrifuge dewatering	2,475,000	123,000	4,142,000
Class A process Alternative 1 (ASP composting)	3,284,000	178,000	5,705,000
Class A process Alternative 2 (thermal drying)	3,182,000	206,000	5,979,000
Class A process Alternative 3 (off-site energy recovery)	1,076,000	172,000	3,415,000

[1] Operating costs will be offset by energy and material (bulking agent) savings in the compost process, and labor savings for reduced materials handling.

Centrifuge dewatering is considered a fundamental improvement for the plant that will benefit existing and future process technologies. Upgraded dewatering will improve performance of the existing compost system and increase effective capacity by approximately 25 percent by increasing dewatered solids concentration. Higher solids content requires less bulking agent and reduces materials handling requirements. Based on the results of this analysis and the Liquids Solids Workshop No. 2, centrifuge dewatering improvements were recommended.

Subsequently, as an immediate improvement to increase compost system capacity, the sawdust dryer equipment procurement and installation is being implemented in 2009. Capital costs are substantially lower than mechanical dewatering, and operation can provide the maximum immediate benefit in terms of compost system capacity. The implementation of the dehydrator will provide the sufficient capacity that centrifuge dewatering is no longer needed. Operating experience with the dehydrator will determine how much capacity is realized and what other options should be considered.

For the alternative Class A process technologies, composting and thermal drying are nearly equal in cost, while off-site energy recovery is much less. Costs for energy recovery need to be confirmed. An evaluation of Class A process upgrades is presented in Figure 3-12.

Technology/Description	Evaluation criteria								Total score
	Relative Present Worth Cost	Energy conservation	Regulatory compliance	Flexibility	Reliability	Operation and Maintenance	Safety	Viability at Newberg WWTP	
Alternative 1 (ASP compost)	2	3	5	4	5	2	4	4	29
Alternative 2 (Thermal drying)	1	1	5	5	4	3	2	4	25
Alternative 3 (Off-site energy recovery)	5	5	5	5	1	5	5	3	34

Note: A higher score is better.

Figure 3-12. Class A Biosolids Technology Rankings

This analysis integrates the more detailed cost estimate with non-cost factors. Results favor offsite energy recovery. Plant staff have indicated that a back-up strategy using a simplified composting technology is desired until the offsite energy recovery has proven reliable for long-term service.

3.17 STAFFING

City water and wastewater utility operation and maintenance (O&M) responsibilities are handled by the Public Works Department Operations Division. Several staff members have assigned duties for both the WTP and the WWTP; others are predominantly performing daily activities in water or wastewater-related facilities. An assessment was made of specific duties identified in time sheets for December 2006 and April 2007. The purpose was to compare workloads during typical wet months and dry months. This information is summarized in Appendix F. Current staffing levels are documented to determine the existing deficiencies and deficiencies with the proposed WWTP improvements and upgrades

The City has 12 full-time employees plus one part-time administrative person (0.65 full-time equivalent) and a temporary labor man-hour pool of up to 2,000 hours annually for an hourly day laborer. The positions are described in Table 3-23.

Table 3-23. Current Positions and Responsibilities

Newberg Public Works Department–Operations Division Staffing		
Positions	No.	Responsibilities
Operations Superintendent	1	Supervise and manage WTP, WWTP, pump stations, wells, and springs
Environmental Supervisor	1	Supervise environmental activities (laboratory, pretreatment program, etc.)
Water Supervisor	1	Supervise WTP, wells, reservoirs, springs, and distribution water sampling
Wastewater Supervisor	1	Supervise wastewater operation including the WWTP and major pump stations
Wastewater Maintenance Sr. Mechanic	1	Maintain wastewater system facilities
Water Operations Sr. Operator and Operator I	2	Provide WTP operation and sampling
Water/Wastewater Maintenance Plant Mechanic	1	Provide maintenance—25 percent to WTP, 75 to percent wastewater system
Wastewater Operations Two Operator II and one Operator I	3	Operate WWTP liquids and solids systems
Laboratory Operations Lab Technician II	1	Collect samples and process lab tests
Administrative Position	0.65	Provide office support
Temporary Laborer	1[1]	Provide temporary assistance for a variety of jobs in both the water and wastewater systems.

[1]The amount of hours for this position varies with each budget cycle generally averaging 2,000 man-hours annually.

In reviewing the distribution of manpower between several areas, the following observations were made based on a review of data for a typical month, April 2007.

- WWTP and pumping stations account for 50 percent of the workforce time.
- The level of effort to operate and maintain the composter requires an additional 16 percent of available labor.
- The water treatment and distribution pumping system requires approximately 20 percent of the available labor.
- A commendable level of effort (6.7 percent) is devoted to safety and training. This percentage may decrease as staff experience increases, and may also fluctuate as current senior staff become eligible for retirement in the next 5 plus years.
- Earned time off (holidays, sick leave, vacation, etc.) averages 8.1 percent of the time, equivalent to one man month.

A more comprehensive summary of work allocations for specific work areas is presented in Appendix F. This information was summarized from time sheets for two monthly time periods: November 20 through December 21, 2006, and March 21 through April 20, 2007, representing a wet period and a relatively dry period.

CHAPTER 4
IMPLEMENTATION

4.1 INTRODUCTION

This chapter summarizes the wastewater treatment improvements recommendations for a 20 year planning period and to ultimate buildout (2040). The proposed implementation schedule is based on the results of the analysis documented in Chapter 3 and both Liquids Solids Workshops conducted with City of Newberg (City) staff to address the future needs of the Newberg Wastewater Treatment Plant (WWTP).

There are three major factors that impact the wastewater service and the WWTP. These are:

- Ability to treat the City's wastewater to the required quality
- Ability to convey and treat the quantity of wastewater (hydraulic capacity)
- Ability to handle solids and compost, and deliver compost product to the public

The WWTP provides exceptional treatment to meet the required water quality requirements. However, the plant is currently limited based on hydraulic capacity and solids handling capacity. Modifications need to be made to provide immediate treatment capacity and future capacity in phases through 2025, and to define the land area needs for ultimate build-out in 2040 including:

- Phase 1: Immediate improvements to 2015 (2007-2025)
- Phase 2: Improvements to 2025 (2015-2025)
- Phase 3: Improvements for ultimate build-out (2025-2040)

The recommended Repair, Renovation, and Expansion Projects (RRE Projects) and phasing are summarized in Figure 4-1.

4.2 RECOMMENDED IMPROVEMENTS

The recommended improvements for the RRE projects are summarized below.

4.2.1 Influent Pump Station (IPS)

The IPS is an essential component of the WWTP. It pumps the wastewater approximately 100 feet between the lowest point in the collection system up to the headworks that provides screenings and grit removal. The pump station is currently under capacity. It cannot convey peak flows when one unit is out of service. Typical high influent flow events could cause permit violations. In addition, there are safety concerns with the existing pump station wet well. The wet well is inefficient and causes frequent problems from rags and debris clogging the pump impellers, which decreases the pumping capacity and requires frequent cleaning. The IPS upgrades and expansion are needed immediately.

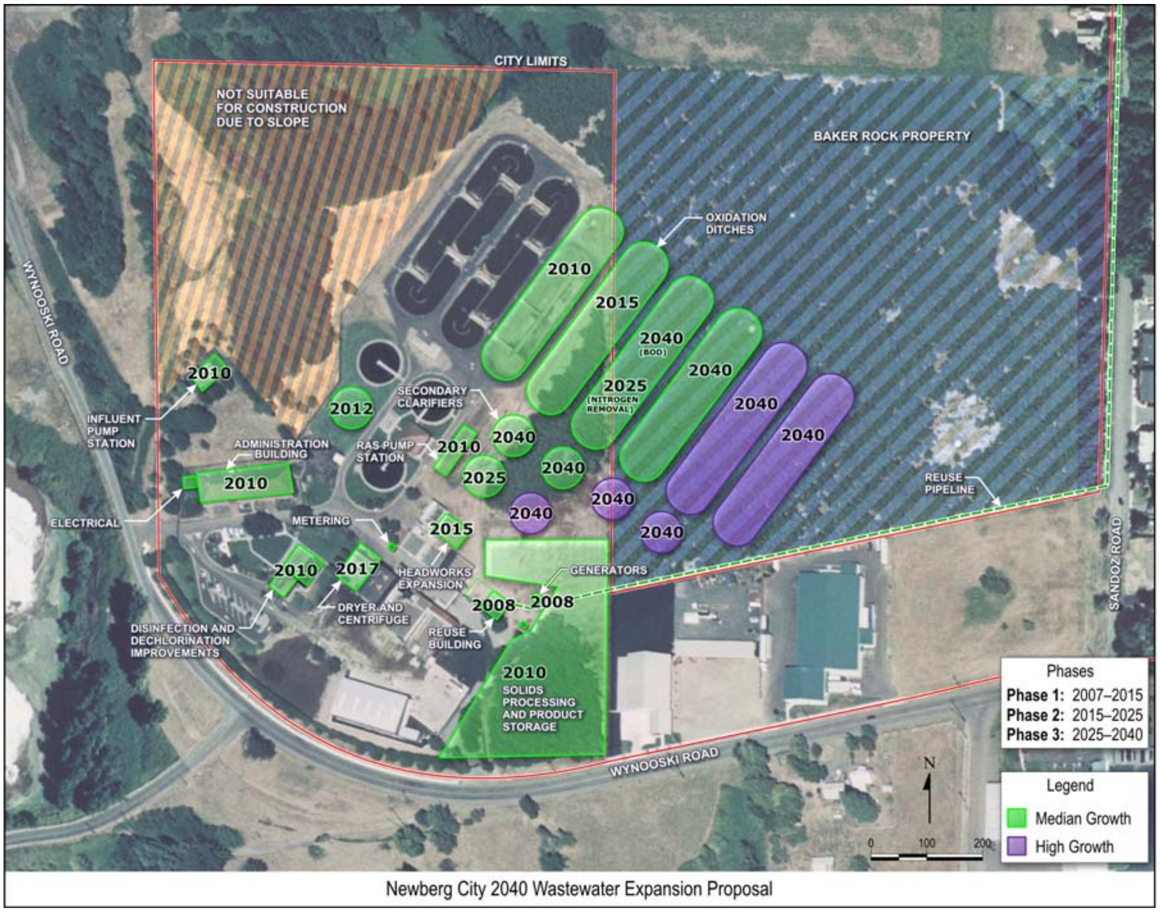


Figure 4-1. Newberg 3-Phased WWTP RRE Projects

The recommended improvements to the IPS, for safety and capacity reasons, include building additional capacity next to it for base flows and upgrading the existing wet well for overflow capacity pumping. The range of flows expected at the IPS is best accommodated by a dual pump station; low and moderate flows would be pumped by a station with a self-cleaning wet well, while higher wet weather flows would be pumped by the overflow pump station with confined inlet pumps. The recommended IPS improvements include modifying the inlet pipe slope, wet well, and related structure for 2040 flow conditions. The pumps selected and installed will be for 2025 flow conditions. The pump station will be able to pump flows in excess of 2015 flows because of the pump sizing constraints that more ideally fit the 2025 phasing. Variable-frequency drives for these pumps are included in the cost estimate. The expansion to Phase 3 will require only modifications or replacement of pumps. The IPS Electrical Room (being designed and constructed by others) is sized for future space requirements.

The proposed pump station layout is shown in Figure 4-2.

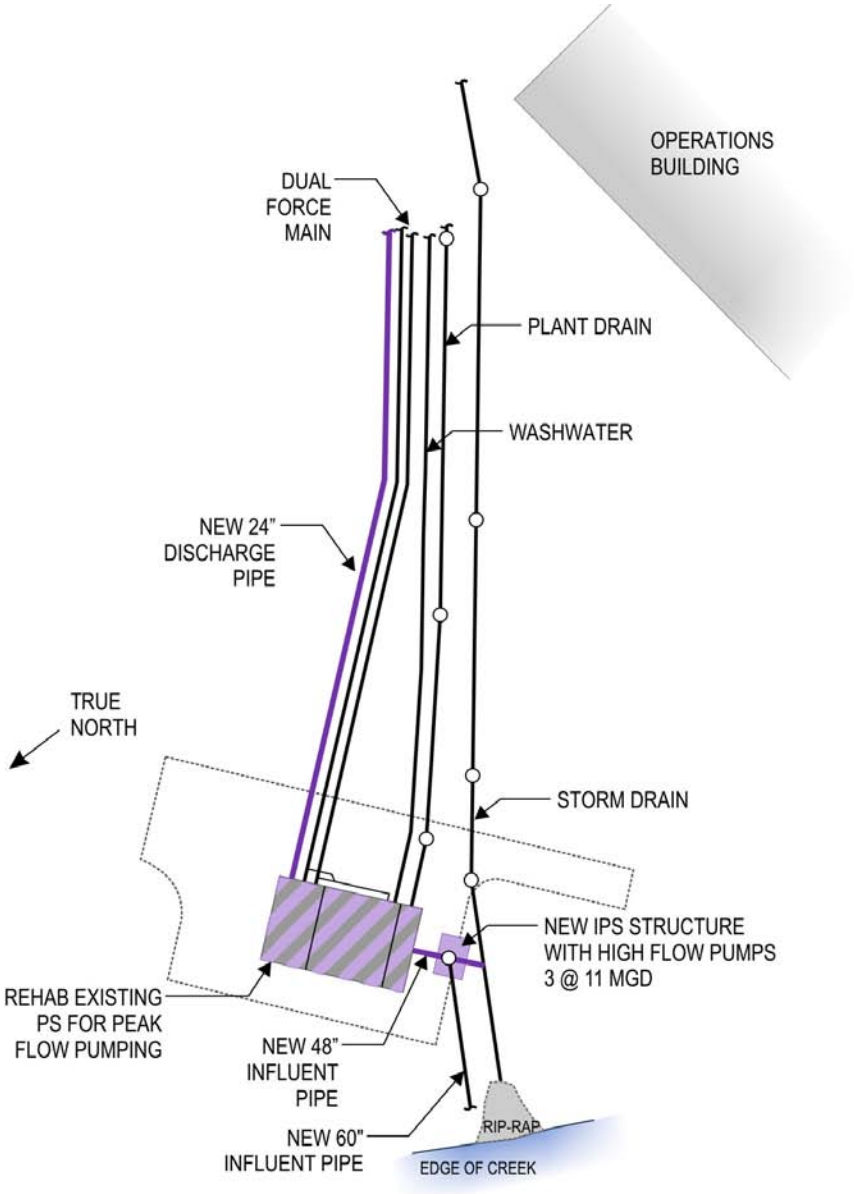


Figure 4-2. IPS Yard Piping Plan

It is also recommended that a section of the influent pipe be elevated sufficiently to remove the slope to the IPS that causes poor influent characteristics and high velocities at peak flows in the IPS. The influent pipe will be a new 60-inch-diameter pipe at a slope of 0.0007 foot per foot to limit inflow velocities to less than 4 feet per second from the first upstream manhole to the new IPS structure. This size pipe is satisfactory for both current and 2040 flow rates so that replacement in the future will not be necessary. When the influent pipe is re-laid, the slope into the wet well will be improved, and the new self-cleaning wet well will be located adjacent to the existing IPS but at a higher elevation.

During the facilities planning process, the Motor Control Center Building location for the IPS was discussed as part of the reuse design process. It was determined that a location to the west of the Administration Building would be optimum. This location avoids the influent piping at the east of the Administration Building, is adequately proximate to the IPS, and avoids the additional costs of construction on a steep slope and where the site is already constrained adjacent to the IPS.

The phased improvements, based on peak hour flow requirements, will provide the incremental IPS capacity, as shown in Figure 4-3.

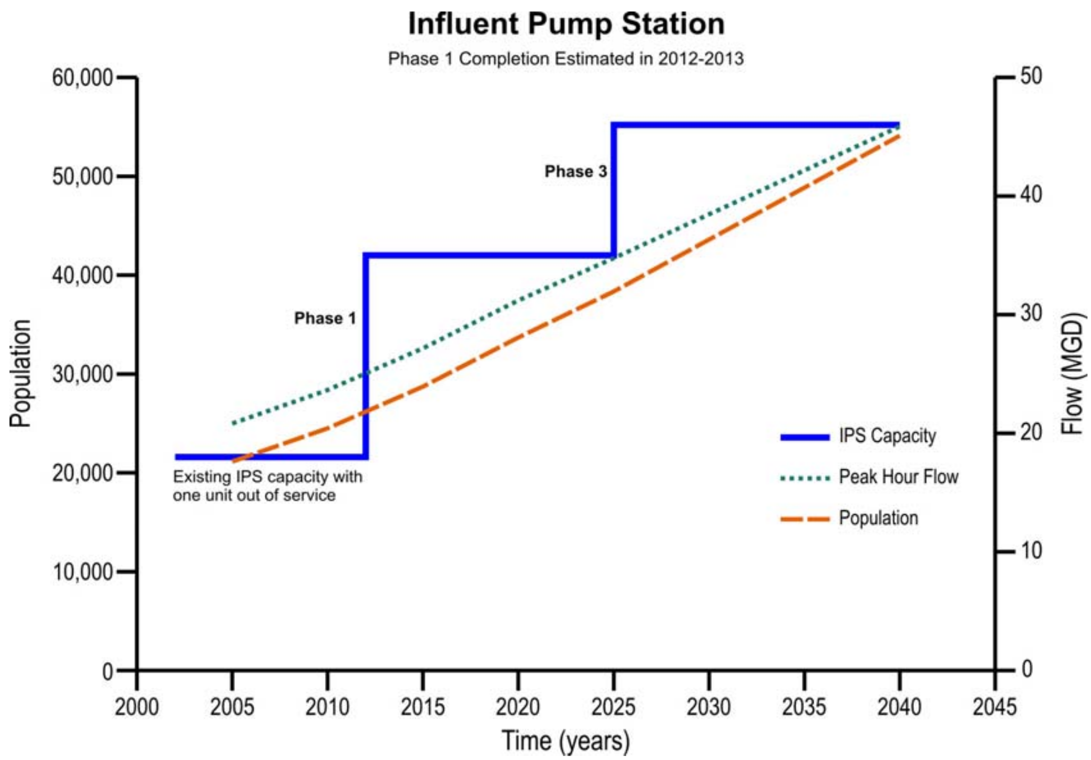


Figure 4-3. Incremental IPS Capacity

4.2.2 Headworks

The headworks processes include screening and aerated grit. The screens remove particles greater than 10 millimeters in diameter. The grit is removed with an aerated grit chamber. Although the screens were recently replaced with new, more reliable screens, the existing channel configuration does not allow conveyance and treatment of the total influent flow when one unit is out of service without bypassing around the process.

It is assumed that expansion will include the same type of screens as existing for ease of operations and maintenance and because they were determined to be cost-effective in 2002 during the Newberg Dump Station and Headworks Study conducted by Brown and Caldwell. The most cost-effective screen was chosen at that time. Plant staff have had positive experiences with these screens.

The screens will be installed in channels on the east side of the existing headworks, as shown in Figure 4-4. Emergency power should be added to ensure that critical headworks functions can continue in the event of a power outage. Odor control should be provided also as a good neighbor policy and to maintain compliance with Oregon Administrative Rules 208 that prohibits nuisance conditions such as odors.

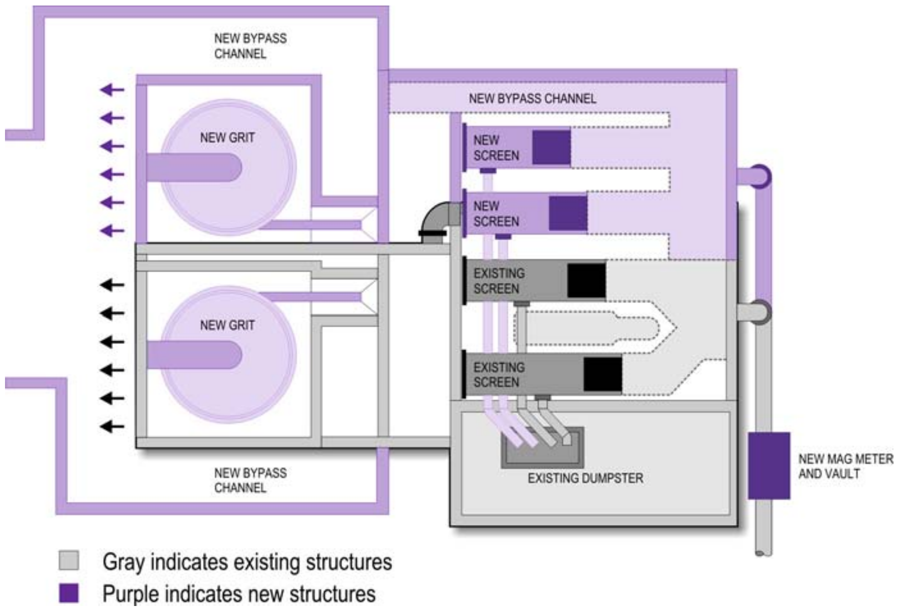


Figure 4-4. Headworks Improvements

The grit removal process is currently undersized, and the recommendation is to provide full grit removal for all flows. Therefore, additional grit removal capacity is needed immediately.
The tray separator system that removes grit using a series of stacked plates is the recommended grit removal system to provide the capacity.

New flow distribution and flow monitoring will need to be provided. The existing magmeters are not installed for accurate flow measurement. Magmeters will be installed approximately 10 to 20 feet upstream of the headworks to more accurately measure flow. The phased improvements, based on peak hour flow (PHF) requirements, will provide the incremental headworks capacity, as shown in Figure 4-5. The headworks will need to be expanded by 2015.

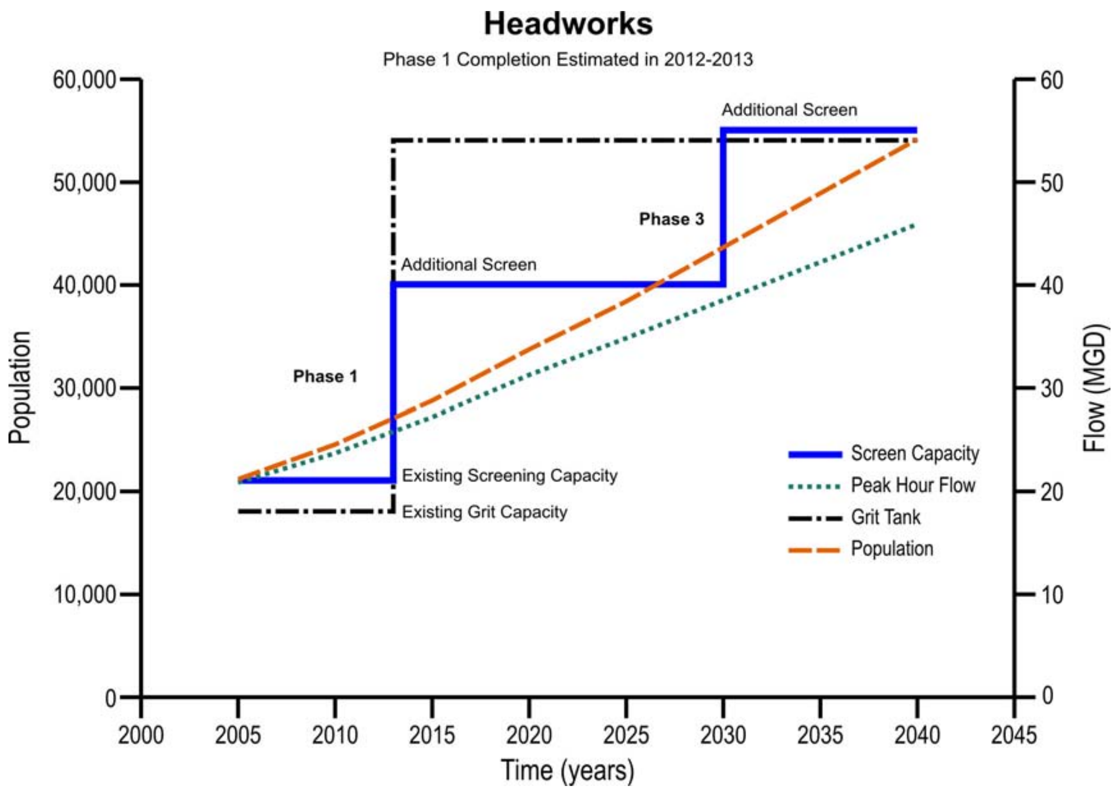


Figure 4-5. Phased Headworks Capacity

4.2.3 Secondary Treatment

The Newberg WWTP currently uses two oxidation ditches for secondary biological treatment to meet regulatory permit requirements. The secondary system is currently undersized for maximum month flow conditions.

4.2.4 Oxidation Ditches

The recommended expansion includes continuing to use the oxidation ditch process because of its low energy and maintenance costs and its ability to treat a wide variation in flows and loads. As shown in Figure 4-1, by 2010, a third oxidation ditch is needed to provide adequate treatment to meet effluent quality requirements. A fourth oxidation ditch is needed by 2015. The City has an interest in acquiring the adjacent Baker Rock property for expansion of the secondary system. However, in the event this land area expansion does not take place, additional processes were considered.

Expansion with the current oxidation ditch and secondary clarifier processes is the preferred alternative. Should site constraints or significantly more stringent effluent quality become an issue, membrane treatment could be added either in conjunction with the oxidation ditches or by replacing the oxidation ditches and secondary clarifiers with MBRs which would significantly reduce the footprint requirements.

The phased capacity expansions for the oxidation ditch process, based on maximum month flow and no nitrogen reduction requirements until 2025, are shown in Figure 4-6.

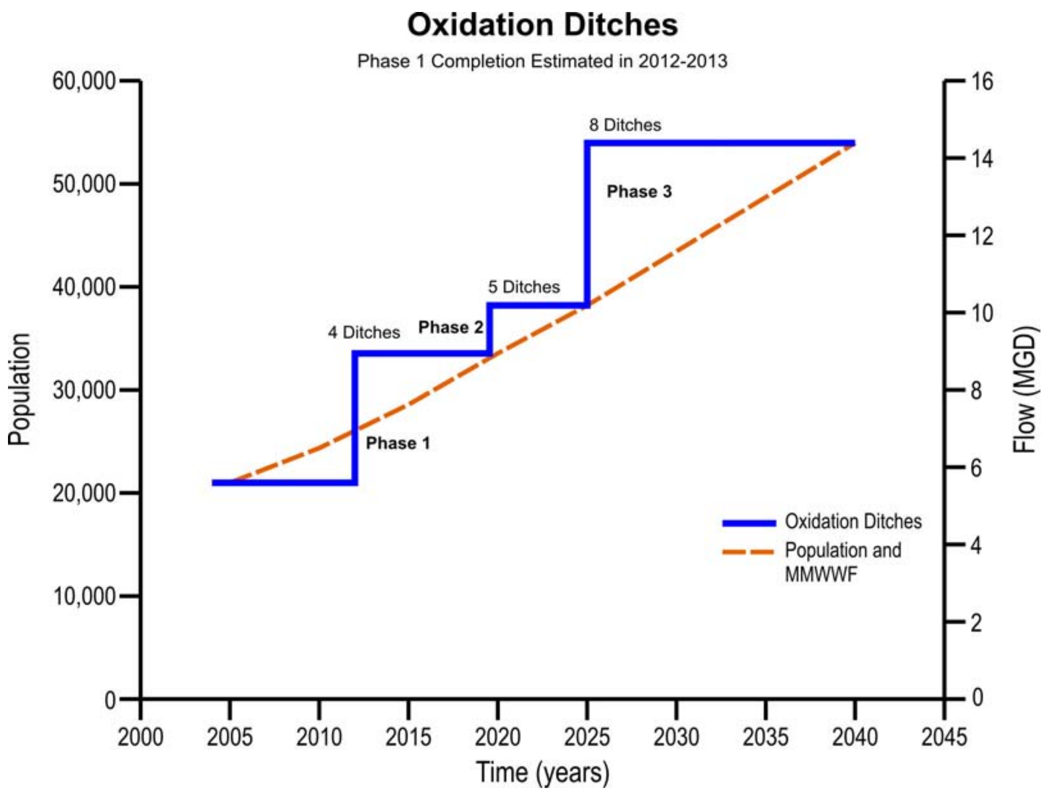


Figure 4-6. Phased Oxidation Ditch Capacity with Nitrogen Reduction Requirements

4.2.5 Secondary Clarification

Secondary clarifiers separate the biological organisms from the biologically treated wastewater prior to disinfection. The capacity of the secondary clarifiers is related to both hydraulic flow and the mass of biological solids from the oxidation ditches. As shown in Figure 4-1, the secondary clarifier process will need to be expanded with increased population and to match the additional oxidation ditch capacity. By 2012, a fourth secondary clarifier will be needed to meet effluent quality requirements. The phased capacity of the secondary clarifier system, based on maximum month flow requirements, is shown in Figure 4-7.

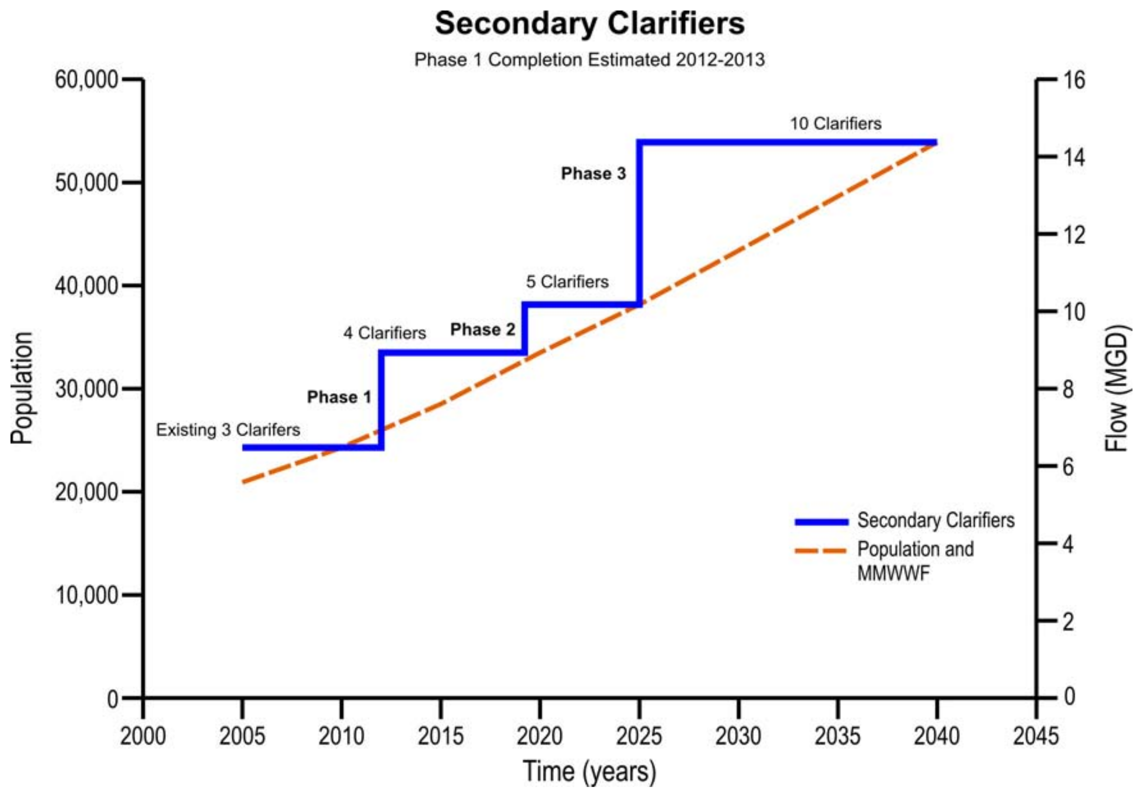


Figure 4-7. Phased Secondary Clarification Capacity with Nitrogen Reduction Requirements

4.2.6 Disinfection Process

Clarified effluent must be disinfected prior to discharge or reuse. Currently, the disinfection process consists of a chlorination system that uses ton cylinders of chlorine gas. Immediate changes are needed to improve the reliability of the effluent quality to continue to meet disinfection permit requirements. These include chemical induction mixer(s) at the chlorine injection point, scum removal, improved effluent flow monitoring, and automatic disinfection control strategy. Roof drainage needs to be re-routed out of the contact basin.

The City will continue with gas chlorine for the first 5-year permit cycle as well as the existing contact basins. The City is considering phasing in hypochlorite when the Newberg WTP is constructed in closer proximity to the WWTP, which is not expected until 2017. High-rate disinfection can be used to increase the effectiveness of the disinfection to accommodate the limited contact time in the existing contact chamber. The City is also investigating the applicability of phasing in ultraviolet (UV) treatment at a later date. UV disinfection may not be feasible at the WWTP since the effluent has iron which can inhibit the effectiveness of the process.

Disinfected wastewater is currently dechlorinated at the outlet of the chlorine contact basins. The dechlorination system requires complete replacement to be more effective, but currently capacity is limited by the configuration of the equipment. A new 1,050-gallon high-density polyethylene storage tank, two new feed mechanical diaphragm pumps, and a new control system are recommended for immediate implementation. The phased disinfection capacity, based on PHF requirements, is shown in Figure 4-8.

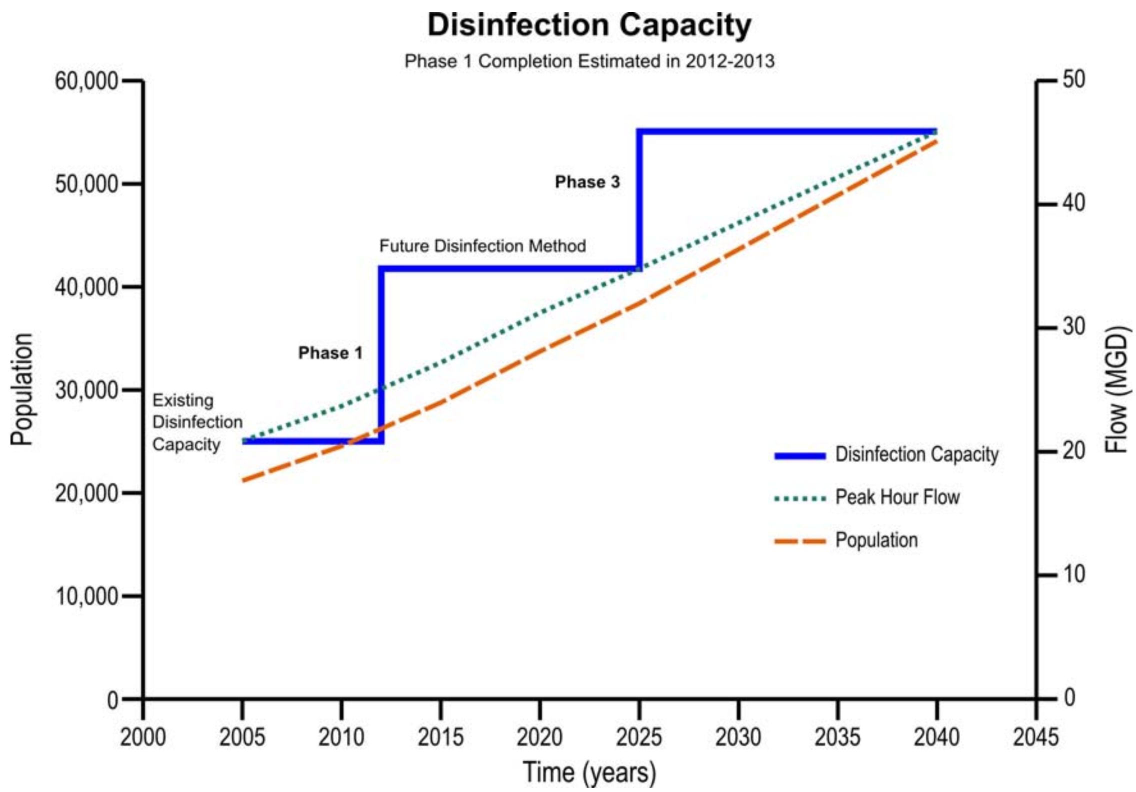


Figure 4-8. Disinfection Capacity

4.2.7 Outfall

The outfall is primarily a conveyance unit process and the capacity is needed to convey peak flows to the river discharge point. Due to hydraulic conditions caused by air entrainment at high flows that are called a hydraulic cannon, the outfall has experienced structural damage to the uphill manhole. In order to alleviate the hydraulic cannon effects, a parallel outfall down the slope is recommended to be implemented immediately for safety reasons. This will prevent the air entrapment and alleviate the hydraulic effects. The phased outfall capacity increase, based on PHF requirements, is shown in Figure 4-9.

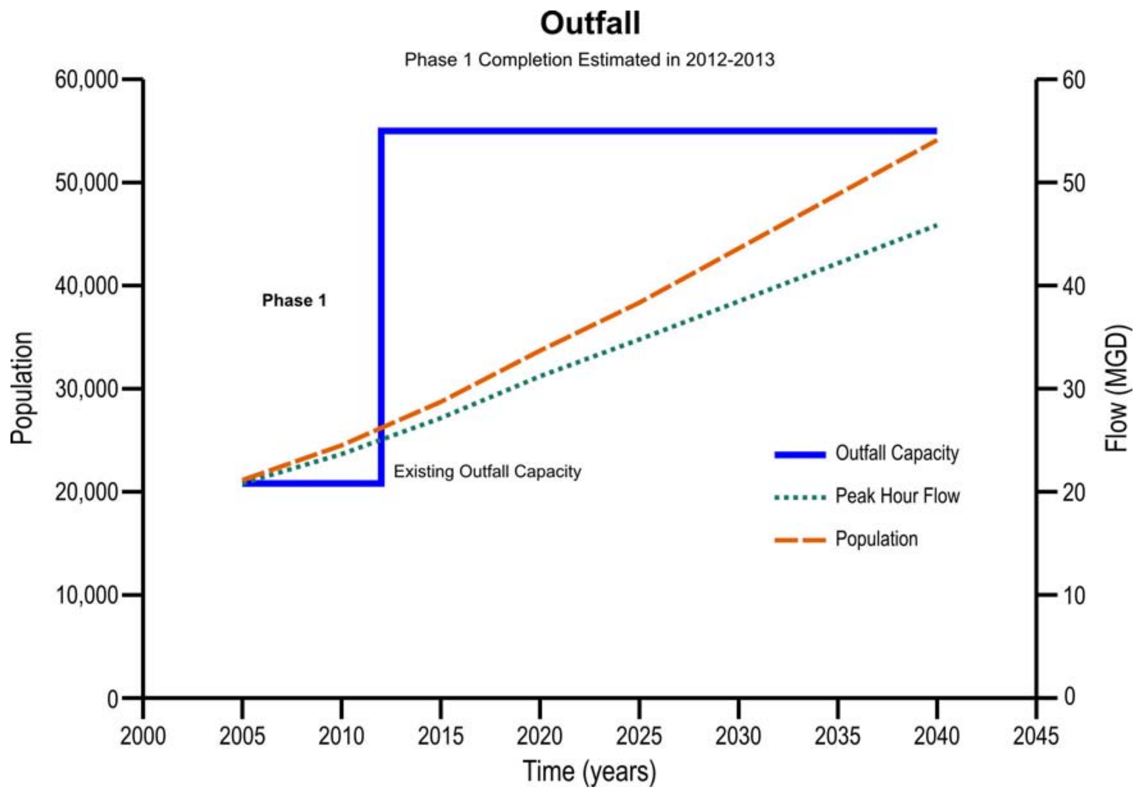


Figure 4-9. Phased Outfall Capacity

4.3 REUSE

The City is implementing plans to use treated effluent for irrigation of a local golf course.

4.3.1 Irrigation

Tertiary treatment using membranes has been selected by the City after a predesign evaluation and recommendation documented in the *Reuse Water System Predesign Study* (CH2M Hill, October 2005). Membranes will be assumed to be the preferred technology for future tertiary treatment for reuse at local golf courses (by others). The City is currently planning to provide variable reuse water from April through October, with the lowest demand expected in April. The peak delivery capacity for the hottest summer months is 1 mgd.

4.3.2 Temperature Compliance

DEQ will be implementing the Willamette total maximum daily load (TMDL) waste load allocations (WLAs) in the City's open National Pollutant Discharge Elimination System permit. The City has the opportunity to track river temperature, and effluent temperature, and flow on a 7-day running average to comply. The City conducted preliminary analyses showing that it will be able to add approximately 40 to 44 million kilocalories of heat energy per day to the Willamette River without violating the WLA. The WLA is based on a 7-day average. In 2005, the summer the maximum

7-day temperature was 23.5 degrees Celsius. According to the City's analysis, this would result in a maximum allowable discharge of approximately 3.0 to 3.5 mgd, though this could vary slightly depending on the temperature of the Willamette River. The maximum 7-day summer flow (May to October) was 4.1 mgd. The City is implementing a reuse program to irrigate local golf courses that will decrease its effluent discharge by 1 mgd.

The City plans to add additional reuse to address the temperature WLA in the future. Depending on the final temperature management plan, some storage may need to be provided. The golf course that will use the treated water for irrigation has 3 million gallons of storage capacity.

4.4 SOLIDS HANDLING AND TREATMENT

The compost process has reached capacity because the compost feed mix has a high moisture content. Compost capacity is based on peak week solids production, solids, and feed sawdust moisture content. Recent market demands for sawdust has resulted in smaller buyers (including the City) receiving wetter sawdust product. This has resulted in an immediate need to provide static compost piles in addition to the mechanized composting operation. Decreasing the moisture in the sawdust with a sawdust dryer would result in a capacity increase and is recommended for immediate implementation. Capital costs are substantially lower than that of mechanical dewatering, and the operation can provide the maximum immediate benefit in terms of compost system capacity. The City has purchased and is currently installing the dehydrator in 2009. Operational experience will determine the capacity realized.

The centrifuge was the recommendation prior to the City's implementation of the dehydrator solution. Since the dehydrator can dry to any desired dryness, there are lower energy options for dewatering solids than using centrifuge technology. These include the screw press and recent improvements to belt press technology.

For Class A process technologies, composting and thermal drying are nearly equal in cost, while offsite energy recovery is much less. Initial evaluations favor offsite energy recovery. Plant staff have indicated that a backup strategy using a simplified composting technology (aerated static pile [ASP]) is desired until offsite energy recovery has been implemented locally and has been proven reliable for long-term service.

4.5 ADMINISTRATION BUILDING

As part of the Facilities Planning process, an evaluation of the Administration Building at the WWTP was conducted. The purpose of the Administration Building evaluation was to develop a concept for a functional, secure, and energy-efficient facility that will improve operations. Built in 1987 and in operation since then, the Administration Building has undergone a number of significant changes in its programmatic functions over the past 20 years. Few design changes and upgrades have occurred over this period leading to a building that is highly inefficient in the use of its available space. For example, major functions such as the maintenance workshop have been moved out of the building into more appropriate locations on the plant site leaving underutilized voids of space; while remaining critical functions, such as the laboratory, administrative areas, and staff support areas, have developed increasing needs for space and technical updating. Most critically, emergency generator exhaust entered the buildings' ventilation system, creating worker safety considerations. The recommendation to move the engine generator out of the existing building is already being implemented.

The planning considered the needs for 2025 and the potential to house the City's Water Treatment Plant (WTP) administrative personnel and certain water treatment plant functions (shop, laboratory, etc.). The Administration Building improvements are recommended to be implemented immediately.

The proposed layout for the Administration Building is shown in Figure 4-10. When complete, the remodeled building will be a much improved facility with increased flexibility for growth, greater efficiency, and expanded functionality; and it will be a more productive environment for the WWTP staff and potentially the WTP staff to carry out its mission to the community.

4.6 WASTEWATER TREATMENT SUPPORT SYSTEMS

Improvements to the wastewater treatment support systems are summarized below.

4.6.1 Emergency generator

The emergency generator needs were established as part of the Administration Building Predesign Report. The emergency generator project is being completed by others as part of the reuse improvements.

4.6.2 Building Improvements

Based on several meetings and a site walk-through on September 29, 2006, miscellaneous improvements were recommended to the following buildings:

- Chlorine Building, chlorine scrubber, and duct
- Secondary Return Activated Sludge/Waste Activated Sludge (WAS) Pump Building
- Solids Building
- Compost Building

4.6.3 Stormwater

Stormwater generated onsite is conveyed by gravity to the IPS along with recycle streams. In-plant stormwater handling alternatives were studied and documented in a previous report entitled *Final Report for the Recommended Plan City of Newberg Dump Station/Headwork Studies* (*Final Dump Station/Headworks Studies* report) (Brown and Caldwell, June, 2002).

The design is based 60,000 gallons of runoff per 1 inch of rain for the current plant site. This value was based upon plant staff observation. Applying this amount to a 5-year rainfall amount of 3.10 inches resulted in a total volume of 186,000 gallons. This volume was applied over a 24-hour period, which resulted in a peak runoff rate of 510 gallons per minute (gpm).

During the facility planning process, plant staff requested that the in-plant pump station be sized for the recommended plant expansion. Scaling up to the plant site for 2025 results in approximately 850 to 900 gpm for a runoff rate. This could be decreased if the plant decreased its impermeable area during new construction. The 50,000 gallons per day (gpd) of recycle water was converted to gpm, assuming 6 hours of flow per day. This rate totaled 140 gpm over a 6-hour period. The analysis also included an allowance for pumping 50,000 gpd of recycled water from plant operations. This rate was added to the peak stormwater runoff rate to arrive at a peak flow rate of 650 gpm. Scaling up is based on increased land area and sizing the wet well for 2025.

The pumps will be installed in a circular 5-foot-diameter wet well consisting of precast manhole sections 8 feet deep. The pump station will be located in the vicinity of Stormwater Manhole No. 1 and will be connected to the stormwater and recycle water systems through new gravity sewers. Flow will be conveyed from the new station to the headworks via approximately 320 feet of 4-inch ductile iron pipe. An overflow pipe will allow flows exceeding the capacity of the pumping station to re-enter the existing stormwater system via a new manhole constructed over the stormwater pipe discharging to the IPS. We have made the following assumptions:

- Pump station is located in a 5-foot-diameter manhole, 8 feet deep
- Two submersible pumps
    - one duty
    - one standby
- Pumps are 10 horsepower
- Outdoor control panel is mounted on a pole next to the manhole

4.6.4 In-plant Reclaimed Water

The current reclaimed water system filters for in-plant use are inadequate and the screening size is too large to be effective. A looped plant water system is recommended that includes adding a source of plant water at the headworks and providing more hose bibs for cleaning at the aeration basins.

4.6.5 Septage Receiving

New septage receiving is recommended. Septage receiving was studied in *Final Dump Station/Headworks Studies* report. The recommended improvements include modifications to the road southeast of the headworks (including a trench drain and catch basin), a Lakeside 31SAP-type septage receiving station, a buried septage receiving tank, duplex pumps in the septage receiving tank, piping to transfer the septage to the screening channel of the headworks, and a new access road around the north side of the headworks. The screenings from the station will be bagged, so no roof over the septage screenings dumpster will be required. Vactor trucks can discharge on the ramp leading to the septage receiving station. Rocks and debris will be manually removed.

4.6.6 Miscellaneous Facility Review Recommended Improvements

Miscellaneous needed improvements were identified in the facility review process. These are summarized in Appendix E. These improvements are being implemented by plant staff or have been identified as a capital improvement project included in the RRE projects.

4.7 PHASING OF RECOMMENDED IMPROVEMENTS

The phasing of the RRE projects is shown in Figures 4-1 and 4-11. Phasing is planned in three increments. Phase 1 is to be completed as soon as funding is available. Figure 4-11 shows the estimated population projections for low, median, and high growth scenarios, year that the estimated growth is expected to occur, and the planned capacity phasing.

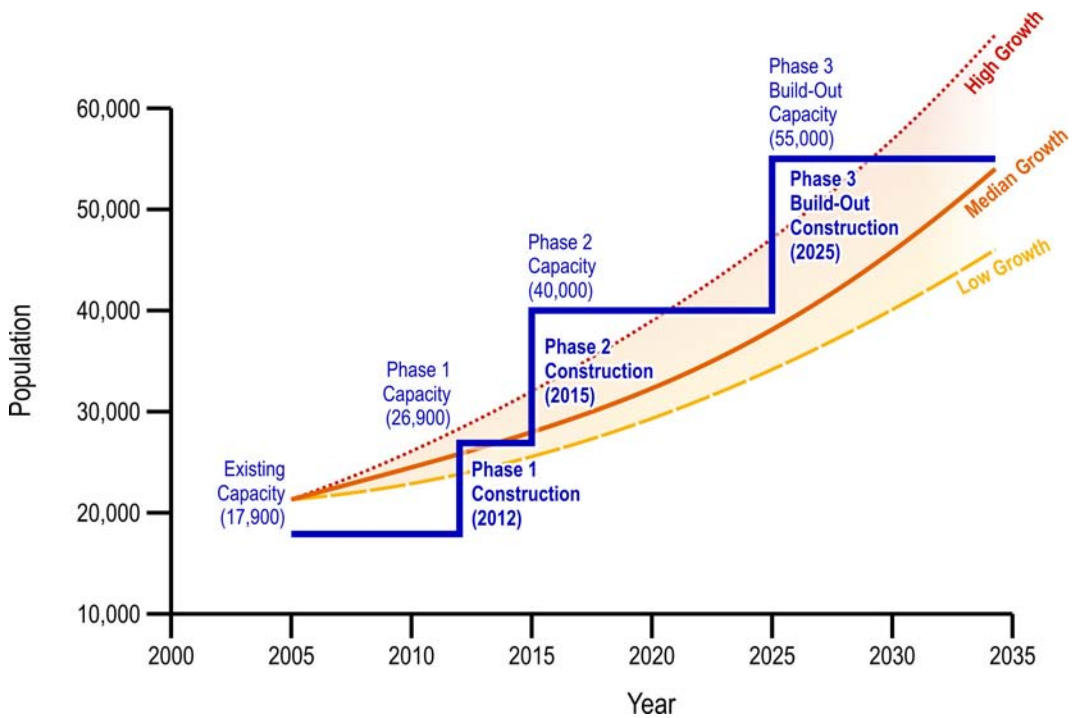


Figure 4-11. Planned Phased Construction Assuming no I/I Removal

The impacts of the Newberg infiltration/inflow (I/I) elimination program will affect the capacity of the WWTP RRE projects. However, these impacts will not immediately reduce the first planned RRE project scope, but will delay, reduce, and/or, postpone future project expansions. The first phase RRE is needed to convey and treat the I/I flows until collection system improvements result in decreased I/I. Reductions in I/I are not expected to occur until after the first phase RRE is implemented. An effective I/I elimination program in the sewer system—after implementation—could potentially postpone future WWTP RRE project construction related to hydraulic expansion.

4.7.1 Phase 1 RRE Projects, 2007 to 2015

The RRE projects that need to be completed immediately to provide service through 2015 are shown in Figure 4-12 and the order-of-magnitude costs or Capital Improvements Program projects are summarized in Table 4-1. The Phase 1 improvements will be implemented immediately and will meet the needs through 2015 based on median growth projections. Actual phasing implementation may vary depending on the City's priorities. The proposed modifications (shown in green) are needed by the date shown on the site plan. The IPS, disinfection and dechlorination improvements, miscellaneous facilities, outfall, and sawdust drying are needed immediately.

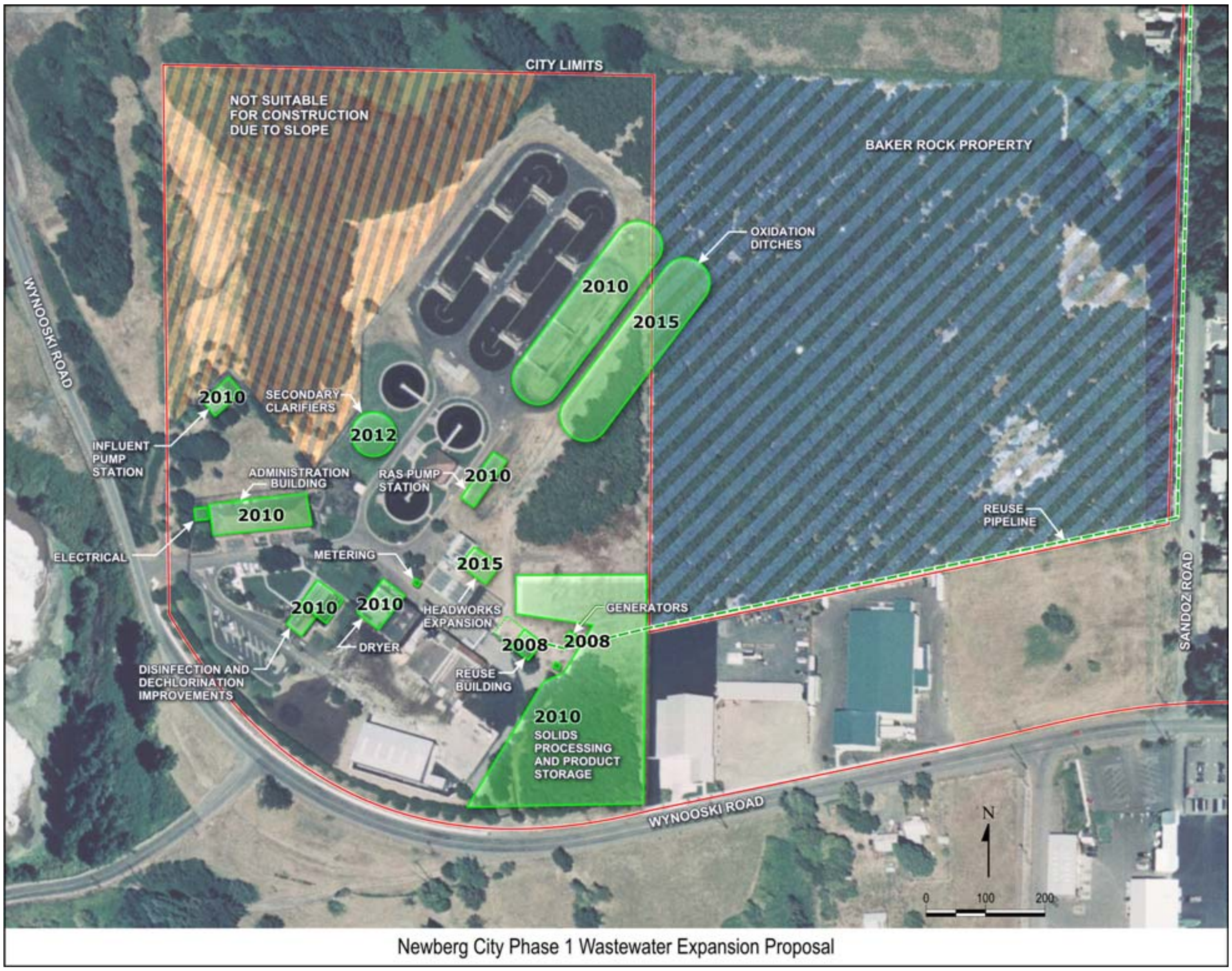


Figure 4-12. Recommended Improvements for Phase 1

Table 4-1. Capital Costs for Phase 1 RRE Projects 2007 to 2015

WWTP improvements	Cost, dollars	Comments
IPS and parallel discharge pipe	3,124,700	Needed immediately
Influent pipeline improvements	287,000	Needed with IPS.
Influent metering	250,000	
Headworks improvements	4,145,700	
Headworks odor control first phase	70,000	
Septage receiving	395,500	
Third and fourth oxidation ditch	6,565,000	
Existing oxidation ditch repairs	573,500	
Fourth secondary clarifier	3,251,200	
Splitter structures	650,000	
Disinfection	425,300	Needed immediately
Dechlorination	339,000	Needed immediately
Outfall	367,900	Needed immediately
In-plant reuse water	85,000	
In-plant stormwater pump station	474,300	
Building upgrades		
Chlorine Building	77,200	
Chlorine scrubber and duct	833,300	
Secondary Building	346,300	
Solids Handling Building	348,100	
Compost Building	468,400	
Sawdust dryer	533,000	2007 dollars; Needed immediately; Energy funding available
Level IV reuse facilities (by others) and storage		Provided by others
Administration Building	1,496,100	
Subtotal, construction cost	25,106,500	
Administration/engineering costs at 25 percent	6,276,600	
Total capital cost	**31,383,100**	**Escalated to 2011 mid-point of construction except as noted**

The upgrades to the IPS in Phase 1 and all ancillary equipment will be able to handle peak flows up to 2040 projected flow rates. The pumps included in Phase 1 will have the capacity to handle peak flows up to 2025 projected flow rates. In 2025, the pumps will be nearing the end of their useful life and will need to be replaced. A third screen will be needed to meet the flow requirements in 2015. Two grit basins will be needed by 2015.

To serve the needs for the 2015 median growth scenario, the plant will need to add two oxidation ditches and one clarifier assuming no additional nitrogen reduction is needed. One of the oxidation ditches will be built in the location of the existing equalization basin. The existing oxidation ditches will be rehabilitated.

The City will continue with gas chlorine for the first 5-year permit cycle as well as the existing contact basins. The immediate disinfection improvements that need to be made include chemical induction mixer(s) at the chlorine injection point, scum removal, improved effluent flow monitoring, and automatic disinfection control strategy. Roof drainage needs to be re-routed out of the contact basin.

The parallel outfall is included as needed improvements in Phase 1. No improvements other than the planned reuse is needed for temperature compliance in Phase 1. A sawdust dryer is needed immediately to increase capacity of the compost system.

4.7.2 Phase 2, RRE Projects for 2015 to 2025

The RRE projects that need to be completed to meet the Phase 2 needs from 2015 to 2025 are shown in Figure 4-13 and the order-of-magnitude costs are summarized in Table 4-2. The fifth oxidation ditch will be required in 2025 if nitrogen reduction becomes a regulatory requirement. If nitrogen reduction is not required, the fifth oxidation ditch will not be needed until 2040. For the purposes of cost estimating, it was assumed that nitrogen reduction was required to meet National Pollutant Discharge Elimination System (NPDES) discharge requirements. This is a conservative estimate. Actual Phase 2 implementation will be based on the NPDES requirements at that time.

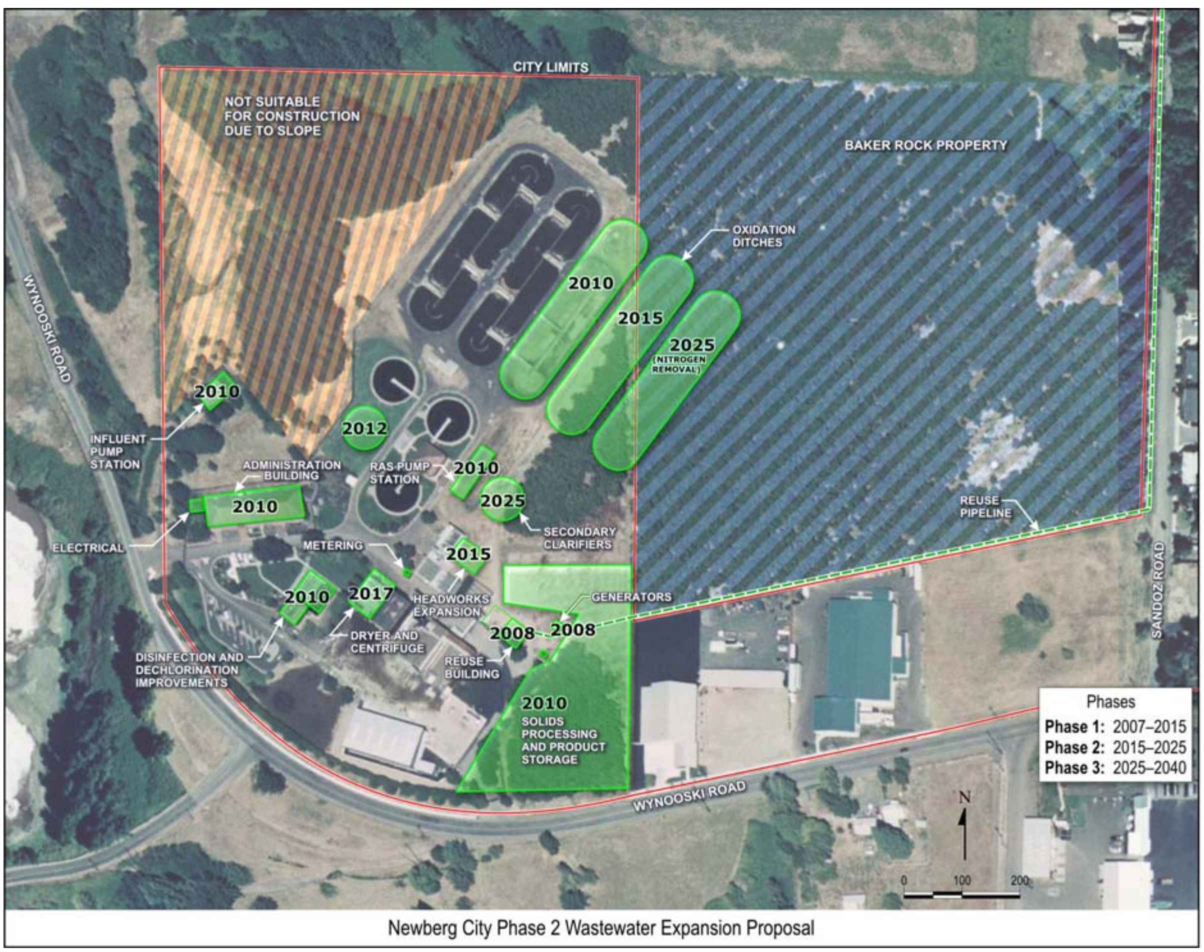


Figure 4-13. Newberg WWTP Phase 2 Improvements 2015 to 2025

Table 4-2. Capital Costs for Phase 2 Improvements 2015 to 2025

WWTP improvements	Cost, dollars	Comments
IPS and parallel discharge pipe	N/A	
Influent pipeline improvements	N/A	
Influent metering	N/A	
Headworks (screenings and grit) improvements	N/A	
Headworks odor control first phase	300,000	Potential for more odor control
Septage receiving	N/A	
Fifth oxidation ditch	4,363,000	Assumes nitrogen reduction requirements
Fifth secondary clarifier	3,251,000	
Splitter structure	600,000	
Electrical building	500,000	
Disinfection	3,065,000	Assumes conversion to UV (effluent testing needed to determine feasibility due to iron in effluent)
Dechlorination	N/A	
Outfall	N/A	
In-plant reuse water	N/A	
In-plant stormwater pump station	N/A	
Building upgrades	N/A	
Composting expansion	3,283,500	
Centrifuge Dewatering	3,508,500	(Only if City dehydrator project is not successful)
Level IV reuse facilities and storage	N/A	Level IV reuse by others
Administration Building	N/A	
Subtotal, construction cost	18,871,000	
Administration/engineering costs at 25 percent	4,717,800	
Total capital cost	**23,588,800**	**(in March 2007 dollars)**

No IPS improvements are needed in Phase 2.

If the recommended headworks expansion is conducted in Phase 1, no improvements are needed to meet 2025 conditions.

The phased expansion to 2025 will require another clarifier. A fifth oxidation ditch will be needed if nutrient reduction is required. The clarifier and oxidation ditch will be required by 2025.

The City is considering phasing in hypochlorite when the Newberg WTP is constructed in closer proximity to the WWTP. The City is also investigating the applicability of UV treatment since its effluent has iron which can inhibit UV effectiveness. The immediate improvements that are being made may be adequate through 2025. The detention time at the currently projected PHF for the median growth projection is 11 minutes. Operating history with the high-rate disinfection will provide input into when to expand the disinfection contact time. The City will have to build additional chlorine contact basins for additional contact time or switch to UV disinfection.

The outfall improvements in Phase 1 were sized to provide capacity through 2040. No additional outfalls are required for capacity. However, diffuser modifications may be required for future regulatory compliance.

When the biosolids system nears capacity, the City should evaluate the current state of composting technology. Current recommendation is based on aerated static pile (ASP) composting process. Brown and Caldwell has become familiar with a new and improved tunnel system that should be considered in future analysis. Presumably, compost upgrade would be based on the failure of the less expensive BacGen/Polaris facility being implemented.

To maintain compliance with the temperature TMDL, the peak week flows will need to be stored. Storage for the peak week flows is recommended in addition to expanding reuse.

4.7.3 Phase 3, RRE Projects for 2025 to 2040

The RRE projects to meet the ultimate buildout needs, as previously discussed, are shown in Figure 4-1. The ultimate build-out is assumed to occur by 2040. Phase 3 was identified to define the potential land requirements to serve the population at ultimate build-out of the urban reserve area. Ultimate build-out is assumed to occur by 2040. The scenarios shown on Figure 4-1 include improvements needed to serve median growth estimates as well as improvements needed if high population growth estimates are realized. Costs for the RRE projects for ultimate buildout are not included in the planning effort. The ultimate buildout was included to define the potential land requirements only.

The upgrades to the IPS have been sized such that the facility and all ancillary equipment will be able to handle peak flows up to 2040 projected flow rates. However, the pumps included in the previous cost estimate have been sized to handle peak flows up to 2025 projected flow rates. In 2025, the pumps will be nearing the end of their useful life and will need to be replaced. The City should re-evaluate its flow rates then and will need to replace the pumps with larger pumps.

A fourth screen will be needed to meet the flow requirements in 2040.

To serve the needs for the 2040 median growth scenario, the plant will need to add one more oxidation ditch and two more clarifiers. To meet the high growth estimate needs, three new oxidation ditches and five new clarifiers will be required.

The immediate improvements that are being made may be adequate until the 2025 population projection. At that time the plant will need to re-evaluate its disinfection needs based on revised population and flow predictions. Most likely the City will have to build additional chlorine contact basins or switch to UV disinfection.

The new composting system can be expanded gradually as the population increases. No additional major expenditures are expected except equipment replacement.

The outfall improvements in Phase 1 were sized to provide capacity through 2040. No additional outfalls are required for capacity. However, diffuser modifications may be required for future regulatory compliance.

For temperature compliance, the peak week flows (PWFs) will need to be stored. Storage for the PWFs is recommended in addition to expanding reuse. This analysis needs to be conducted with the reuse project, so it is not included in the recommended CIP.

4.8 STAFFING

A review of the current staffing level was conducted as part of the Facilities Planning process, and recommendations were developed for three future time periods relating to modifications and additions to the WWTP. Staffing needs will increase with the improvements and upgrades associated with the RRE projects. The staffing assessment findings from Appendix F were used to make projections for staffing levels for Phases 1 through 3.

It is recognized that many factors will have an impact on staffing recommendations, and they will need to be updated as more of the elements of the plan become firm.

4.8.1 Estimated Staffing 2007 to 2015

The recommended added personnel are one Senior Laboratory Technician/Environmental Specialist and one Operator II. Both should have cross-training for water and wastewater treatment, in order to provide flexibility in allocating manpower as needed within the operations division. Personnel might be also added at the Utility Worker level with emphasis on moving the person to an operation or maintenance role in the future. These recommendations are based on an analysis that shows a need for 2.9 full time equivalents (FTEs) for the Phase 1 time period.

The proposed facility plan calls for additions to the WWTP that will require increased manpower. Specific planned alternatives for additions to the WWTP include:

- Influent pumping
- Headworks
- Septage receiving
- Oxidation ditches
- Secondary clarification/solids separation
- Disinfection
- Outfall
- Reuse (by others)
- In-plant drains
- Solids handling
- Class A biosolids treatment
- Laboratory/Environmental

The manpower requirements for the above WWTP additions and unit processes have been reviewed and estimates are based on the following conclusions:

- Additional influent pumping station (IPS) capacity. With the new station parallel to the existing facility, more surveillance will be required.
- Two new screens will be added to the headworks. This addition will ease challenges during wet weather and consequently increase O&M attention.
- A Eutek Headcell grit removal system will be installed, increasing O&M attention.
- A new septage receiving station is planned to handle septic tank pump truck discharges that will require added maintenance.
- Expansion of the current oxidation ditch process is the preferred alternative for the future. Workload allocations will be expanded when the addition is completed.
- High-rate disinfection is recommended for the immediate improvements with ultraviolet (UV) disinfection phased in for incremental expansion when needed in the future if determined to be feasible based on collimated beam testing on the effluent. An upgraded system and potentially different technology will require more O&M personnel attention.
- Immediate improvements of the sodium bisulfite feed system, including a more advanced control system with updated and larger pumps, will require added O&M attention.
- The reuse tertiary treatment system will be added under a separate contract but will require operator attention for surveillance and periodic membrane maintenance and cleaning. It will also require daily sampling and testing for turbidity, total coliform and chlorine residual to meet Oregon Administrative Rules Division 55. O&M personnel will be required.
- The addition of a rotary dryer for drying sawdust will gain more capacity for the existing composter system. The sawdust dryer requires attendance during operation. Manpower impacts of the sawdust dryer are assumed to be neutral at this time, per discussions with plant personnel.
- The installation of a centrifuge dewatering system will gain additional capacity for the existing composter system. The requirement for this system may be delayed depending on future decisions (by others) dealing with treatment solids energy recovery options. This unit process will add O&M requirements.
- Increased laboratory and regulatory sampling, testing, and reporting for both the treatment plants and collection system with emphasis on the pretreatment program will require additional staffing. As noted above, several key staff members will be eligible for retirement within the next 5 years making it imperative to hire and train qualified staff.

Based on the above summaries of areas requiring more attention by plant staff, the following FTE estimates are presented in Table 4-3.

Table 4-3. Estimated Staffing Required 2007 to 2015

Function	Hours per week	FTEs per year
IPS	10.3	0.26
Headworks	5.8	0.14
Odor control	3.8	0.09
Septage receiving	6.4	0.16
Third oxidation ditch	5.2	0.12
Fourth oxidation ditch	5.2	0.12
Disinfection	4.2	0.10
Dechlorination	2.7	0.07
In-plant reuse	3.8	0.09
Solids dewatering/Onix system	11.0	0.28
Other processes	3.0	0.08
Laboratory/Environmental	40.0	1.0
Subtotal of Direct Labor	101.4	2.6
Safety and training (6.7 percent)	2.7	0.07
Paid time off sick/holiday/vacation (12.8 percent)	14.2	0.13
Total	118.3	2.71

Table 4-3 presents the estimated level of effort for facilities to be added over the initial phase of the plant additions. The largest single category requiring additional staffing will be the laboratory and environmental activities, with one FTE. Other unit processes will require smaller increments of attention as noted in Table 4-2. A more detailed spreadsheet is provided in Appendix F that shows the anticipated staff additions for each year. Year 1 is scheduled for 2.2 FTEs which includes the laboratory/environmental person.

Using existing job titles, the recommended added personnel should be one Senior Laboratory Technician/Environmental Specialist and one Operator II. Both should have cross-training for water and wastewater treatment. Personnel should also be added at the Utility Worker level with emphasis on moving the person to an operation or maintenance role in the future.

4.8.2 Estimated Staffing 2015 to 2025

The recommended staffing addition is a minimum of three people (3 FTEs) over the span of 10 years. These positions recommend to be filled by an additional Plant Mechanic, a Senior Environmental Technician, and an Operator II.

The facilities plan identifies increased unit process additions in the period between 2015 and 2025 as outlined in the following areas:

- Some improvements to the headworks odor control system are projected that require additional manpower.
- By 2025, a fifth oxidation ditch and a fifth secondary clarifier are scheduled to be added to the flow pattern with consequent manpower needs.
- Although there is a potential for an offsite energy process that would use dewatered biosolids, this estimate is based on the possible addition of an aerated static pile (ASP) to augment the existing tunnel composting process. This would require additional O&M personnel.
- As the plant equipment ages, maintenance needs increase. It is projected that one more full-time maintenance person will be required.
- Increased coverage for laboratory testing and regulatory reporting will also be needed, requiring additional personnel for these activities.

Table 4-4 summarizes additional FTE requirements for the Phase 2 time period.

Table 4-4. Estimated staffing required 2015 to 2025

Function	Hours per week	FTEs per year
Odor control	2.2	0.06
Fifth oxidation ditch	5.2	0.13
Fifth secondary clarifier	5.7	0.14
ASP composting process	27.0	0.68
Added maintenance needs	40.0	1.0
Laboratory/Environmental	40.0	1.0
Subtotal	120.1	3.0
Safety and training (6.7 percent)	8.0	0.2
Paid time off sick/holiday/vacation (12.8 percent)	15.4	0.38
Total	133.5	3.35

A review of the above information shows the need for the addition of a minimum of three people over the span of 10 years. These positions are recommended to be filled by an additional Plant Mechanic, a Senior Environmental Technician, and an Operator II. Specific details relating to Table 4-4 are presented in Appendix F.

These projections do not reflect potential additional personnel needs for the WTP which is proposed to be constructed during this timeframe. Staffing projections will be addressed in the design of the new facility.

4.8.3 Estimated Staffing 2025 to 2040

It is estimated that a total of three more FTEs will be required during this time period and the recommended job titles are an additional Operator I, a Utility Worker, and an Environmental Technician.

In Phase 3, unit process modifications and additions are projected to include the following items:

- IPS pumps are scheduled to be replaced in this time span. This should not entail adding more personnel.
- A fourth screen will be added to the headworks requiring some added O&M attention.
- An additional oxidation ditch and two secondary clarifiers will be added requiring added O&M attention.
- The facilities plan assumes that a new disinfection system will be required by 2025, and a potential UV system is used to project labor needs.
- Several considerations for biosolids handling are being considered during earlier years of the study period, and systems installed should be adequate though 2040. Labor needs should not be affected during the 2025 to 2040 span of years.
- Temperature regulations may require expansion of the reuse system as well as facilities for effluent storage. Increased labor and monitoring will require additional personnel.

Table 4-5 summarizes additional FTE requirements for the Phase 3 time period.

Table 4-5. Estimated staffing required 2025 to 2040

Function	Hours per week	FTEs per year
IPS added pumps	3.1	0.08
Fourth headworks screen	3.6	0.09
Sixth oxidation ditch	4.9	0.12
Sixth and seventh secondary clarifiers	6.5	0.16
UV process	7.4	0.19
Reuse expansion and storage	6.0	0.15
Added utility worker	40.0	1.0
Laboratory/Environmental	40.0	1.0
Safety and training (6.7 percent)	6.4	0.16
Paid time off sick/holiday/vacation (12.8 percent)	14.8	0.36
Total	126.2	3.2

It is estimated that a total of three more people will be required during this time period, and the recommended job titles are an additional Operator I, a Utility Worker, and an Environmental Technician. Specific details relating to Table 4-5 are presented in Appendix F.

4.8.4 Relevant Staffing Considerations

Relevant staffing considerations include sharing personnel and documenting standard operating procedures (SOPs).

4.8.4.1 Cooperative Use of Personnel for Water and Wastewater Systems

The current use of personnel in both the treatment of wastewater and the treatment of potable water is a valid use of the staff for both utilities. Personnel are routinely shifted between the facilities, allowing cross-training and backup for emergencies. This practice is particularly valuable seasonally when more emphasis is needed in the water system during summer months, and in winter when excessive flows and loading require additional wastewater system attention. It is recommended that this practice be continued and maximized as more staff members reach retirement age.

4.8.4.2 Recommended Modifications to SOP

Many utilities are facing a "brain-drain" caused by an increasing number of experienced personnel retiring, taking with them valuable knowledge on O&M of facilities. It is recommended that some formalized method be used to capture and codify this knowledge in SOP. This can be done through electronic O&M manuals, videos, and other systems. Note the City Public Works Department is currently addressing this issue by developing a department-wide Standard Operating Procedure Initiative in accordance with APWA standards. This can then be further refined into a more detailed electronic O&M procedure.

4.9 PUBLIC INVOLVEMENT

The City held an open house for the public at the City Library on October 17, 2006, to provide outreach on the public works projects including the Newberg WWTP Facilities Plan Update. The handout to explain the WWTP Facilities Plan Update is provided in Appendix C.

4.10 ENVIRONMENTAL REVIEW

The City is conducting the environmental review process for the proposed improvements as a stand alone document. Appendix H has been set aside for the environmental review document once the review process is completed.

CHAPTER 5
CONVEYANCE SYSTEM EVALUATION

5.1 BACKGROUND

The first sanitary sewers in the City of Newberg (City) were built in the late 1910s and early 1920s. Sewer construction continued as the City's population grew and the service area expanded. Today, the City has over 73 miles of sanitary sewers and seven lift stations to serve its 21,000 residents.

The capacities of the trunks and interceptors were recently analyzed as part of the 2007 Sewerage Master Plan Update (SMPU). In 2009, the SMPU was appended to include a basin transfer from the Dayton trunk line to reduce the costs of future improvements. The purpose of the SMPU is to provide up-to-date recommendations for maintaining and expanding the sanitary sewer collection system. The SMPU includes an update of flow projections, development, and prioritization of capital improvements to address collection system deficiencies, recommendations for infiltration/inflow (I/I) reduction activities, and preparation of recommendation costs. The SMPU identifies the growth needs of the City's sanitary sewer system for the next 20 years with consideration of future flows through 2040.

5.2 DESCRIPTION OF EXISTING FACILITIES

A basic description of the City's sanitary sewer and lift stations is included below. The information is based on data provided by the City from its geographic information system (GIS) and from interviews with City staff.

Information on the sanitary collection system maintenance program can be found in the Maintenance Program Evaluation, included as Appendix A of the SMPU. For more detailed information on the lift stations, please see Appendix B, Lift Station Assessment included in the SMPU.

5.2.1 Existing Collection System

According to the City's GIS, the sanitary collection system includes over 73 miles of gravity sewer, approximately 3 miles of force main, nearly 1,700 access structures (i.e., manholes and cleanouts), and seven lift stations. Figure 5-1 shows the locations of the existing lift stations and other major components of the sanitary collection system. The number of service connections or laterals is estimated to be about 6,400. The City maintains the laterals from the mainline to the property line. Approximately 80 percent of the laterals have a cleanout at the house. Dual service connections made after 2005 have the cleanout at the property line as per City policy. The cleanout requirements for single service connections are made on a case-by-case basis.

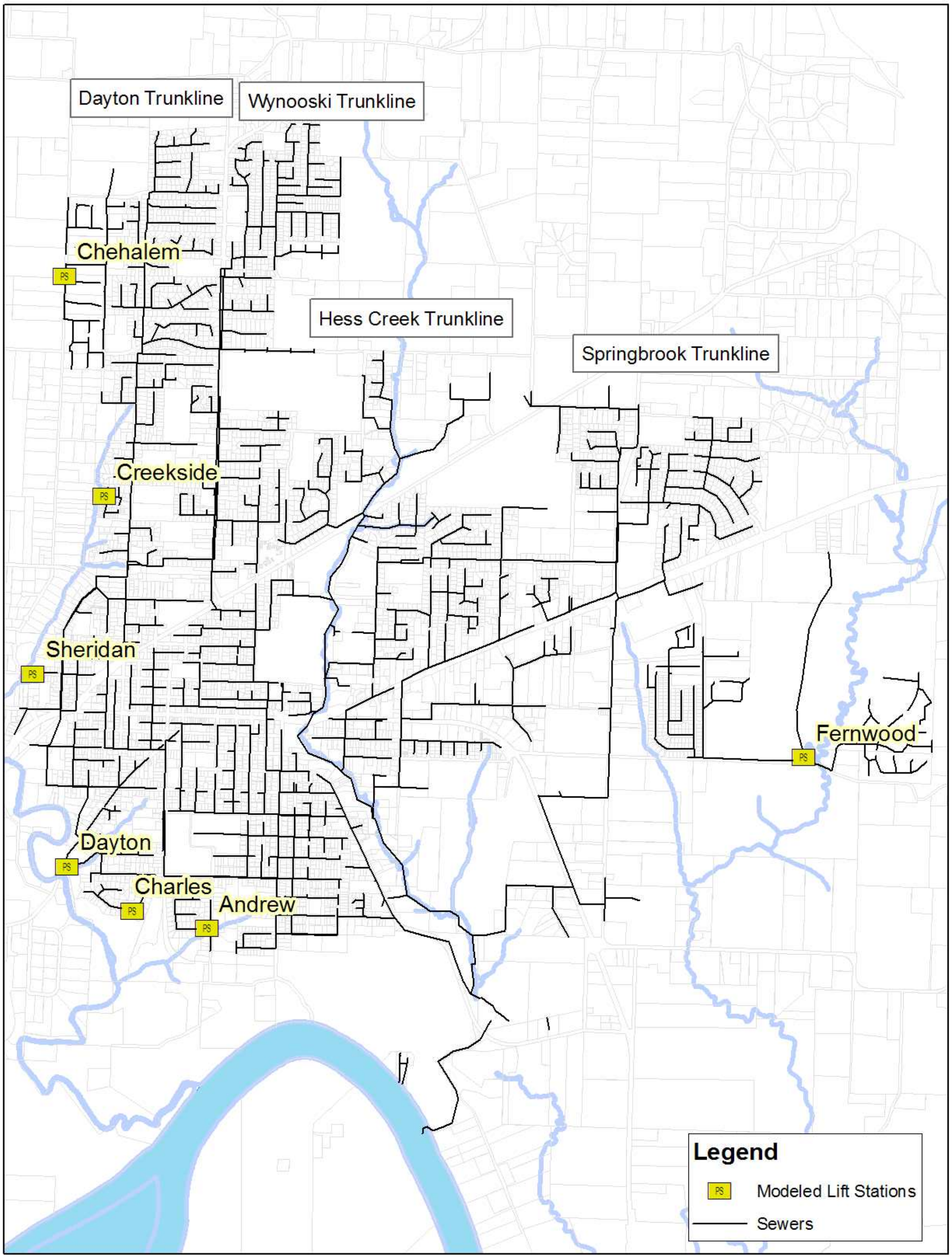


Figure 5-1. Sanitary Collection System, 2007

The size distribution of pipes within the sanitary collection system is shown in Figure 5-2. Approximately 62 percent of the system consists of 8-inch-diameter pipe.

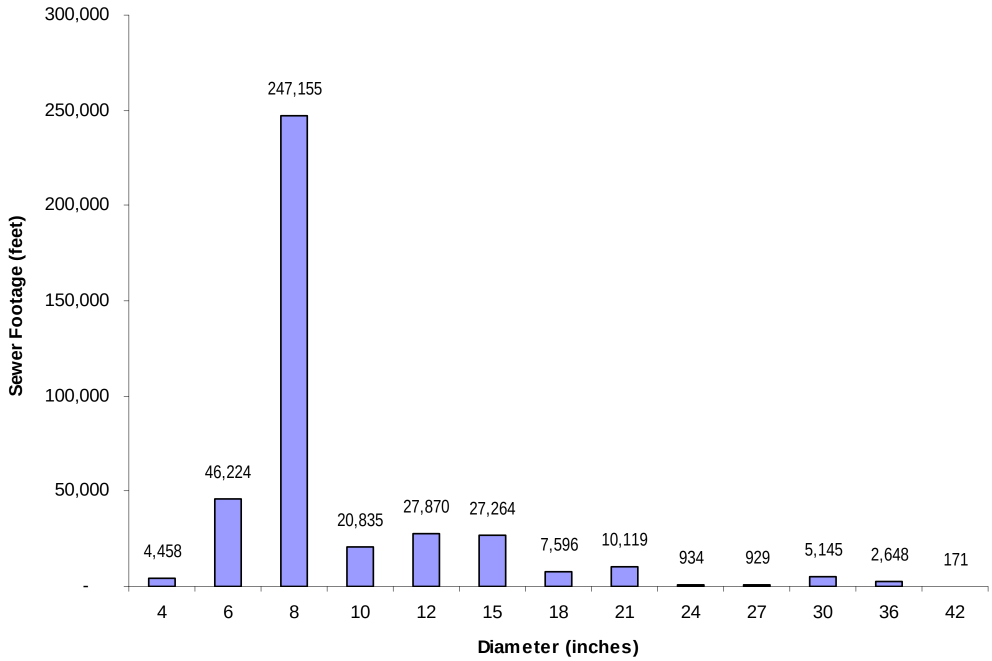


Figure 5-2. Pipe Size Distribution, Sanitary Collection System

The distribution of pipe materials is shown in Figure 5-3. This figure includes the footage of force mains and gravity sewers. Nearly all the ductile iron pipe that is included in the inventory is used for force mains. Most new construction has been made using poly-vinyl chloride pipe. According to City staff, clay pipes constructed during the City's early years have many faulty (leaky) joints.

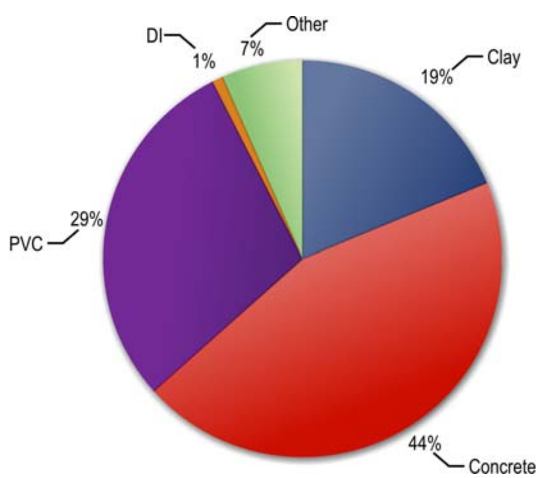


Figure 5-3. Pipe Material Distribution, Sanitary Collection System

5.2.1.1 Collection System Condition

The age distribution of the collection system is shown in Figure 5-4. Approximately 62 percent of the system is less than 30 years old and is in good condition. About 16 percent of the system is over 50 years old with many pipes in service for 80 to 90 years. While the serviceable life of a sanitary sewer is generally assumed to be at least 75 years, pipes deteriorate over time and the effects of this deterioration are evident in the City's collection system. Specifically, the older sections of Newberg were constructed with vitrified clay pipe. The joints in many of these clay pipes have failed, allowing stormwater and groundwater to enter into the sanitary collection system.

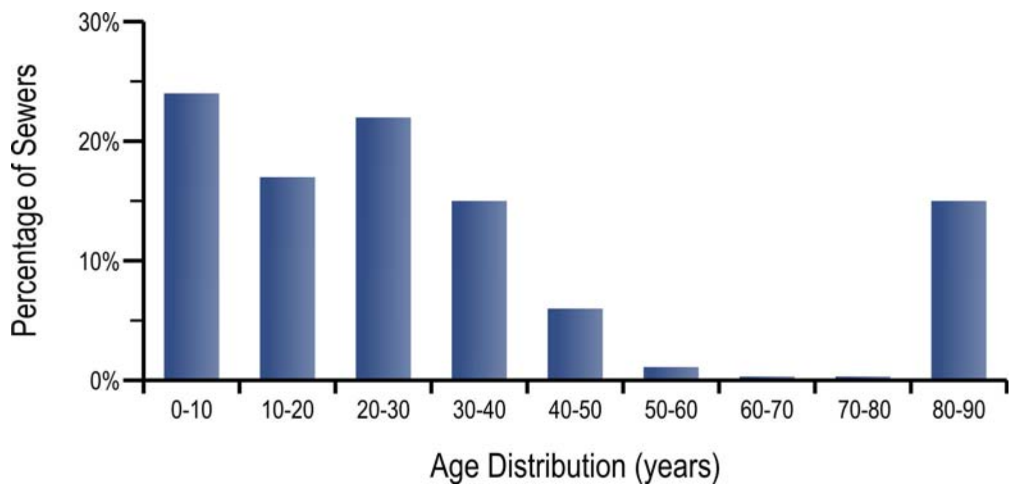


Figure 5-4. Pipe Age Distribution, Sanitary Collection System

The structural and operational condition of the sanitary collection system was documented through interviews with City maintenance staff. Figure 5-5 was prepared to show areas of the sanitary collection system with structural and operational deficiencies. Structural problems include a number of older vitrified clay pipes with broken or missing joints. Operational defects include grease, roots, and I/I. Additional information on the sanitary collection system is provided in the SMPU.

The addition of non-sanitary flows as I/I into the collection system is an operational defect that decreases the available hydraulic capacity in the existing pipes and increases the size and cost of facilities. The extent of I/I is summarized in section 5.4.

5.2.2 Existing Lift Stations

The City has five small lift stations: Andrew Street, Charles Drive, Chehalem Drive, Creekside, and Sheridan Street; and two large lift stations: Dayton Avenue and Fernwood Road. Lift station locations are shown in Figure 5-1. A detailed description of lift station physical and operational information is available in the Technical Memorandum B-1 included as Appendix B of the SMPU.

Complete information was available for all lift stations except Creekside, which was missing wet well depth, bottom elevation, volume, force main elevation, and pump on and off levels.

The hydraulic capacity for each lift station is listed in Table 5-1 along with the predicted flow requirements for the existing condition (2007) and for 2040. Three of the lift stations will require hydraulic upgrades to convey the future flows.

Table 5-1. Lift Station Hydraulic Capacity

Lift station	Current pumping rated capacity[1], gallons per day (gpm)	Model predicted peak flows to wet well[2], gpm		Upgrades required?
		2007 (existing)	2040	
Andrew Street	150	142 (1)	149	No
Charles Drive	150	136 (1)	144	No
Chehalem Drive	630	484 (1)	983	Yes
Creekside	153	50 (1)	56	No
Sheridan Street	105	17 (1)	17	No
Dayton Avenue	2,100	2,356 (2)[3]	2,538	Yes
Fernwood Road	280	725 (2)[4]	3,312	Yes

[1] For each lift station (except Fernwood Road), the rated pumping capacity is based on one-pump operation without the use of the second (redundant) pump. For the Fernwood Road Lift Station, future plans call for this to be a triplex station with one of the three pumps redundant. Use of all the pumps at a lift station does not provide pumping redundancy as per Oregon Department of Environmental Quality (DEQ)/U.S. Environmental Protection Agency requirements.

[2] The values in this column represent the modeled flow into the wet well as predicted by the hydraulic model. The number in parentheses is the number of pumps that would need to run to pump the predicted flow.

[3] Dayton Lift Station lacks capacity to convey the current (2007) flows with both pumps running. Flows listed do not reflect the trunkline flow transfer alternative discussed in Section 5.5.

[4] As modeled, the predicted flows into the Fernwood Road Lift Station exceed current pumping capacity with both pumps operating; however, staff report that there have been no overflows recorded at this pump station.

A condition assessment was performed on each lift station in addition to evaluating the hydraulic requirements. A summary of these findings is included below. The lift stations generally meet DEQ design standards. See Technical Memorandum B-1 included as Appendix B of the SMPU for more information.

5.2.2.1 Andrew Street Lift Station

In 2001, a new station was constructed to replace the original. The maximum rated pumping capacity is 150 gpm for one pump operation. The second pump is the redundant pump. Pump condition is considered to be good. In January 2006, the pumps ran 40.3 percent of the time (about 87,000 gallons per day [gpd]). The station is in good condition and is well maintained.

5.2.2.2 Charles Drive Lift Station

This lift station was completely upgraded in 2001. The maximum rated pumping capacity is 150 gpm for one pump. The second pump is the redundant pump. Pump condition is considered to be good. In January 2006, the pumps ran 29.8 percent of the time (about 64,400 gpd). The station was recently upgraded, is in good condition, and is well maintained.

5.2.2.3 Chehalem Drive Lift Station

This lift station was built in 2004. The maximum rated pumping capacity is 630 gpm for one pump. The second pump is the redundant pump. The pump condition is considered to be good. In January 2006, the pumps ran 8.6 percent of the time (about 78,000 gpd). The station is in good condition and is well maintained.

5.2.2.4 Creekside Lift Station

In 1998 there was an upgrade to the existing lift station. The maximum rated pumping capacity is 153 gpm with one pump. The second pump is the redundant pump. The pump condition is considered to be good. In January 2006, the pumps ran 5.7 percent of the time (about 12,000 gpd). The station is 9 years old, is in good condition, and is well maintained.

5.2.2.5 Sheridan Street Lift Station

This lift station was built in 2001. The maximum rated pumping capacity is 105 gpm with one pump. The second pump is the redundant pump. The pump condition is considered to be good. In January 2006, the pumps ran 2.5 percent of the time (about 4,000 gpd). The station is 6 years old, is in good condition, and is well-maintained.

5.2.2.6 Dayton Avenue Lift Station

This lift station was upgraded in 1993. The maximum rated pumping capacity is 2,100 gpm for one pump and the second pump is redundant. The pump condition is considered to be fair. In January 2006, the pumps ran 75.7 percent of the time (about 2.3 million gpd). The station is in fair condition with some operation and maintenance issues.

5.2.2.7 Fernwood Road Lift Station

This lift station was built in 2001. The maximum rated pumping capacity is 280 gpm with one pump. The second pump is the redundant pump. Future expansion allows for installation of a triplex pump system with one of the pumps redundant. This would provide 1,480 gpm capacity with one pump in operation and 2,100 gpm with two pumps in operation. The pump condition is considered to be excellent. In January 2006, the pumps ran 45 percent of the time (about 194,000 gpd). The station is in excellent condition and is well-maintained.

5.3 MODEL DEVELOPMENT

The City's sewer collection system was modeled to determine the hydraulic capacity of the system for existing and future growth planning scenarios. The hydraulic model was developed to include the main lines within the existing collection system. The City's collection system discharges to the Newberg Wastewater Treatment Plant (WWTP). This section describes the sanitary sewer model, model development, and model updating.

5.3.1 Collection System Model

The City's collection system was modeled using InfoSWMM, which is a product of MWH Soft, Inc. InfoSWMM is a fully GIS-integrated, highly advanced, and comprehensive hydrologic, hydraulic, and water quality simulation model that can be used for the management of urban stormwater and wastewater collection systems. Built atop ESRI ArcGIS, InfoSWMM seamlessly integrates advanced sewer collection system modeling and optimization functionality with the latest generation of ArcGIS. InfoSWMM's direct ArcGIS integration enables powerful GIS analysis and hydraulic modeling in a single environment using a single dataset.

InfoSWMM can be used to model the entire land phase of the hydrologic cycle as applied to urban stormwater and wastewater collection systems. The model can perform single event or long-term (continuous) rainfall-runoff simulations accounting for climate, soil, land use, and topographic conditions of the watershed. In addition to simulating runoff quantity, InfoSWMM can also predict runoff quality, including buildup and washoff of pollutants from primarily urban watersheds. The routing portion of InfoSWMM transports flows using either steady flow routing, kinematic wave routing, or dynamic wave routing through a conveyance system of pipes, channels, storage/treatment devices, pumps, and hydraulic regulators such as weirs and orifices. The model offers advanced Real-Time Control scheme for the operational management of hydraulic structures. While the water quality feature is a component of the model, this feature was not used for the master planning effort.

5.3.5 Model Development

The hydraulic model was developed by importing network components directly from the City's GIS coverage. Specifically, the sewerpoints and sewerlines layers were used. The manholes within the model area were imported directly into the model from the sewerpoints layer. The conduit (pipe) file was built from the sewerlines file. Maps of streets, parcels, and land use were displayed as background images, allowing for confirmation of the network layout. The extents of the hydraulic model are shown in Figure 5-6. Only the major segments of the piped system were included in the model which includes approximately 408 manholes, 409 pipe segments, and seven lift stations. Lift station capacity, number of pumps, and pump on and off levels were obtained from the Lift Station Assessment Technical Memorandum, Brown and Caldwell, October 2006, Revised April 2007.

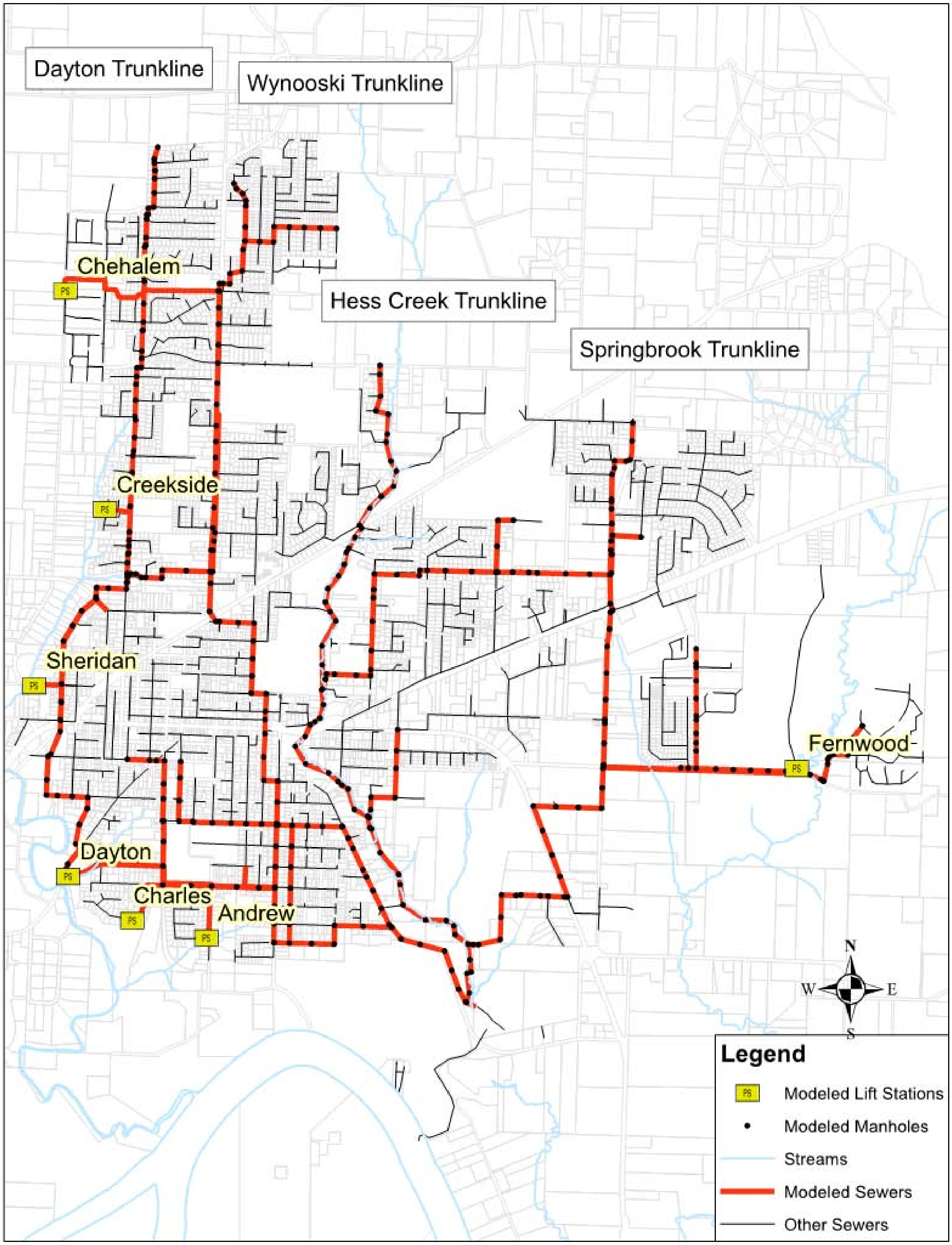


Figure 5-6. Model extents for Newberg collection system

5.4 FLOW PROJECTIONS

This section documents the sewer flow projections developed for the existing and future planning periods. Flow monitoring data, City land use designations, and unit flow factors developed from flow monitoring analyses were used as the basis for calculating existing and future flow projections. The following section describes the wastewater components including base flow projections, rainfall-derived I/I (RDII) projections, model calibration, and future flow projections.

5.4.1 Base Flow Projections

Base wastewater flow (BWF) is sanitary flow generated from residential, commercial, industrial, and public or institutional sources that discharge into the wastewater collection system. It may vary in magnitude throughout the day, but generally follows a predictable and repeatable diurnal pattern with peak flow usually occurring during the morning hours. During the winter, there is very little or no irrigation, such that most of the potable water used by the community is discharged to the collection system. Therefore, BWF was estimated from winter water consumption data. The City supplied total water consumption data for January 2006 to assist in estimating BWF. As part of the calculations, unit flow rates were determined for all major land use designations (single-family residential, multi-family residential, commercial, and industrial). To streamline the flow generation process, the City land use zones identified in the Newberg zoning map (2006) were consolidated for use in developing the flows. Refer to the SMPU for estimated unit flow per capita rates.

5.4.2 Groundwater Infiltration (GWI)

GWI is groundwater that infiltrates into the sewer system through defects in manholes and pipes. GWI rates vary depending on time of year, the condition of the sewers, soil type, and groundwater levels. However, GWI rates stay fairly consistent throughout the day. GWI was calculated as the difference between metered dry weather flow and BWF at each flow meter. The calculated GWI was applied evenly as a flow per acre to the entire area upstream of each flow meter. Table 5-2 summarizes the modeled GWI flow that was used for each site.

Table 5-2. GWI Rates

Flow meter	Area, acre	Infiltration, cfs[1]	Infiltration, cfs per acre
Dayton	423	0.1	0.00024
Wynooski	935	0.16	0.00017
North Central (Hess Creek)	684	0.4	0.00058
Springbrook	891	0.15	0.00017
Total	2,933	0.81	0.00028 (average)

[1]cfs = cubic feet per second

For future areas, GWI was calculated by identifying the subbasin where the future land is located. Then, the corresponding GWI rate from Table 5-2 was multiplied by the future land area to calculate the GWI flow.

5.4.3 RDII

RDII consists of stormwater that enters the collection system either as direct inflow of stormwater runoff or rainfall-induced infiltration. Inflow occurs when stormwater flows directly into the collection system through connected catch basins, manhole covers, area drains, or downspouts. Inflow usually occurs very rapidly during a storm event and can become more severe if surface flooding occurs and manholes are submerged. Rainfall-induced infiltration is caused by stormwater that percolates through the ground and enters the sewer pipes, manholes, and service laterals through cracks and defective joints.

According to DEQ regulations, collection systems should be designed to handle the peak flows generated by the one-in-5-year, 24-hour storm event. This peak flow consists of base flow, GWI, and RDII. The calculated RDII rates were applied evenly as a flow per acre to the entire area upstream of each flow meter. The calculated RDII for the four trunklines is listed in Table 5-3.

The RDII for the future areas assumes that the new pipes are less leaky than the existing collection system. For these areas, RDII was calculated as three times the sum of the BWF and GWI. This yielded a total peak flow of four times the dry weather flow.

Table 5-3. Five-year, 24-hour peak RDII rates

Flow meter	Area, acre	Peak I/I, cfs	Peak I/I, cfs per acre
Dayton	423	4.25	0.010
Wynooski	935	12.9	0.014
North Central (Hess Creek)	684	9.73	0.014
Springbrook	891	2.89	0.0032
Total	2,933	29.7	0.010 (average)

5.4.4 Hydrologic Modeling

Hydrologic models were developed to simulate the response of the sanitary collection system to sanitary, groundwater, hydrologic, and rainfall-derived flows. The Rainfall-Flow Regression Method was used to estimate RDII. Once constructed and calibrated over one full wet season of flow data, the models were used to project flows under wet weather conditions for existing and future conditions. Further detail on model development can be found in Chapter 4 of the SMPU.

5.4.5 Existing and Future Flows

Three different planning horizons were evaluated: existing (2007), 2025, and 2040. Existing and future flows were based on existing water use data, GWI determinations, and peak RDII flows as described above. Chapter 2 of the SMPU describes the area and land use associated with each of the planning horizons. Table 5-4 summarizes the existing and future flows for each main trunkline and Appendix E of the SMPU summarizes the existing and future flows for each input node in the hydraulic model.

Table 5-4. Flows per Trunkline for Existing and Future Conditions

	BWF, cfs			GWI, cfs			RDII, cfs			Total		
Sub-basin	**Existing**	**2025**	**2040**	**Existing**	**2025**	**2040**	**Existing**	**2025**	**2040**	**Existing**	**2025**	**2040**
Dayton[1]	0.47	0.55	0.55	0.10	0.11	0.12	4.25	4.54	4.54	4.82	5.20	5.21
Wynooski[1]	0.93	1.66	2.32	0.16	0.26	0.32	12.85	15.33	17.50	13.94	17.25	20.14
North Central (Hess Creek)	0.36	1.11	1.33	0.40	0.63	0.73	9.73	12.66	13.60	10.49	14.40	15.65
Springbrook	0.59	2.52	2.98	0.15	0.30	0.37	2.89	9.13	10.71	3.63	11.95	14.06
Totals, cfs	2.35	5.84	7.18	0.81	1.30	1.52	29.72	41.66	46.35	32.88	48.80	55.06
Totals, mgd	1.52	3.77	4.64	0.52	0.84	0.98	19.21	26.93	29.96	21.25	31.54	35.59

[1] Flows are based on the existing 2007 conditions and do not include the proposed Dayton trunkline to Wynooski trunkline flow transfer discussed in Section 5.5.

RDII contributes approximately 90 percent of total existing flows during the one-in-5-year, 24-hour storm event. GWI provides a much smaller contribution at approximately 2.5 percent. Although this information is based on flow monitoring performed over dry and wet periods of time, it is not possible to establish the I/I contributions derived from only inflow.

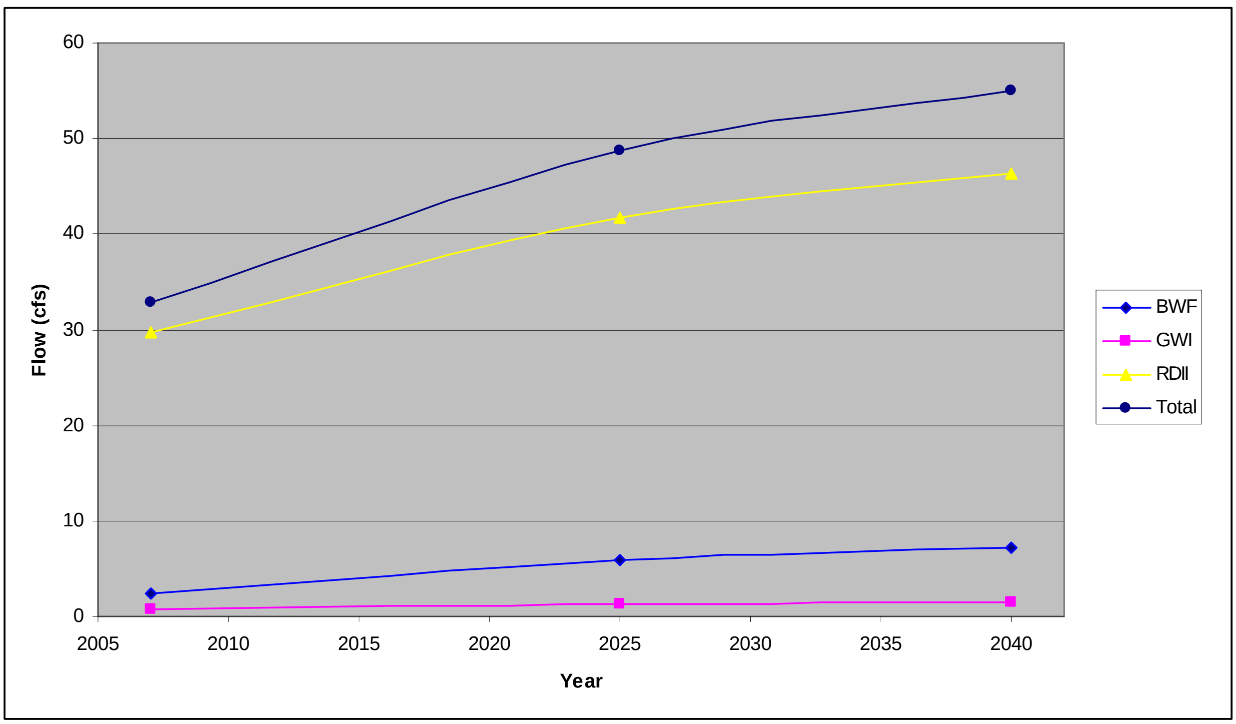


Figure 5-7. Flows for Existing and Future Conditions

As shown in Figure 5-7, the rate of flow increase is not linear (constant) with City growth. The slopes of the RDII and GWI curves do not increase proportionally with the City's growth. It is assumed that the new sewer pipes installed with growth will allow less I/I than the existing sewers.

5.5 HYDRAULIC ANALYSIS

This section documents the results of the hydraulic analysis used to evaluate the existing collection system under existing and future flow conditions for the SMPU. The hydraulic analysis included consideration of a flow transfer between trunklines to reduce the immediate and long-term capital cost of required improvements.

5.5.1 Assessment Criteria

This section discusses the criteria used to determine the adequacy of existing and future collection system infrastructure.

The ratio of maximum predicted flow (Q) to pipe capacity (Qm) is used as the primary parameter to identify undersized sewers. The Q/Qm index compares the calculated peak flow in each pipe with the theoretical pipe capacity according to Manning's equation, which assumes unpressurized flow (no surcharging). A ratio of greater than 1 indicates that the pipe is carrying more flow than is theoretically possible for unpressurized flow for a given pipe slope, diameter, and internal roughness. A Q/Qm ratio of greater than 1.0 is an indication of a surcharged pipe.

In an unpressurized pipe, or a pipe with open-channel flow characteristics, the hydraulic grade line (HGL) is the elevation of the water surface within the pipe. In a pipe that is surcharged (pressurized flow), the HGL is defined by the elevation to which water would rise in an open pipe, or manhole, as shown in Figure 5-8. In hydraulic terms, the HGL is equal to the pressure head measured above the crown of the pipe.

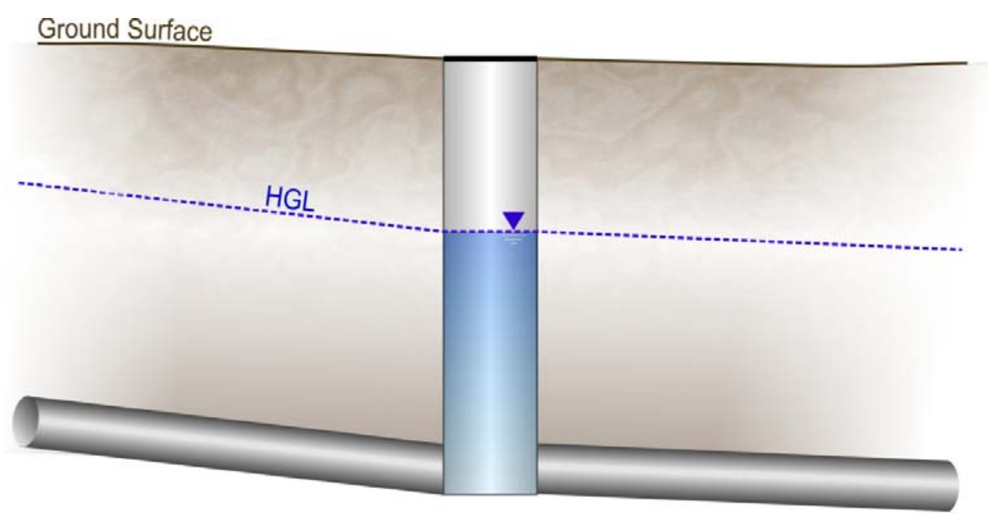


Figure 5-8. HGL for Surcharged Condition

The pipe replacement criterion for the SMPU was to replace all surcharged pipes with larger pipes, or to recommend other alternatives such that the HGL is contained within the pipe. This approach will help ensure that the City has adequate capacity for conveying the design flows. Allowing the sewers to surcharge would increase the potential for sanitary sewer overflows (SSOs), including basement backups and spills to the environment, and would decrease the structural support of the surrounding soil over the long term.

Lift stations were modeled based on existing wet well and pump operational data. Thus, recommendations for increased pumping capacity were made when influent flows to the wet well exceeded existing stated capacities.

Force mains were upsized when velocities exceeded 7 feet per second.

5.5.2 Trunkline Flow Transfer

In 2008, after completion of the 2007 Sewerage Master Plan Update, City staff conceived and developed the capital improvement alternative of removing a portion of the flow from the Dayton trunkline near Highway 240 and inserting this flow into the Wynooski trunkline in order to reduce the costs of capital improvements in the Dayton trunkline including required upgrades to the Dayton Lift Station and force main. To address the immediate under capacity issues in the Dayton trunkline, the first phase of the Highway 240 Lift Station alternative would transfer up to 600 gpm to the Wynooski trunkline. By 2040, the lift station would be expanded to transfer up to 1,000 gpm to convey the planned increase in flows resulting from the higher population. Figure 5-9 shows the location of the proposed lift station and the route of the force main.

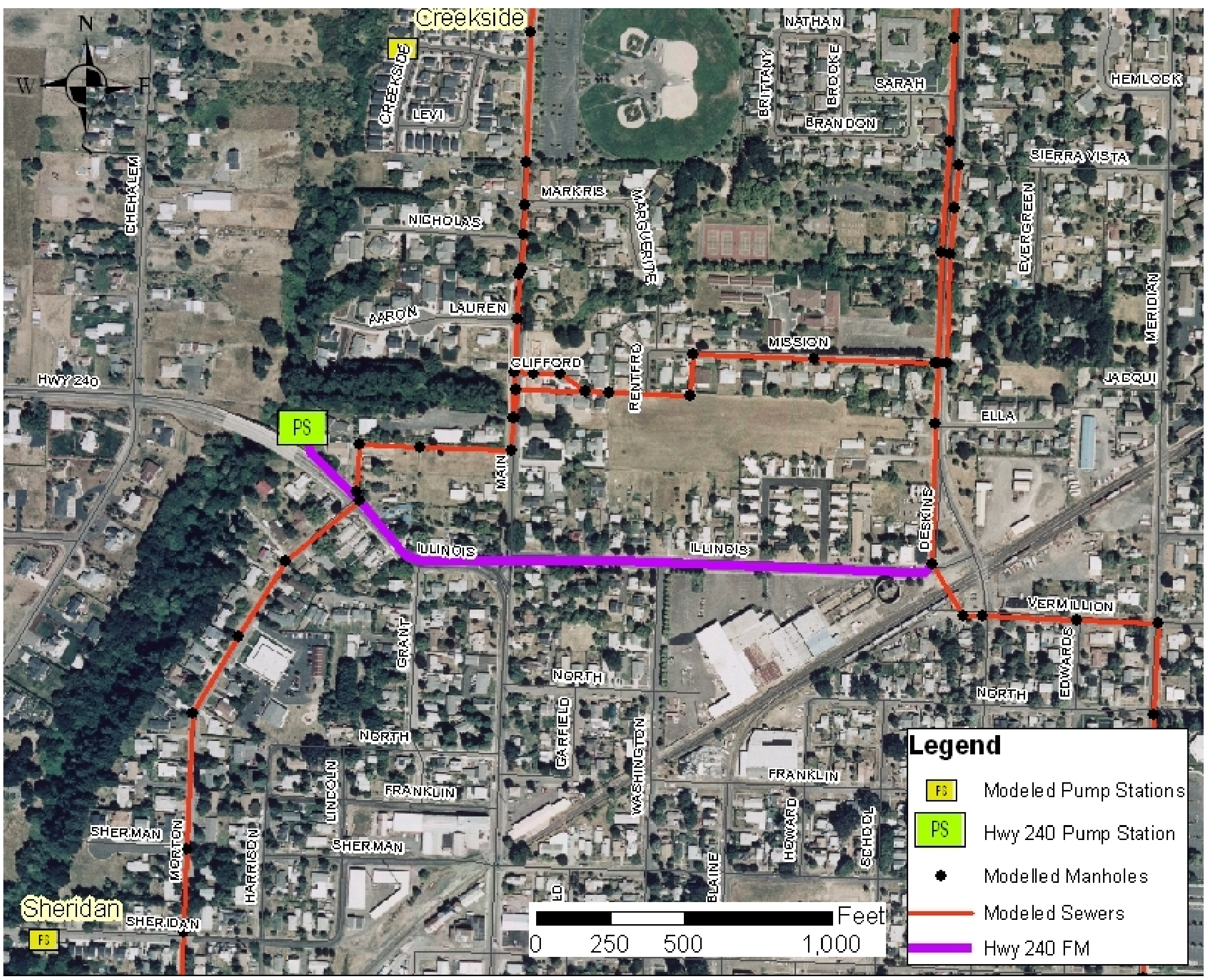


Figure 5-9. Location of the Proposed Lift Station and Force Main Route

As shown, the proposed alternative would result in a net savings to the City of approximately $4.2 million based on the future 2040 planning scenario. Based on the forecasted savings, in 2009 the City decided to move forward with the design and construction of this alternative. This decision required that the SMPU be appended in 2009 to reflect this flow transfer. Consequently, all of the flow and capital improvement recommendations in the SMPU and in the following sections of this document reflect this flow transfer decision.

5.5.3 Existing Collection System Modeling Results

The hydraulic modeling results for the existing condition scenario are discussed below. The detailed results for the current (existing) conditions planning scenario are provided in Appendix G of the SMPU. Undersized gravity sewers are shown in Figure 5-10 for all planning scenarios (i.e., existing [2007], 2025, and 2040.)

5.5.3.1 Gravity Sewers

In addition to identifying pipes that should be replaced, the existing planning scenario analysis should be used to help identify a priority ranking of capital projects. Pipes that are undersized for current conditions should be upsized prior to pipes undersized for future flows.

5.5.3.2 Lift Stations and Force Mains

Under existing conditions and prior to the proposed flow transfer from the Dayton trunkline to the Wynooski trunkline, the modeling shows that the Dayton Avenue Lift Station and the Fernwood Road Lift Station require upsizing to convey existing peak flows. Upon completion of the proposed Highway 240 Lift Station and force main, the resulting flow transfer will eliminate the need to make improvements to the Dayton Lift Station and force main. City staff report that the Fernwood Road Lift Station has not experienced capacity problems. Consequently, improvements to this lift station will be delayed until the flows near pumping capacity.

5.5.4 Future Collection System Modeling Results

The results of the future 2025 and 2040 modeling are provided in this section.

5.5.4.1 Gravity Sewers

Detailed modeling results and recommended improvements are provided in the SMPU, Appendices G, H, and I, for the existing, 2025 and 2040 planning scenarios, respectively.

Please keep in mind that the 2040 planning horizon should be consulted when selecting pipe sizes.

5.5.4.2 Lift Stations and Force Mains

Under the 2025 and 2040 peak flows, the Chehalem Drive and Fernwood Road Lift Stations will be undersized and will require improvements. The planned expansion of the Fernwood Road Lift Station includes switching over to the new 12-inch force main. All other force mains are adequately sized for future flows. Specific flow information for each lift station is listed in Table 5-1.

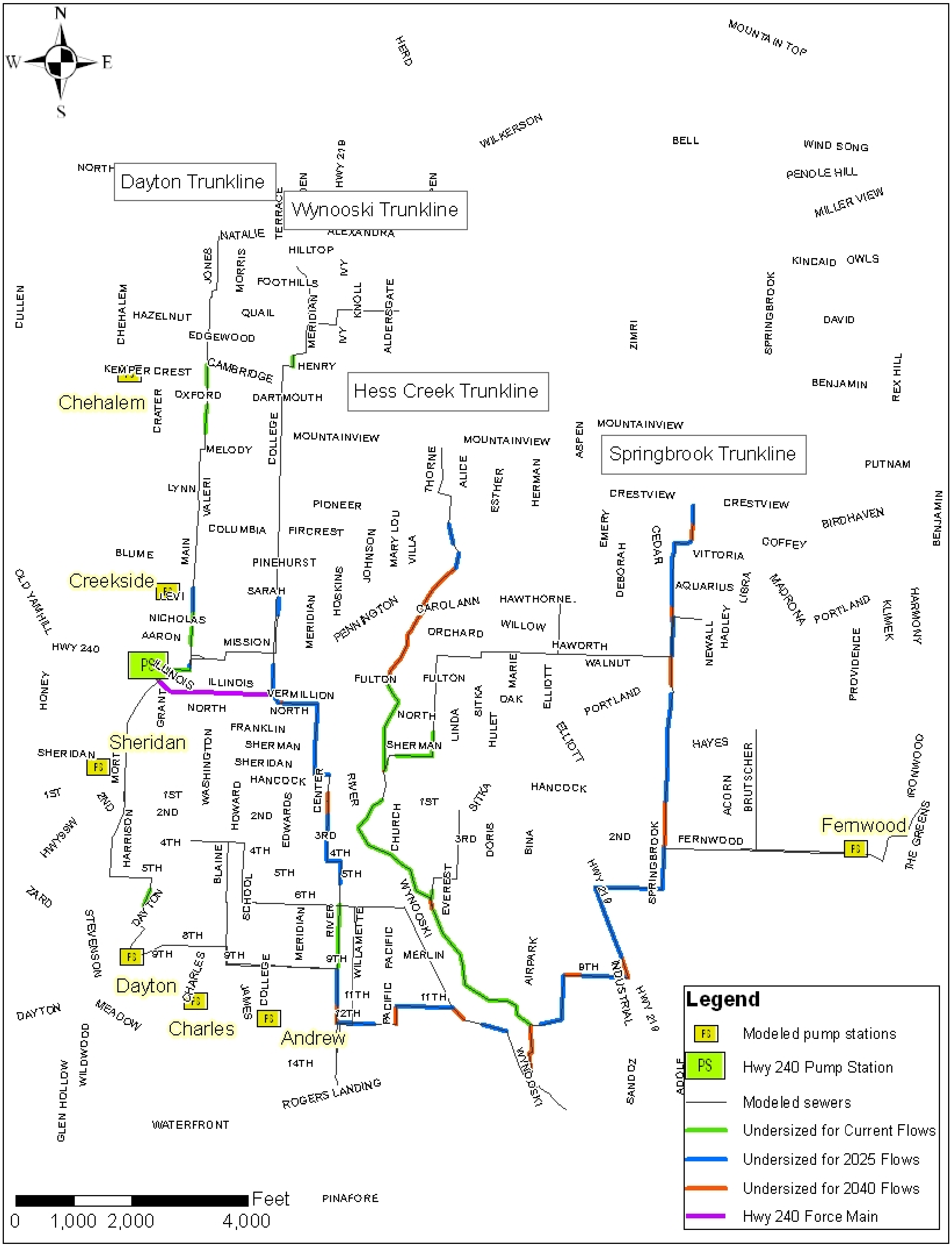


Figure 5-10. Undersized Sewers

5.6 CAPITAL IMPROVEMENT RECOMMENDATIONS

This section presents the recommended capital improvement plan for the City's sanitary sewer collection system. The plan addresses existing deficiencies in the system and provides guidance for expanding the system to meet the City's future growth needs. The projects include pipe replacements and lift station improvements for conveying the projected flows. Other improvements are recommended to address the needs of aging elements of the collection system and to reduce the amount of I/I that enters the system.

Capital improvements were developed in the SMPU for three planning scenarios: existing (2007), 2025, and 2040. It is understood that the DEQ facilities plan requirements call for a 20-year planning horizon for the Newberg WWTP which would fall on the year 2030, but major conveyance pipes could be designed with a 30- to 50-year planning horizon. The SMPU recommended improvements to the collection system that were based on the 2040 planning horizon, representing a 33-year look into the future.

All pipe replacement recommendations were based on sizing pipes to convey the 2040 flow. A detailed listing of the recommendations is included in Chapter 6 of the SMPU. The recommended replacement pipes are grouped into project packages that facilitate design and bid activities. Each package typically includes two or more contiguous pipes with a project package cost in the range from about $300,000 to $600,000 in design and construction costs. The following figures, located at the end of this section, illustrate the location of capital improvement recommendations. Figure 5-11 shows pipe upsizing recommendations to meet existing (2007) flows. Figure 5-12 shows recommended system extensions required by 2025. The extensions connect new areas brought into the service area to the existing collection system. Figure 5-13 shows pipes that require upsizing to convey the 2040 flows. Figure 5-14 shows system extensions in 2040.

In addition to pipe replacement, the modeling effort identified hydraulic deficiencies at some of the City's existing lift stations. The improvements and costs for expanding the capacities of these lift stations are included in Chapter 6 of the SMPU.

Another recommendation of the SMPU is the implementation of a rehabilitation and replacement (R&R) program to address the needs of an aging sanitary collection system that has structural and operational deficiencies, including conditions that allow for unacceptable levels of I/I. The R&R program will focus on restoring the pipes to good structural and operational condition while reducing the amount of I/I that enters the system. Sufficient re-investment in the sanitary collection system through the R&R program will reduce sewer maintenance requirements, decrease the potential for catastrophic failures, and delay expenditures at the WWTP.

A priority ranking of projects is included in Chapter 6 of the SMPU. In general, the projects are ranked in accordance with when increased capacity will be required. City staff should re-prioritize the list each year to ensure that specific needs are addressed appropriately.

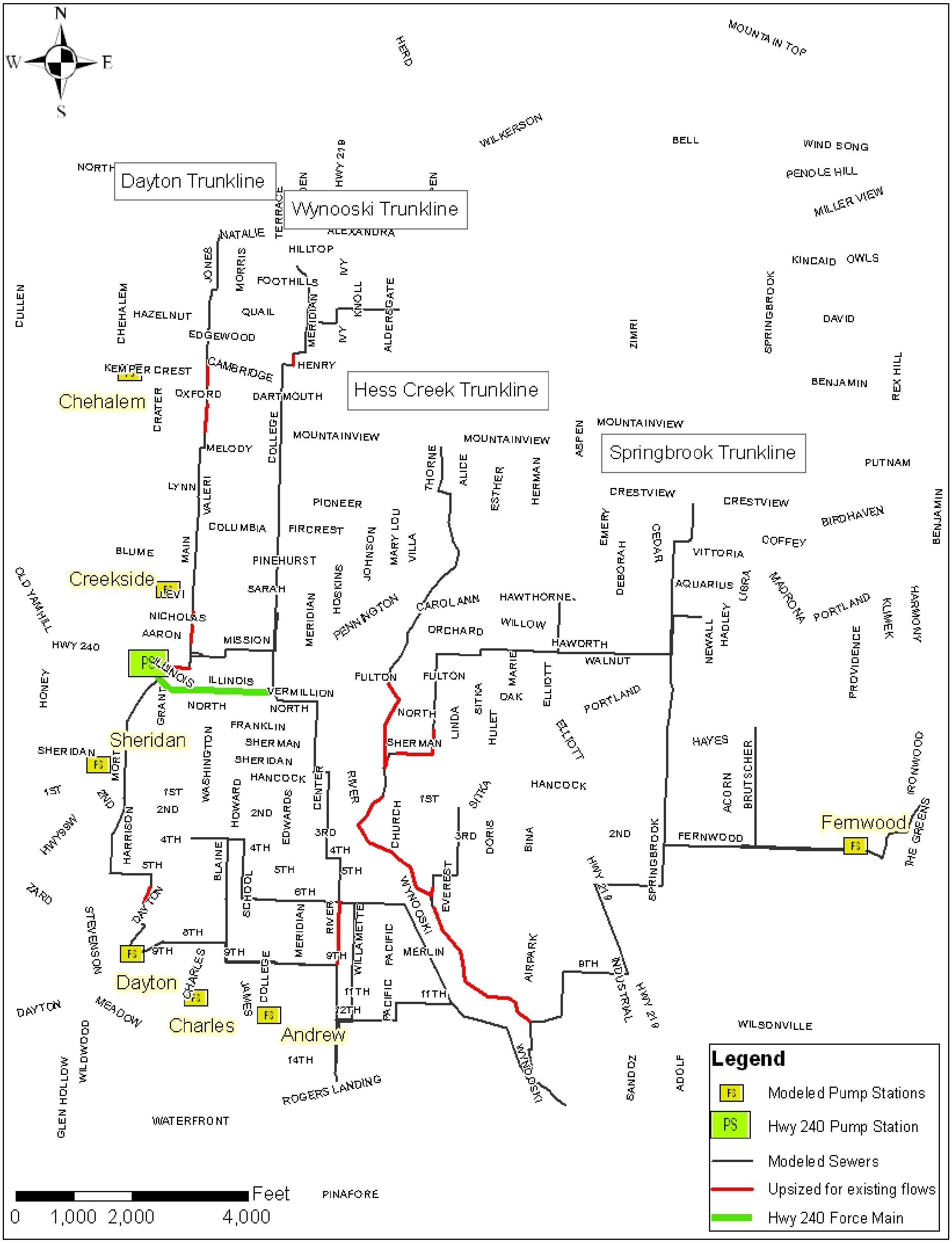


Figure 5-11. Existing (2007) Planning Horizon Recommendations

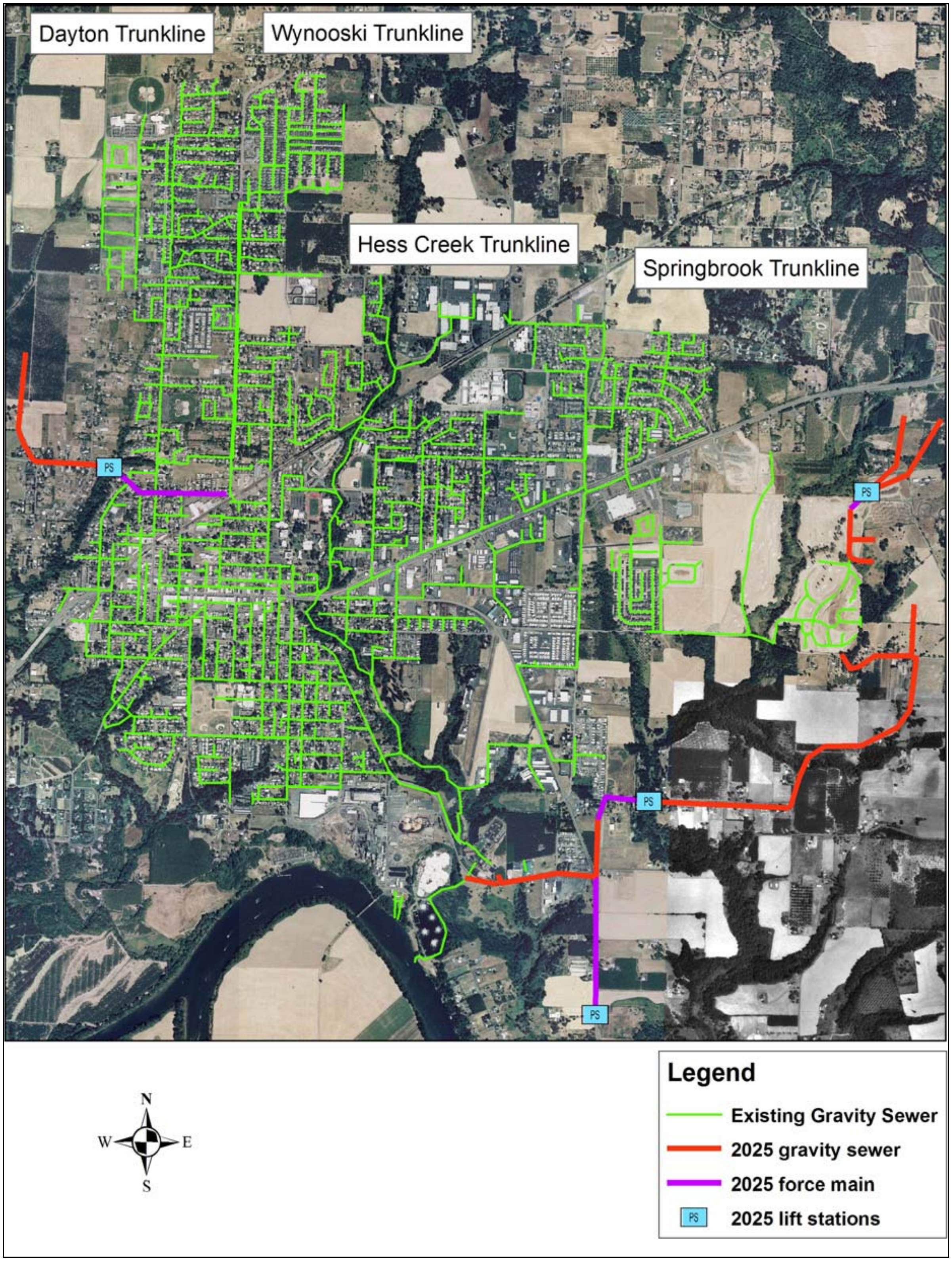


Figure 5-12. Trunkline Extensions, 2025

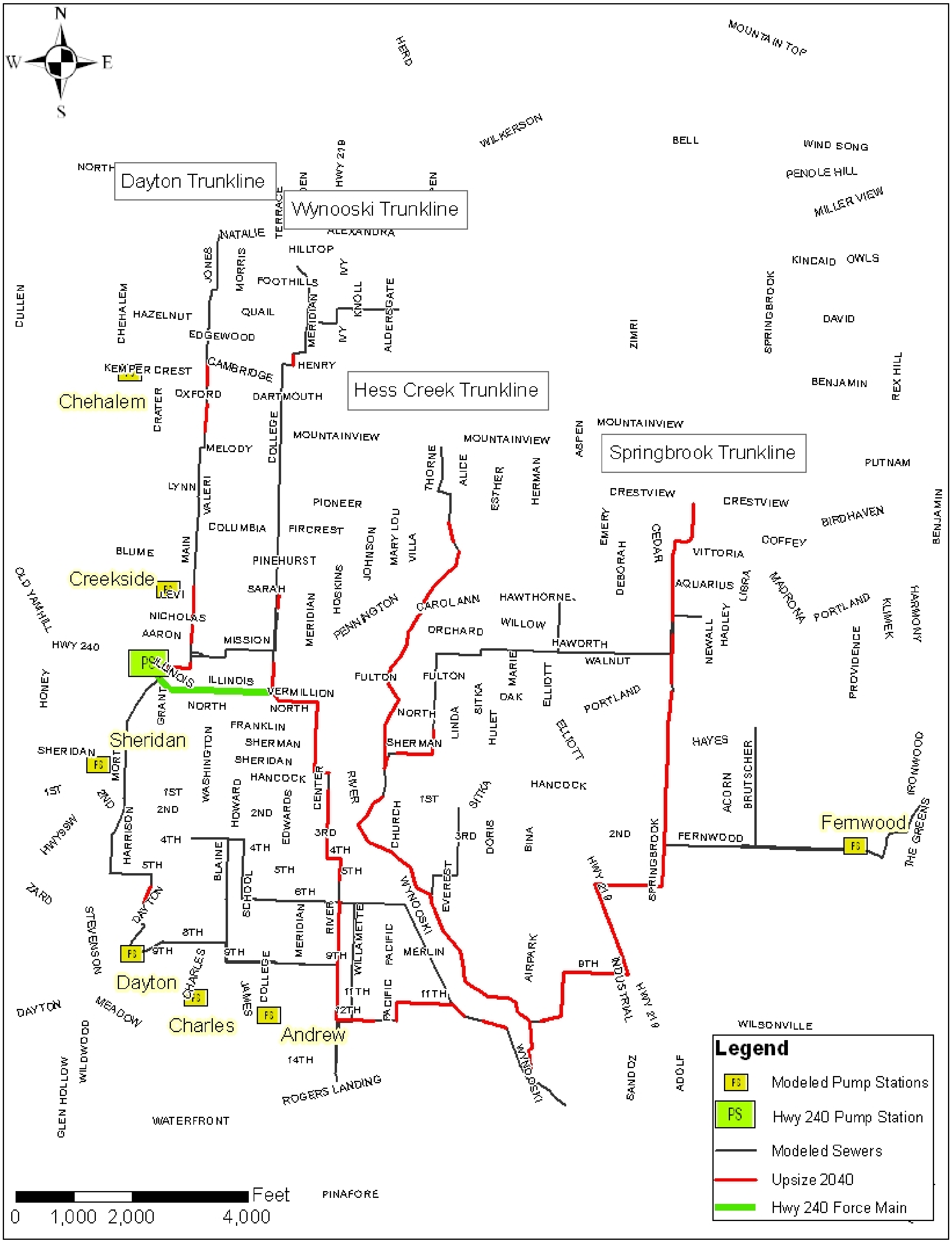


Figure 5-13. Capital Improvement Recommendations, 2040

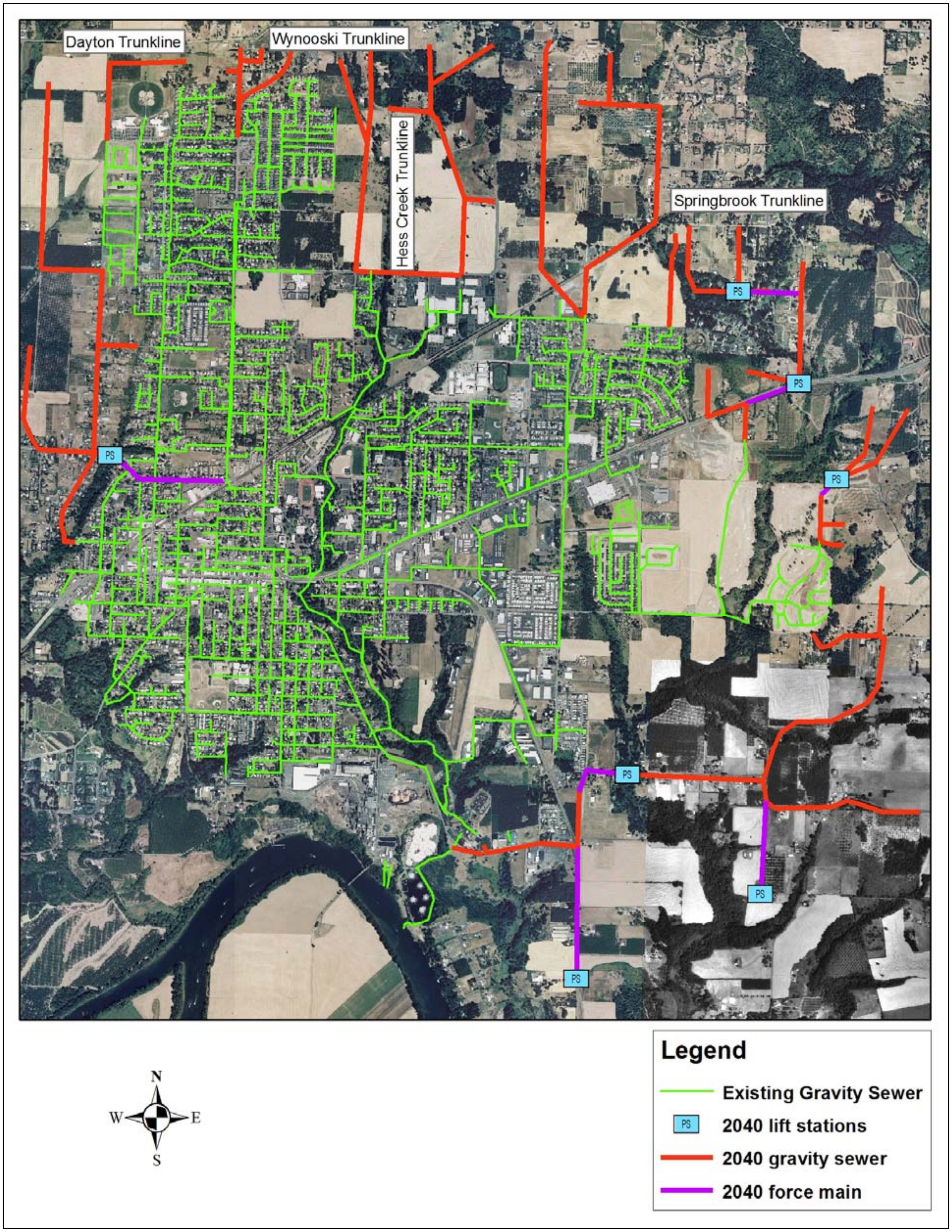


Figure 5-14. Trunkline Extensions, 2040

Table 5-5 lists the total cost of recommendations by category, including pipe replacement, lift station improvements, and collection system extensions (trunklines and lift stations) for the 2025 and 2040 planning scenarios. Also listed are the annual costs for implementing an R&R program.

Table 5-5. Total Cost of SMPU Recommendations

Component	Estimated cost of improvements, dollars
Pipe replacement, 2040	21,828,000
Lift station upgrades, 2040	3,374,000
Collection system extensions, 2025	9,641,000
Collection system extensions, 2040	21,838,000
Total	56,681,000
Annual costs	
R&R program	1,100,000

5.6.1 Plan to Eliminate SSOs

The original SMPU identified two major areas undersized for current conditions that will require upgrades to prevent SSOs. The planned construction of the Highway 240 Lift Station and force main will eliminate the need to make upgrades to the Dayton Lift Station and force main and to some of the pipes upstream from the lift station (see Section 5.5.2 for more details). The other major area requiring upgrades is along the lower reaches of Hess Creek where City staff report that SSOs have occurred during large storm events. In support of this latter finding, the modeling performed for the SMPU shows extensive surcharging of the Hess Creek trunkline under the existing build-out condition, one-in-5 year, 24-hour storm event with the HGL breaking the ground surface at several locations.

Long-term solutions to the Hess Creek trunkline problem include upsizing the existing pipes to larger diameters that can convey the flow, reducing the flow in the trunkline by redirecting portions of the flow, or possibly a combination of these two approaches. The City is currently evaluating the alternatives to determine the best possible solution. In addition, the solution will most likely involve acquiring environmental regulatory permits to perform work in and near to Hess Creek. The City is exploring the time and effort that may be required to get the necessary permits.

In the near-term for the Hess Creek trunkline, the City plans on securing manhole covers to their frames and tying the manhole concrete structures together so that they do not pull apart during surcharging. This activity will keep sanitary flow in the pipe while protecting the collection system from possible damage.

The City's implementation of these planned activities will eliminate SSOs by 2010.

5.6.2 I/I Cost-Effectiveness Analysis

Experience from other Oregon communities and from across the U.S. has shown that cost-effective I/I reduction is limited to inflow removal. Based on this knowledge, an I/I cost-effective analysis was determined to be unnecessary and not a good use of City funds because most of the I/I is believed to be infiltration. Regardless, the City understands the importance of reducing I/I in the collection system. To this end, SMPU recommends that the City implement an I/I reduction program. that The I/I reduction program would be linked with the City's sewer rehabilitation program to address structural problems as well as I/I reduction.

The I/I reduction program outlined in the SMPU calls for additional flow monitoring to pin-point areas with high I/I contributions. Other techniques to identify I/I sources would include smoke testing, dye testing, and additional visual inspections. A priority ranking for sewer and manhole repairs and rehabilitation would be developed so that repairs would be performed first in areas with the highest I/I contribution. Additional information on the proposed program may be found in Appendix D of the SMPU.